APPENDIX A

ALTERNATIVE SKETCHES

SUMMARY

This appendix contains concept level sketches for the alternatives presented in the main report:

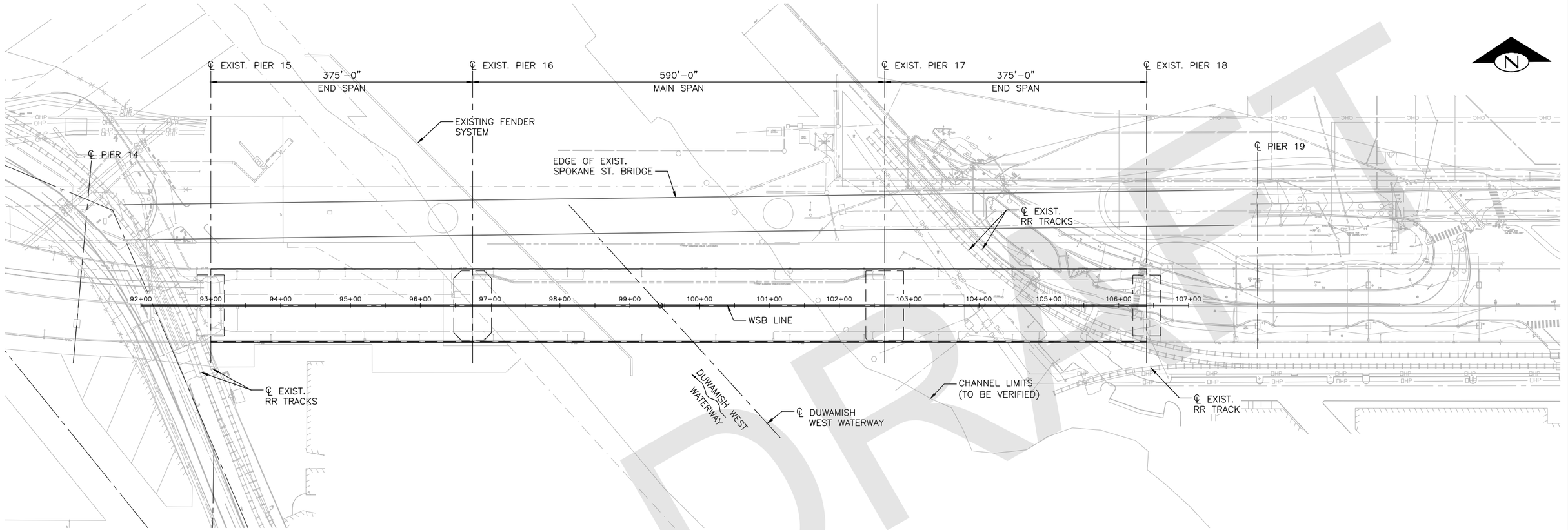
- Alternative 1: Temporary Shoring to Restore Live Load
- Alternative 2: Direct Strengthening to Restore Live Load
- Alternative 4: Accelerated Superstructure Replacement
- Alternative 5: Accelerated Bridge Replacement

Alternative 3, which consisted of replacing the center portion of the mid span with a steel box girder was deemed unfeasible early in the process and not developed further.

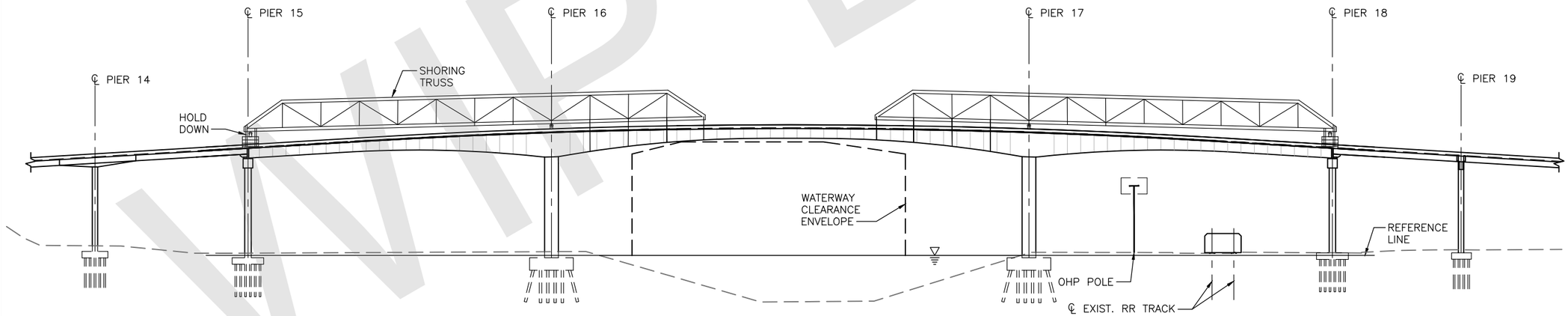
Additionally, this appendix contains concept level sketches describing:

- Standard Demolition Concept Sequence
- Proposed Retrofit

WSP USA
Suite 300
33301 Ninth Avenue South
Federal Way, WA 98003-2600
wsp.com



PLAN – SHORING TO RESTORE LIVE LOAD
CONCEPT ONLY, NOT TO SCALE



ELEVATION – SHORING TO RESTORE LIVE LOAD
CONCEPT ONLY, NOT TO SCALE

WIP DRAFT

ALTERNATIVE 1

WSP USA Inc.
1301 Fifth Avenue
Suite 1200
Seattle, WA 98101-2677
TEL: 206-357-5600
FAX: 206-357-5601

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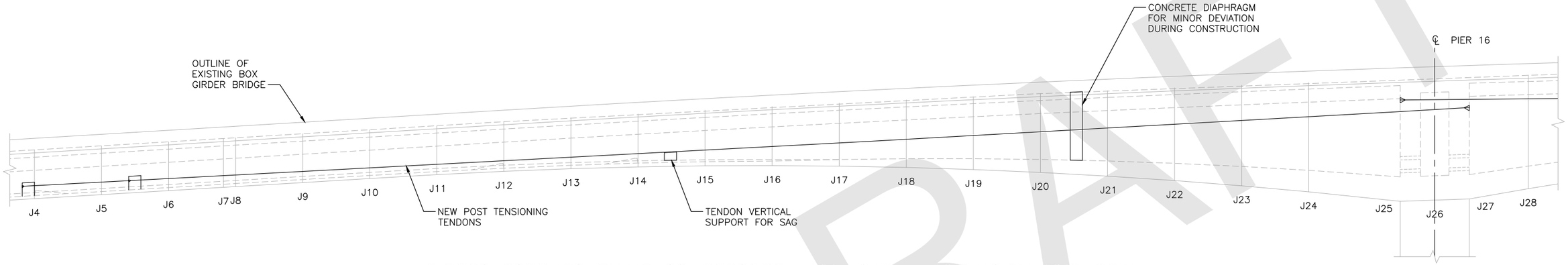
SCALE:

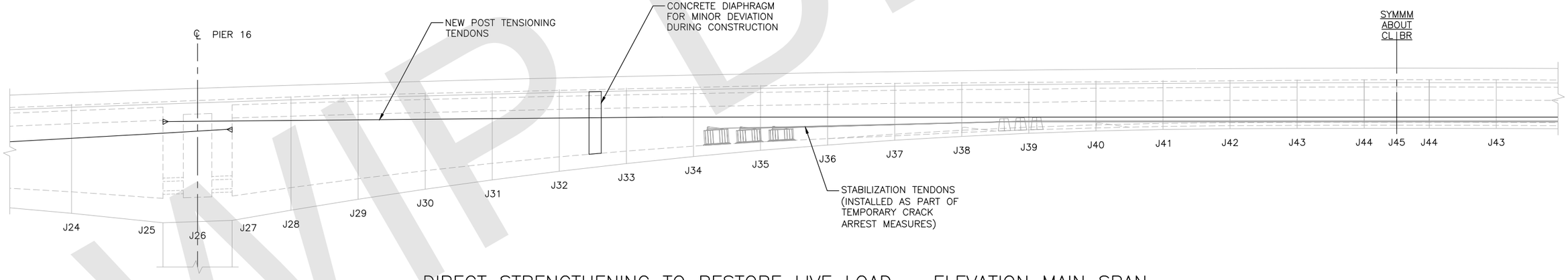
WEST SEATTLE HIGH RISE BRIDGE COST BENEFIT ANALYSIS

TEMPORARY SHORING TO RESTORE LIVE LOAD

JOB PC CO
VPI #
SHEET 1 OF 6



DIRECT STRENGTHENING TO RESTORE LIVE LOAD – ELEVATION END SPAN
CONCEPT ONLY, NOT TO SCALE



DIRECT STRENGTHENING TO RESTORE LIVE LOAD – ELEVATION MAIN SPAN
CONCEPT ONLY, NOT TO SCALE

WIP DRAFT

ALTERNATIVE 2

WSP USA Inc.
1301 Fifth Avenue
Suite 1200
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FAX: 206-357-5601

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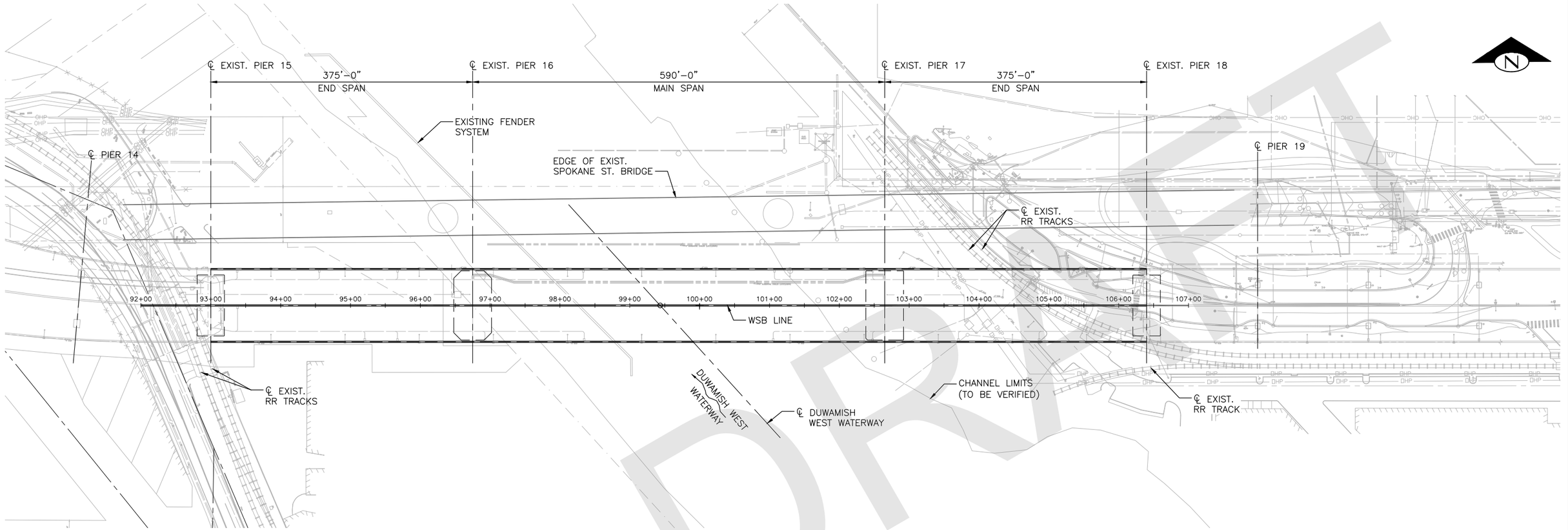
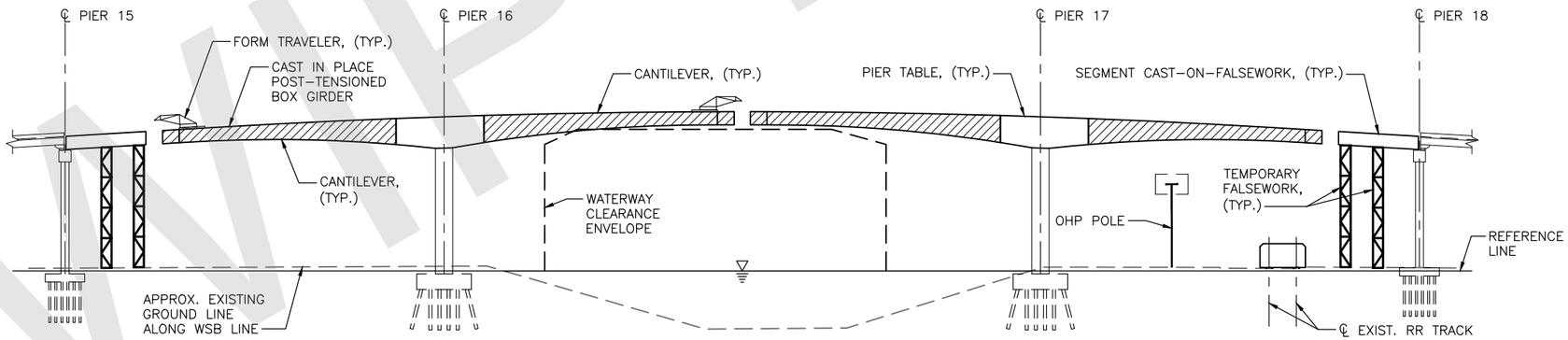
WEST SEATTLE HIGH RISE BRIDGE COST BENEFIT ANALYSIS

DIRECT STRENGTHENING

JOB PC CO
VPI #
SHEET 2 OF 6



PLAN – ACCELERATED SUPERSTRUCTURE REPLACEMENT
CONCEPT ONLY, NOT TO SCALE



ELEVATION – ACCELERATED SUPERSTRUCTURE REPLACEMENT
CONCEPT ONLY, NOT TO SCALE

ALTERNATIVE 4

WP DRAFT

WSP USA Inc.
1301 Fifth Avenue
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Seattle, WA 98101-2677
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FAX: 206-357-5601

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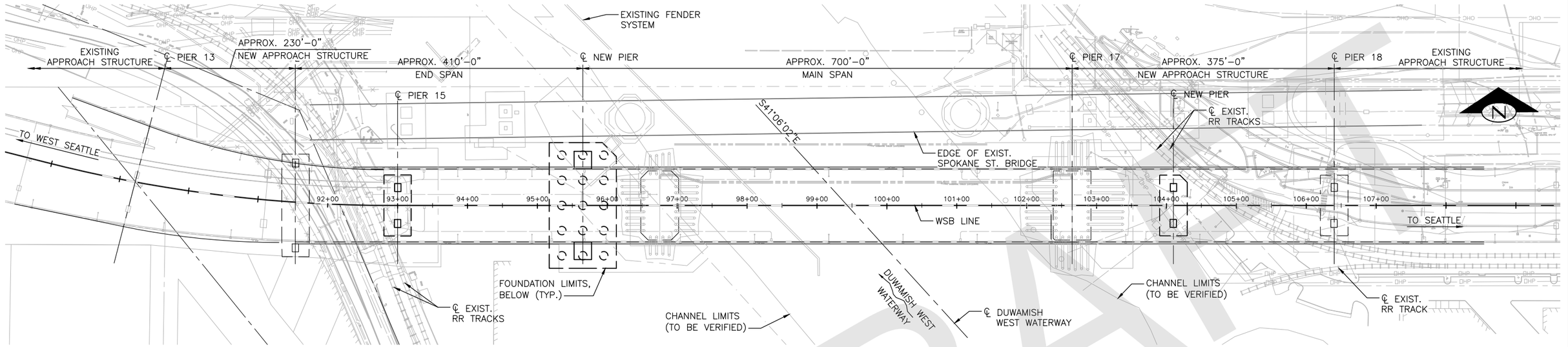
SCALE:

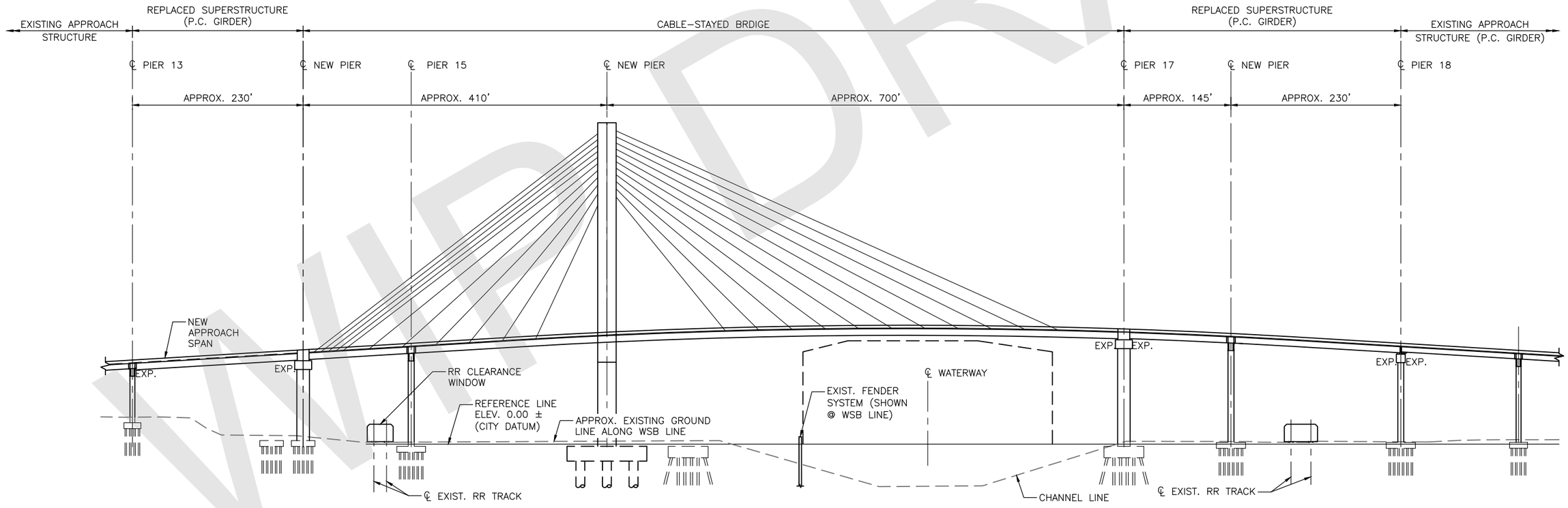
WEST SEATTLE HIGH RISE BRIDGE COST BENEFIT ANALYSIS

FULL SUPERSTRUCTURE REPLACEMENT

JOB PC CO
VPI #



PLAN – IN–KIND BRIDGE REPLACEMENT
CONCEPT ONLY, NOT TO SCALE



ELEVATION – IN–KIND BRIDGE REPLACEMENT
CONCEPT ONLY, NOT TO SCALE (GRADE ELEVATIONS SHOWN ARE AT TOP OF ROADWAY)

ALTERNATIVE 5

DRAFT

WSP USA Inc.
1301 Fifth Avenue
Suite 1200
Seattle, WA 98101-2677
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FAX: 206-357-5601

APPROVED FOR ADVERTISING
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DEPARTMENT OF FINANCE & ADMINISTRATIVE SERVICES
SEATTLE, WASHINGTON . . . . . 20 . . .
BY: . . . . . PURCHASING AND CONTRACTING DIRECTOR

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WEST SEATTLE HIGH RISE BRIDGE COST BENEFIT ANALYSIS

ACCELERATED BRIDGE REPLACEMENT

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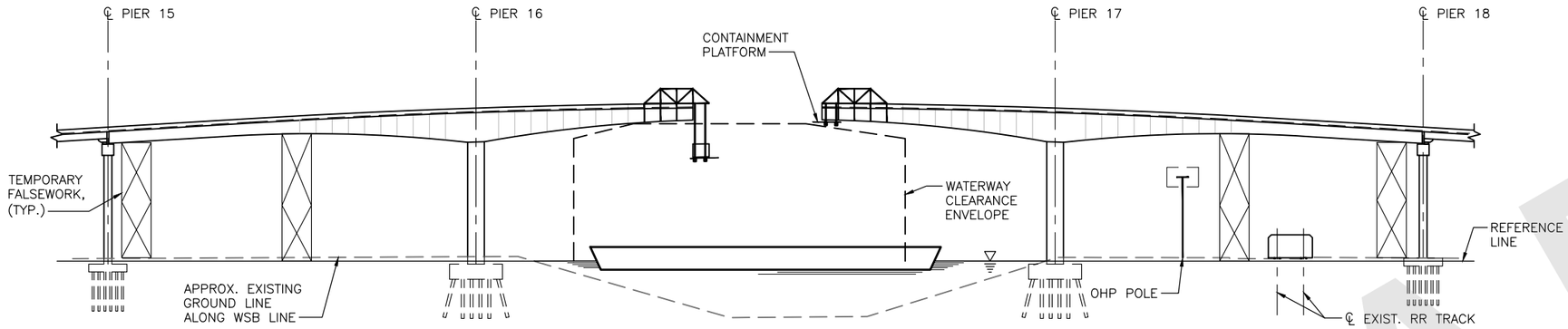
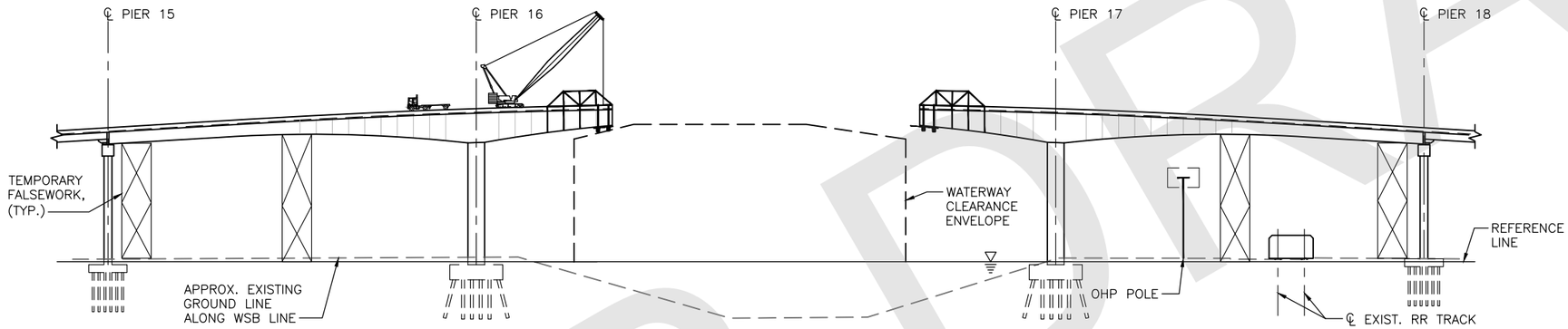
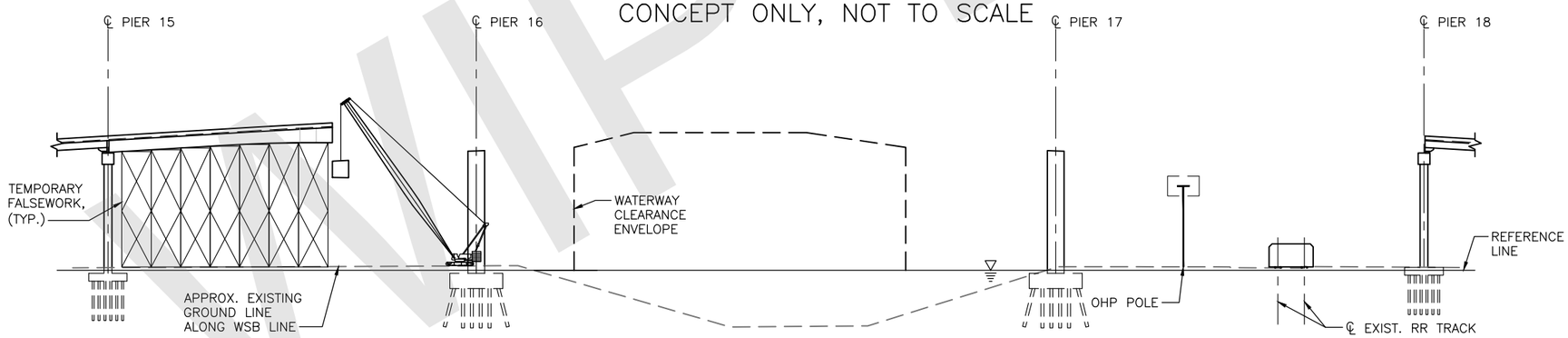
VPI #
SHEET 4 OF 6



ELEVATION – DEMO STAGE 1
CONCEPT ONLY, NOT TO SCALE



ELEVATION – DEMO STAGE 2
CONCEPT ONLY, NOT TO SCALE



ELEVATION – DEMO STAGE 3
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1301 Fifth Avenue
Suite 1200
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FAX: 206-357-5601

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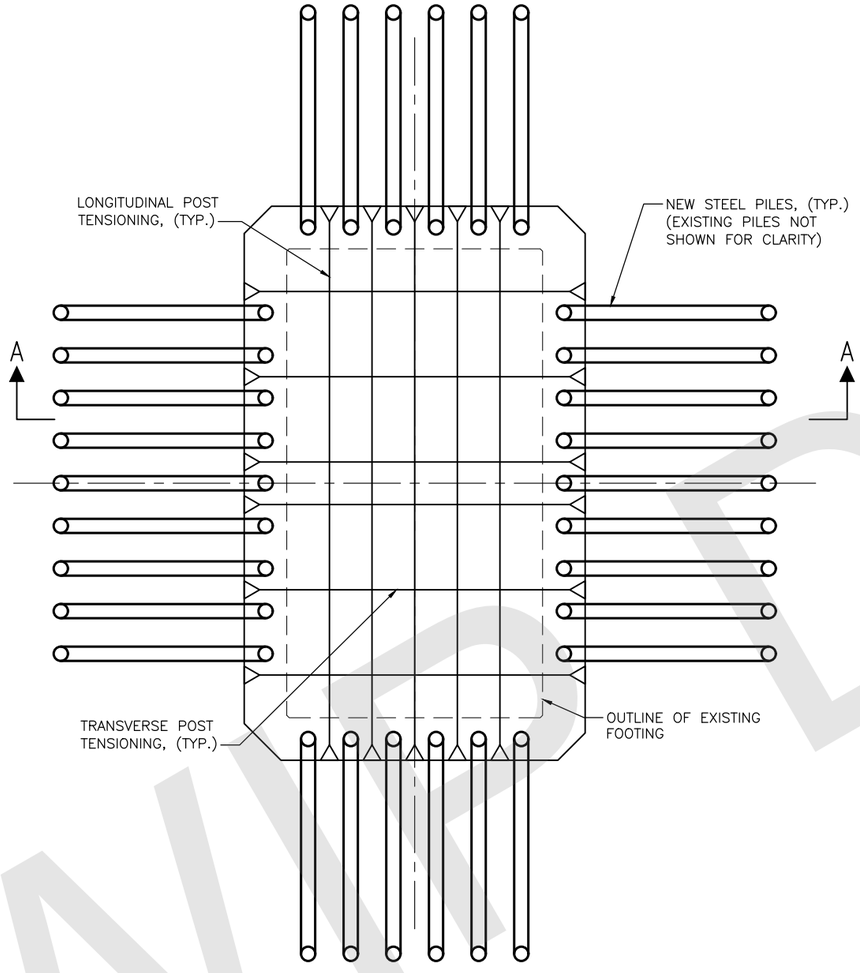
Seattle Department of Transportation

ORDINANCE NO. PW NO.
FUND:
SCALE:

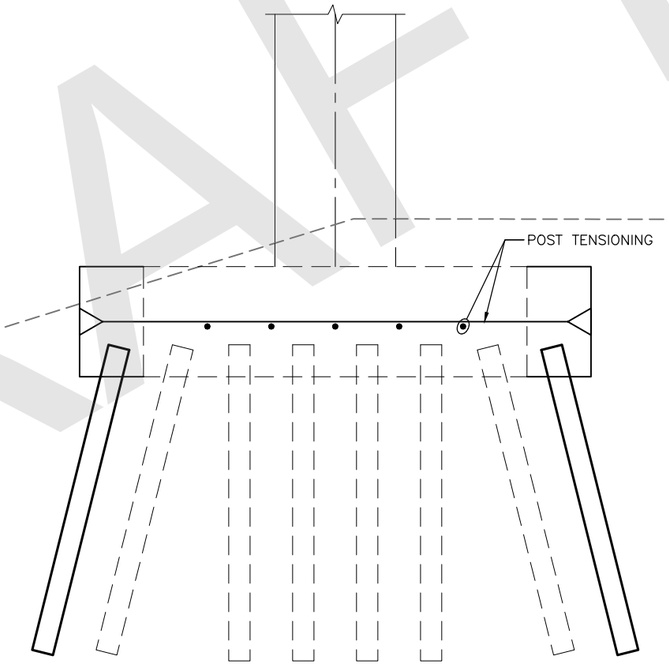
WEST SEATTLE HIGH RISE BRIDGE COST BENEFIT ANALYSIS

STANDARD DEMOLITION CONCEPT

JOB PC CO
VPI #
SHEET 5 OF 6



PLAN – RETROFIT EXISTING FOUNDATION
CONCEPT ONLY, NOT TO SCALE



SECTION A–A
CONCEPT ONLY, NOT TO SCALE

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Seattle Department of Transportation

ORDINANCE NO. PW NO.
FUND:
SCALE:

WEST SEATTLE HIGH RISE BRIDGE COST BENEFIT ANALYSIS

FOUNDATION RETROFIT

JOB PC CO
VPI #
SHEET 6 OF 6

APPENDIX B

PRELIMINARY GEOTECHNICAL INFORMATION

SUBMITTED TO:
WSP USA
33301 9th Ave S., Suite 300
Federal Way, WA 98003

BY:
Shannon & Wilson, Inc.
400 N. 34th Street, Suite 100
Seattle, WA 98103

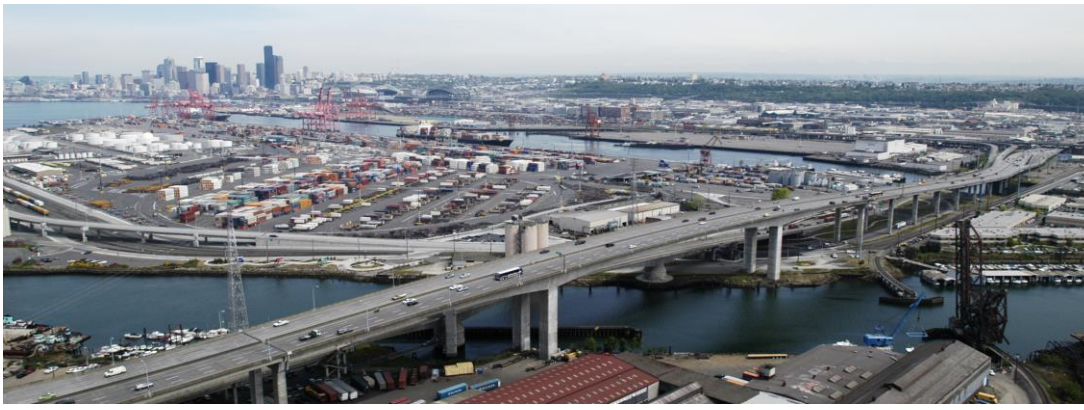
(206) 632-8020
www.shannonwilson.com

DRAFT

GEOTECHNICAL ENGINEERING REPORT

West Seattle High Level Bridge

SEATTLE, WASHINGTON



SHANNON & WILSON

October 5, 2020

Shannon & Wilson No: 105492-001

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Submitted To: WSP USA
33301 9th Ave S., Suite 300
Federal Way, WA 98003
Attn: Mr. Greg Banks

Subject: DRAFT GEOTECHNICAL ENGINEERING REPORT, WEST SEATTLE HIGH LEVEL BRIDGE, SEATTLE, WASHINGTON

Shannon & Wilson prepared this report and participated in this project as a subconsultant to WSP USA, Inc.. This report presents our Draft Geotechnical Engineering Report and was prepared by the undersigned. The site-specific site response analysis was recently completed and is included in Appendix A. The results of this analysis will be subsequently incorporated into the axial and lateral pile analysis and slope stability evaluations.

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

Sincerely,

SHANNON & WILSON, INC.

Robert Mitchell, PE
Vice President

SSS:AJB:KDK:AAS:RAM/ajb

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Appendices

Appendix A: Site-Specific Site Response Analysis
Important Information

ACRONYMS

AASHTO	American Association of State Highway Transportation Officials
BDM	Bridge Design Manual
BPF	blows per foot
FEE	Functional Evaluation Earthquake
FS	factor of safety
GDM	Geotechnical Design Manual
LRFD	Load and Resistance Factor Design
PGA	peak ground acceleration
SEE	Safety Evaluation Earthquake
SPT	Standard Penetration Test
WSDOT	Washington State Department of Transportation

ACRONYMS

1 INTRODUCTION

This draft report presents the results of our on-going geotechnical engineering studies for the existing West Seattle High Level Bridge project in Seattle, Washington. Our geotechnical and earthquake engineering studies for Piers 15 through 18 support the analysis of the existing condition of the West Seattle Bridge being performed by WSP USA, Inc.

Our scope of services includes:

- Developing axial and lateral resistances for the existing driven precast concrete and steel pipe piles that support Piers 15 to 18.
- Developing ground motions for the 210 and 975-year design ground motion levels. We developed code-based and site-specific response spectra for the two design levels.
- Assess geologic hazards including slope stability and liquefaction potential. The slope stability evaluations were performed for Piers 16 and 17 considering the static, seismic, and post-seismic cases. The liquefaction potential evaluations were performed using empirical methods and the existing borings that were drilled at Piers 15 through 18.

1.1 Use and Basis of Report

We understand this draft report will be used for on-going analysis of the West Seattle High Level Bridge project. This report may not be relied upon for any other purpose without Shannon & Wilson's authorization. Our studies were based on the following:

- The limitations of our scope, schedule, and budget described in Professional Services Subcontract between WSP and Shannon & Wilson.
- Our understanding of the project and information provided WSP.
- The results of existing explorations and testing performed along the project corridor, as summarized in this report.
- Our previous experience in the project area.

We have prepared the document "Important Information About Your Geotechnical/Environmental Report" (Appendix B) to assist you and others in understanding the use and limitations of this report.

This report includes subsurface information and geotechnical recommendations related to seismic considerations and foundations. This report will be updated as design proceeds.

We have performed our studies based on the information contained in this report. If the design team develops additional information concerning the project configuration, the

recommendations presented herein may need to be revised. Shannon & Wilson should be made aware of revised or additional information so that we can evaluate our recommendations for applicability.

2 PROJECT UNDERSTANDING

The West Seattle Bridge is a six-lane, 1-mile long bridge and roadway system consisting of elevated structures and interchanges located at the east and west ends of the project. The main span section of the structure extends across the West Waterway with a vertical clearance of 140 feet above high water.

On March 23, 2020, the West Seattle High Level Bridge was closed to vehicle traffic. This bridge is the City's top arterial by volume, typically carrying an average of over 100,000 vehicles every week. The Seattle Department of Transportation (SDOT) continues to inspect and monitor the structure.

Our geotechnical and earthquake engineering studies for Piers 15 through 18 support the analysis of the existing condition of the West Seattle Bridge being performed by WSP USA, Inc.

2.1 Site Description

The West Seattle Bridge project site is near the mouth of the Duwamish River where it empties into Puget Sound on the south shore of Elliot Bay. In this area the Duwamish River flows northward through a wide valley that lies between West Seattle and Beacon Hill. About a mile before the river reaches Elliot Bay, it divides into two channels, the West Waterway and the East Waterway, which flow on either side of Harbor Island. Both waterways are about 350 feet wide, and the West Waterway, which is used as a navigation channel for large vessels, is about 30 to 40 feet deep.

In vicinity of Harbor Island, the Duwamish Valley is about a mile wide, and the ground elevation is flat and generally less than 10 feet above mean high tide. When the West and East Waterways were dredged, the existing land surface of what is now Harbor Island was built by the placement of hydraulic fill. The southeast portion of the valley had been reclaimed earlier by similar placement of hydraulic fill. On the west flank of the valley, the ground rises abruptly and forms a ridge, known as Puget Ridge, running north and south about 200 feet above the valley floor. The northern end of Puget Ridge extends within about 3/4-mile of the south shore of Elliot Bay. To the west of Puget Ridge is a smaller valley beyond which rise the West Seattle hills.

Factories, industrial buildings, port-oriented facilities, and city streets have been constructed in the vicinity of the West Seattle Bridge.

3 REVIEW OF EXISTING DATA

We developed our understanding of the subsurface conditions at the site based on existing data generated by previous studies at the project location and structural components from as-built drawings. These reports include previous geotechnical investigations and studies for the bridge design which including geotechnical borings, geophyiscal downhole and crosshole testing, and construction monitoring.

The references used to develop our recommendations include:

- Geotechnical Engineering Studies, West Seattle Freeway Bridge Replacement (Shannon & Wilson, Inc., 1980)
- City of Seattle West Seattle Freeway Bridge Replacement Main Span Substructure as-built drawings (Parsons, Brickerhoff Tudor Engineering Co., 1980)
- City of Seattle West Seattle Freeway Bridge Replacement Main Span Superstructure as-built drawings (Parsons, Brickerhoff Tudor Engineering Co., 1980)

3.1 Existing Foundation System

Foundation support for the main span piers, Piers 16 and 17, consists of 36-inch-diameter, ¾-inch-wall, steel pipe piles driven open-ended into the underlying glacially overridden deposits. Both vertical and battered piles (batter not greater than 1 horizontal to 4 vertical) were incorporated for the design compressive loads for each pier foundation pile of 1,200 and 2,400 kips under static and seismic loadings, respectively.

Foundation support for Piers 15 and 18, on the west and east approach structures, consist of vertically driven, 24-inch octagonal prestressed concrete piles. These act as friction piles which derive their support primarily from the glacially-overridden clay and silt at Pier 15 and terminate in the alluvial deposits at Pier 18. Design compressive loads for these pile foundations were 400 and 800 kips under static and seismic loading, respectively.

Ground improvements were performed around Piers 16 and 17 during construction and extend to elevation -30 and -35 feet, respectively. For Pier 15 and 18, the ground improvements extend to elevation -24 feet. We understand that the vibro-compaction ground improvements were installed in an equilateral triangle pattern to achieve the required densification. The required densification criteria varied as a function of depth. For areas east of the west waterway the required standard penetration test blow count ranged

from 11 to 30 blows per foot between 5 and 40 feet below the ground surface. For areas west of the west waterway the required standard penetration test blow count ranged from 14 to 34 blows per foot between 5 and 40 feet below the ground surface.

3.2 Existing Geotechnical Data

Field explorations to determine subsurface conditions were performed during geotechnical engineering studies (Shannon & Wilson, Inc, 1980) and consisted of test borings and Dutch cone probes. The boring locations are shown in Figure 1 and the field explorations performed are summarized in Exhibit 3-1.

Exhibit 3-1: Existing Explorations

Bridge Section	Borings	Dutch Cone Probes	Pile Load Tests
Main Span Substructure	14	0	2
Harbor Island Structure	12	7	2

A laboratory testing program was also conducted towards determining the index and engineering properties of the foundation soils encountered along the bridge alignment. Index property tests included visual classification, natural water content, grain-size distribution, Atterberg limits, natural unit weight, pH, and sulfate content. Selected soil samples were also tested to evaluate the engineering properties of the soil including constant head permeability, consolidation, direct shear, triaxial compression, and dynamic tests consisting of resonant column and cyclic triaxial compression tests.

The east and west channel piers for the Main Span Substructure over the West Waterway were supported with 36-inch diameter by 3/4-inch wall steel pipe piles. Two load tests were made at these pier locations designated as Sites C (Pier 17) and D (Pier 16). It was considered not feasible to conduct a pile load test on a 36-inch diameter pipe pile. Therefore, a 24-inch diameter by 1 ½ inch thick wall steel pipe pile driven open-ended was tested.

The East Approach, Harbor Island and West Approach Structure piers are supported on 24-inch prestressed hollow concrete piles and pile load tests were made at three locations designated as Sites A, B and E. The load test at Site B was performed at Pier 24 and is the closest prestressed hollow concrete pile load test to Piers 15 and 18. The test pile at Site B consisted of a 24-inch prestressed hollow concrete pile.

4 SUMMARY OF SUBSURFACE CONDITIONS

The subsurface conditions encountered at the main span are generalized in the subsurface profile shown on Figure 2. Subsurface soils along the bridge alignment consist of loose to medium dense fill overlying alluvial and fluvial-marine deposits, which typically consist of loose to very dense sand with scattered layers of normally consolidated clay and silt. Glacially overridden deposits, consisting of interbedded hard clay and silt and very dense sand were encountered underlying the alluvial and fluvial-marine deposits. The depth to these deposits varied from 50 feet on the west to 200 feet on the east. The following is a general description of each unit:

1. Fill - includes both dumped and hydraulically placed materials. Hydraulically paced materials consist essentially of loose to medium dense, clean to slightly silty, fine to medium sand. Dumped materials consist essentially of sand, gravel, slag, and other debris.
2. Alluvium - consists of loose to very dense, clean to silty, fine to medium sand with scattered layers of soft to stiff sandy and/or clayey silt contains organic materials, wood, and some shell fragments.
3. Fluvial-Marine Deposits - consists of silty sand to sandy silt with scattered areas containing thick interbeds of silty clay with shell and wood fragments and organic debris.
4. Glacial Deposits:
    a. Reworked Sediments - consist of stiff to hard clay and clayey silt that contain clay fragments, wood debris, and occasional shell fragments.
    b. Lacustrine Sediments - consist of interbedded hard silty clay, clayey silt, and very dense fine sandy silt, and silty fine sand.
    c. Till - consists of very dense or hard unsorted mixture of gravel, sand, silt, and clay.

Fill was observed along the proposed alignment varying in thickness from several to about 15 feet. The Mean Lower Low Water (MLLW), indicated on the subsurface profile, indicates the approximate elevation of the ground surface prior to placement of hydraulic fill in the early 1900's.

4.1 Geotechnical Soil Units

Based on the existing subsurface information, the subsoils encountered along the Main Span Substructure (Piers 15 to 18) are described in the following:

- Alluvium - underlying the fill along the alignment is alluvial sand and silt. This material was encountered down to approximately -28, -40, -140, and -180 feet at Piers 15, 16, 17, and 18, respectively. Based on Standard Penetration Resistance (N-values)

obtained from the borings, most of the alluvial sand encountered west of the West Waterway is generally loose to medium dense (4 to 30 blows per foot, bpf). Below about 30 feet at Piers 17 and 18, the alluvial sand generally ranges from medium dense to very dense (20 to over 50 bpf). At Piers 17 and 18, silt layers are generally stiff to very stiff (10 to 20 bpf). Consolidation tests indicate these materials are normally consolidated. At Pier 16 a soft to stiff (3 to 11 bpf) clayey silt and sandy silt layer exists at about elevation -30 feet and ranges in thickness from about 10 to 20 feet. This layer slopes downward towards the West Waterway channel.

- Fluvial-Marine Deposits - the alluvium is underlain by Fluvial-Marine Deposits consisting primarily of silty sand and sandy silt at Piers 16 and 17 and part of Pier 15. The thickness of these deposits are on the order of 5, 35, and 28 feet at Piers 15, 16, and 17, respectively. The N-value of these deposits range from about 6 to 30 bpf at the West Waterway and from 11 to about 70 bpf east of the West Waterway.
- Glacial Deposits - Reworked Sediments - underlying the Fluvial-Marine Deposits are Reworked Glacial Deposits at Pier 16. These deposits are approximately 12 feet thick and consist primarily of stiff to very stiff (11 to 32 bpf) clay with scattered hard clay chunks.
- Glacial Deposits - Lacustrine Sediments - the Lacustrine Glacial Deposits generally consist of hard silty clay and clayey silt interbedded with very dense fine sandy silt and silty fine sand. They were observed at about elevation -28 feet to the bottom of the borings at Pier 15, with N-values ranging from 31 to 85 bpf above elevation -74 feet and over 100 bpf below this elevation. These deposits were observed at elevations varying from -77 to -92 feet at Pier 16 and from -153 to -184 feet at Pier 17. The N-values are normally over 100 bpf, except at boring WS-4 where N-values ranged from 31 to 82 bpf. At Pier 16, scattered cobbles up to 5 inches were encountered between about elevation -100 and -115 feet in an area adjacent to boring WS-6.
- Glacial Deposits - Till - underlying the Glacial Lacustrine Deposits is Till which consists of a very dense or hard unsorted mixture of gravel, sand, silt, and clay. The N-values of the Till are over 100 bpf. The Till was encountered at elevations varying from -156 to -184 feet at Pier 16, from -184 to -228 feet at Pier 17, and from -177 to -179 feet at Pier 18 down to the bottom of all borings drilled at these pier locations.

4.2 Groundwater

Based on observation wells installed in exiting borings, groundwater levels in the alluvial sand and silt were generally between elevations +5 and -5 feet. However, past observations have shown that within about 300 feet of the East and West Waterway channels, groundwater levels fluctuate with the tide. Based on past observations and records, groundwater is anticipated in the alluvial sand at the base of Puget Ridge will also be near the existing ground surface.

5 SEISMIC GROUND MOTIONS AND HAZARD EVALUATIONS

5.1 Design Ground Motions

The seismic design considers the two-level performance criteria for Essential bridges described in the WSDOT BDM. The two-level criteria includes the Safety Evaluation Earthquake (SEE) and Functional Evaluation Earthquake (FEE). The SEE corresponds to a 7% probability of exceedance in 75 years, or a 975-year return period. The FEE corresponds to a 30% probability of exceedance in 75 years, or a 210-year return period.

We performed both code-based and site-specific site response analysis for the project in accordance with AASHTO and WSDOT GDM requirements considering the two ground motion hazard levels. The ground surface site-specific design spectra were developed based on one-dimensional effective stress site response analyses. The shear wave velocity profiles included in the site-specific analysis were based on the downhole shear wave velocity tests conducted as part of the 1984 geophysical subsurface investigation program (Shannon & Wilson, 1984). Two crosshole and two downhole tests were identified at the east and west banks of the Duwamish west waterway, west of the existing West Seattle High Level Bridge. The crosshole shear wave velocity measurements included borings WSB-2 (near Pier 17) and WSB-4 (near Pier 16) and downhole shear wave velocity measurements included borings W-4 (near Pier 17) and W-7 (near Pier 16). Details of our site-specific site response analysis, assumptions, and subsurface data used to develop the design acceleration response spectra are provided in Appendix A of this report. Figure 3 presents the ground surface code-based and site-specific spectra for the project SEE and FEE hazard levels.

5.2 Liquefaction and Slope Stability Hazard Evaluation

Soil liquefaction is a phenomenon in which pore pressure in loose, saturated, granular soils increases during ground shaking to a level near the initial effective stress, thus resulting in a reduction of shear strength of the soil (a quicksand-like condition). This reduction in shear strength during liquefaction can cause a reduction in vertical and lateral foundation restraint and related ground settlement. Liquefied soils may undergo permanent lateral ground deformations and affect the overall stability of structures founded in or on them. Settlement resulting from liquefaction could produce permanent and differential displacements or downdrag on deep foundations.

The following sections describe our liquefaction evaluation for the project based on the empirical and site-specific site response analysis.

5.2.1 Seismically-Induced Liquefaction/Excess Pore Pressure Development

We used two empirical methods to evaluate liquefaction potential at this site based on the SPT data: Youd and others (2001) and Idriss and Boulanger (2008). We performed the liquefaction analyses for an earthquake of magnitude 7.6 and the site class modified design acceleration of 0.51 g. The characteristic magnitude was determined from the deaggregation analyses conducted by the USGS. The results of the empirical liquefaction analysis for Piers 15 through 18 are presented in Figure 4.

The nonlinear effective stress site-response analyses results performed for the project will be utilized to evaluate the liquefaction potential and excess pore pressure development for the site soil at Piers 15 through 18. Figure A-34 through A-37 the present the peak pore pressure ratio as a function of depth for the FEE and SEE ground motion levels at the Pier 15 and 16 and Pier 17 and 18 locations. The existing ground improvement was not considered in this round of analysis and will be included in a subsequent analysis.

The WSDOT GDM considers liquefaction to occur in site response analysis when the pore pressure ratio is equal to 1.0. In general, results of our site-specific site response analysis indicated limited instances of liquefaction with a pore pressure ratio equal to 1.0. However, as shown in Figures A-34 through A-37, the analysis indicates excess pore pressure development occurs with typical average excess pore pressure ratios ranging between 0.2 and 0.8. As noted above, the existing ground improvement was not considered in this round of analysis and will be included in a subsequent analysis.

5.2.2 Seismically-Induced Sloping Ground Lateral Pressures

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 downslope 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.

We utilized global stability analyses to evaluate slope instability at Piers 16 and 17. Our analyses considered static, seismic, and post-seismic cases. In accordance with the WSDOT GDM, the required factor-of-safety (FS) for global stability analyses are 1.5, 1.1, and 1.1 for static, seismic, and post-seismic cases.

The seismic condition evaluates the global stability factor of safety for pier foundations with an applied horizontal acceleration coefficient equal to one-half of the ground surface acceleration (As), defined for the SEE hazard level. Post-seismic analysis pertains to the embankment stability under liquefied conditions using residual soil shear strength properties and no horizontal seismic acceleration coefficient.

All stability analyses were performed using the limit equilibrium stability program GeoStudio 2018 SLOPE/W by Geo-Slope International (2018). The Morgenstern-Price method, which satisfies both force and moment equilibrium, was used to estimate FS values. We estimated soil properties for the stability analysis based on results of the subsurface explorations and laboratory testing and our experience in similar subsurface conditions. The liquefiable and reduced strength soil strength and extents were estimated based on the empirical liquefaction analysis described in above.

Seismic and post-seismic slope instability could result in the loss of foundation resistance and may impose additional lateral loads on foundations. The seismic and post-seismic global stability analyses presented in Figure 5 did not obtain the required factor of safety for Piers 16 and 17. For this case we estimated the foundation shear resistance required to reinforce the subject soils and obtain the required seismic or post-seismic factor of safety.

Figure 5 sheets 3, 5, and 7 provide the unfactored shear reinforcement requirements for Piers 16 and 17. The forces reflect a shear demand per foot of bridge pier parallel to the existing channel and should be multiplied by the foundation width to obtain the total unfactored shear force. The shear demand may be considered using two approaches:

- Apply the shear demand as a shear force at a depth equal to two-thirds of the depth of the shear plane.
- Apply the shear demand as a triangular distribution, or equivalent fluid weight (EFW), acting from the top of the pile cap to the depth of the shear plane. The EFW may be estimated as:

$$EFW = \frac{2 \cdot F}{d^2}$$

where F is the required shear force and d is the shear plane depth. The required shear force and depth of the shear plane is presented on Figure 5 sheets 3, 5, and 7.

6 EXISTING PILE FOUNDATION RESISTANCE

To estimate the behavior of West Seattle High Level Bridge soils, we reviewed the results of the exploration and laboratory testing programs in previous reports, in addition to the results of the pile load testing programs, which include static and dynamic pile testing analyses on test piles.

6.1 Axial Resistance

Figures 6 through 9 present the estimated axial resistance plots for existing foundations at Piers 15 through 18, respectively. The plots present nominal side and base resistance and nominal total compression resistance in accordance with AASHTO and GDM requirements. The unit nominal side and base resistance values used to develop these plots consider the static and dynamic test piles performed along the alignment on the respective precast concrete or steel piles. The appropriate AASHTO and GDM required resistance factors should be applied to the side and base resistance curves.

6.2 Lateral Resistance

Lateral loads acting on the structure may be resisted by lateral soil resistance provided by the driven piles. Table 1 presents recommended input parameters for lateral resistance analysis using the LPile program (Ensoft, 2018). The input parameters account for existing ground improvements around the existing piers. The liquefiable layers indicated in Table 1 were based on the empirical liquefaction analysis for Piers 15 through 18 presented in Figure 4. The liquefiable layers will be updated to consider the liquefiable layers determined from the site-specific analyses.

Group interaction effects reduces the lateral efficiency of closely spaced pile groups. Lateral efficiency may be incorporated in the analysis through p-multiplier (reduction) factors based on the pile group spacing. AASHTO Table 10.7.2.4-1 provides p-multipliers depending on the load direction and pile group configuration.

6.3 Force Deformation Response Curves

Our recommended axial soil input models are in the form of custom (or user-defined) side and base resistance load transfer curves, also referred to as t-z and q-z curves, respectively, which describe mobilized side ("t") and base ("q") resistance for a corresponding foundation displacement ("z"). For the bridge pier foundations, our recommended t-z and q-z curves were developed based on the "Driven Pile Sand (API)" and "Driven Pile Clay (API)" t-z and q-z curve models recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019). The nominal (or ultimate for Allowable Stress

Design) unit side and base resistances used to develop these axial soil models are based on the unit side and base resistance results obtained from the pile load tests completed along the alignment. We evaluated the data presented on the logs of borings accomplished near the test pile sites and developed correlations of average SPT N-value data to determine the estimated unit side and base resistance results. The data for the t-z and q-z curves at Piers 15 through 18 are presented in Tables 2 through 5, respectively. The t-z and q-z curves at Piers 15 through 18 are presented graphically in Figures 10 through 13, respectively.

7 REFERENCES

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TABLE 1
WEST SEATTLE BRIDGE
RECOMMENDED SOIL PARAMETERS FOR LPILE ANALYSES

Structure	Abutment	Boring		Soil Type	Depth	Elevation	Groundwater Elevation	LPILE p-y Curve Model	Liquefiable?	Effective Unit Weight, γ'	Friction Angle, ϕ		Modulus of Subgrade Reaction, k		Undrained Shear Strength	Strain Factor e_{50}
											Static	Liquefied	Static	Liquefied		
									(-)	(pcf)	(degrees)		(pci)		(psf)	(-)
West Seattle Bridge	Pier 15	WSB-2	Fill	Slightly Silty, Gravelly Sand	0 to 4 ft	4 to 0 ft	-5 ft	Sand	N	120	31	31	75	75	-	-
			Alluvium	Slightly Silty Sand	4 to 9 ft	0 to -5 ft		Sand	N	120	32	32	90	90	-	-
				Slightly Silty Sand	9 to 10 ft	-5 to -6 ft		Sand	N	58	32	32	60	60	-	-
				Sandy Gravel	10 to 21 ft	-6 to -17 ft		Sand	Y	68	38	30	65	50	-	-
				Slightly Silty Sand	21 to 32 ft	-17 to -28 ft		Sand	Y	58	33	12	65	25	-	-
			Glacial Deposits - Lacustrine	Silty Clay with Sand	32 to 55 ft	-28 to -51 ft		Stiff Clay w/out Free Water	N	53	-	-	-	-	4,000	0.004
				Clayey Silt and Silty Clay	55 to 75 ft	-51 to -71 ft		Stiff Clay w/out Free Water	N	53	-	-	-	-	6,000	0.004
				Sandy, Clayey Silt and Sandy, Silty Clay	75 to 124 ft	-71 to -120 ft		Stiff Clay w/out Free Water	N	53	-	-	-	-	8,000	0.004
				Silty Sand and Sandy Silt	124 to 170 ft	-120 to -166 ft		Sand	N	63	39	39	125	125	-	-
	Pier 16	WSB-6	Fill	Slightly Silty, Sandy Gravel	0 to 8 ft	4 to -4 ft	-4 ft	Sand	N	125	32	32	25	25	-	-
			Alluvium	Slightly Silty Sand	8 to 18.5 ft	-4 to -14.5 ft		Sand	Y	48	29	8	15	5	-	-
				Slightly Silty Sand with scattered Silt layers	18.5 to 32 ft	-14.5 to -28 ft		Sand	Y	58	33	20	60	40	-	-
				Sandy, Clayey Silt	32 to 43 ft	-28 to -39 ft		Sand	N	48	26	26	20	20	-	-
			Fluvial-Marine Deposits	Silty Sand	43 to 64 ft	-39 to -60 ft		Sand	Y	58	31	10	45	20	-	-
				Slightly Clayey, Sandy Silt	64 to 79 ft	-60 to -75 ft		Sand	Y	58	29	10	55	25	-	-
			Glacial Deposits - Lacustrine	Slightly Clayey, Sandy Silt with Clay Fragments	79 to 89 ft	-75 to -85 ft		Stiff Clay w/out Free Water	N	58	-	-	-	-	4,000	0.004
				Sandy, Clayey Silt and Silty Clay	89 to 123 ft	-85 to -119 ft		Stiff Clay w/out Free Water	N	58	-	-	-	-	8,000	0.004
				Silty Sand	123 to 125 ft	-229 to -121 ft		Sand	N	63	39	39	125	125	-	-
	Pier 17	WSB-11	Alluvium	Sand with Gravel	0 to 7 ft	2 to -5 ft	-5 ft	Sand	Y	115	32	8	70	20	-	-
				Sand with Gravel	7 to 27 ft	-5 to -25 ft		Sand	Y	53	32	8	50	15		
				Silty Sand with Clayey Silt and slightly Silty Sand layers	27 to 55 ft	-25 to -53 ft		Sand	N	63	36	36	85	85	-	-
				Silty Sand	55 to 83 ft	-53 to -81 ft		Sand	N	63	38	38	115	115	-	-
				Clayey, Sandy Silt	83 to 89 ft	-81 to -87 ft		Sand	N	53	27	27	35	35	-	-
				Silty Sand	89 to 103 ft	-87 to -101 ft		Sand	N	63	32	32	95	95	-	-
				Interbedded Silty Clay and Sandy Silt	103 to 128 ft	-101 to -126 ft		Sand	N	53	34	34	110	110	-	-
				Slightly Silty to Silty Sand	128 to 138 ft	-126 to -136 ft		Sand	N	63	39	39	125	125	-	-
				Clayey, Sandy Silt with Silty Sand layers	138 to 143 ft	-136 to -141 ft		Sand	N	43	24	24	5	5	-	-
			Fluvial-Marine Deposits	Slightly Clayey Silt with Sand	143 to 149 ft	-141 to -147 ft		Sand	N	48	39	39	125	125	-	-
				Silty Sand	149 to 178 ft	-147 to -176 ft		Sand	N	63	39	39	125	125	-	-
			Glacial Deposits - Lacustrine	Silty Clay with scattered Sand and Gravel	178 to 188 ft	-176 to -186 ft		Stiff Clay w/out Free Water	N	53	-	-	-	-	8,000	0.004
			Glacial Deposits - Till	Gravelly, Sandy, Silty Clay	188 to 215 ft	-186 to -213 ft		Stiff Clay w/out Free Water	N	58	-	-	-	-	12,000	0.004
	Pier 18	WSB-15	Fill	Sandy, Silty Clay	0 to 7 ft	8 to 1 ft	-4 ft	Soft Clay	N	90	-	-	-	-	400	0.02
			Alluvium	Slightly Silty Sand	7 to 12 ft	1 to -4 ft		Sand	Y	120	32	8	85	25	-	-
				Slightly Silty Sand	12 to 32 ft	-4 to -24 ft		Sand	Y	58	32	8	55	15	-	-
				Slightly Silty Sand	32 to 63 ft	-24 to -55 ft		Sand	Y	58	35	22	80	50	-	-
				Interbedded Silty Sand and slightly Clayey, Sandy Silt	63 to 80 ft	-55 to -72 ft		Sand	Y	58	33	5	65	10	-	-
				Interbedded Silty Sand and slightly Clayey, Sandy Silt	80 to 128 ft	-72 to -120 ft		Sand	N	58	33	33	65	65	-	-
				Slightly Clayey to Clayey, Sandy Silt	128 to 144 ft	-120 to -136 ft		Sand	N	53	29	29	65	65	-	-
				Silty Sand with Clayey, Sandy, Silt layers	144 to 183 ft	-136 to -175 ft		Sand	N	63	39	39	120	120	-	-
				Sandy, Clayey Silt	183 to 189 ft	-175 to -181 ft		Sand	N	53	28	28	45	45	-	-
			Glacial Deposits - Till	Gravelly, Clayey, Silty Sand	189 to 193 ft	-181 to -185 ft		Sand	N	68	39	39	125	125	-	-
				Sandy, Silty Clay with Gravel	193 to 230.5 ft	-185 to -222.5 ft		Stiff Clay w/out Free Water	N	63	-	-	-	-	12,000	0.004

Notes:
1. Parameters shown above are generalized based on the soil conditions encountered in the borings drilled for this project.
2. ft = feet; bpf = blows per foot; pcf = pounds per cubic foot; pci = pounds per cubic inch; psf = pounds per square foot

TABLE 2
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 15 AND 18

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
15	24	-11	-66	-66	0	-11	0	0	0.0
							0.1	0.68	4,504.3
							1.2	0.68	4,504.3
					2	-13	0	0	0.0
							0.1	0.74	4,901.8
							1.2	0.74	4,901.8
					4	-15	0	0	0.0
							0.1	0.81	5,365.4
							1.2	0.81	5,365.4
					6 End Layer 1	-17	0	0	0.0
							0.1	0.87	5,762.9
							1.2	0.87	5,762.9
					6 Begin Layer 2	-17	0	0	0.0
							0.1	0.77	5,100.5
							1.2	0.77	5,100.5
					8	-19	0	0	0.0
							0.1	0.82	5,431.7
							1.2	0.82	5,431.7
					10	-21	0	0	0.0
							0.1	0.87	5,762.9
							1.2	0.87	5,762.9
					12	-23	0	0	0.0
							0.1	0.92	6,094.1
							1.2	0.92	6,094.1
					14	-25	0	0	0.0
							0.1	0.89	5,895.4
							1.2	0.89	5,895.4
					16	-27	0	0	0.0
							0.1	0.94	6,226.6
							1.2	0.94	6,226.6
					18 to 40	-29 to -51	0	0	0.0
							0.1	2	13,248.0
							1.2	2	13,248.0
					40 to 56	-51 to -67	0	0	0.0
							0.1	3	19,872.0
							1.2	3	19,872.0

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Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
18	24	-5	-147	-147	0	-5	0	0	0.0
							0.1	0.52	3,444.5
							1.2	0.52	3,444.5
					2	-7	0	0	0.0
							0.1	0.57	3,775.7
							1.2	0.57	3,775.7
					4	-9	0	0	0.0
							0.1	0.61	4,040.6
							1.2	0.61	4,040.6
					6	-11	0	0	0.0
							0.1	0.66	4,371.8
							1.2	0.66	4,371.8
					8	-13	0	0	0.0
							0.1	0.71	4,703.0
							1.2	0.71	4,703.0
					10	-15	0	0	0.0
							0.1	0.75	4,968.0
							1.2	0.75	4,968.0
					12	-17	0	0	0.0
							0.1	0.8	5,299.2
							1.2	0.8	5,299.2
					14	-19	0	0	0.0
							0.1	0.84	5,564.2
							1.2	0.84	5,564.2
					16	-21	0	0	0.0
							0.1	0.89	5,895.4
							1.2	0.89	5,895.4
					18	-23	0	0	0.0
							0.1	0.94	6,226.6
							1.2	0.94	6,226.6
					20	-25	0	0	0.0
							0.1	1.02	6,756.5
							1.2	1.02	6,756.5
					22	-27	0	0	0.0
							0.1	1.07	7,087.7
							1.2	1.07	7,087.7
					24	-29	0	0	0.0
							0.1	1.12	7,418.9
							1.2	1.12	7,418.9
					26	-31	0	0	0.0
							0.1	1.17	7,750.1
							1.2	1.17	7,750.1

TABLE 2
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 15 AND 18

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					28	-33	0	0	0.0
							0.1	1.22	8,081.3
							1.2	1.22	8,081.3
					30	-35	0	0	0.0
							0.1	1.27	8,412.5
							1.2	1.27	8,412.5
					32	-37	0	0	0.0
							0.1	1.32	8,743.7
							1.2	1.32	8,743.7
					34	-39	0	0	0.0
							0.1	1.37	9,074.9
							1.2	1.37	9,074.9
					36	-41	0	0	0.0
							0.1	1.42	9,406.1
							1.2	1.42	9,406.1
					38	-43	0	0	0.0
							0.1	1.47	9,737.3
							1.2	1.47	9,737.3
					40	-45	0	0	0.0
							0.1	1.51	10,002.2
							1.2	1.51	10,002.2
					42	-47	0	0	0.0
							0.1	1.56	10,333.4
							1.2	1.56	10,333.4
					44	-49	0	0	0.0
							0.1	1.61	10,664.6
							1.2	1.61	10,664.6
					46	-51	0	0	0.0
							0.1	1.66	10,995.8
							1.2	1.66	10,995.8
					48	-53	0	0	0.0
							0.1	1.71	11,327.0
							1.2	1.71	11,327.0
					50 End Layer 2	-55	0	0	0.0
							0.1	1.76	11,658.2
							1.2	1.76	11,658.2
					50 Begin Layer 3	-55	0	0	0.0
							0.1	1.38	9,141.1
							1.2	1.38	9,141.1
					52	-57	0	0	0.0
							0.1	1.42	9,406.1
							1.2	1.42	9,406.1

TABLE 2
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Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					54	-59	0	0	0.0
							0.1	1.46	9,671.0
							1.2	1.46	9,671.0
					56	-61	0	0	0.0
							0.1	1.5	9,936.0
							1.2	1.5	9,936.0
					58	-63	0	0	0.0
							0.1	1.54	10,201.0
							1.2	1.54	10,201.0
					60	-65	0	0	0.0
							0.1	1.57	10,399.7
							1.2	1.57	10,399.7
					62	-67	0	0	0.0
							0.1	1.61	10,664.6
							1.2	1.61	10,664.6
					64	-69	0	0	0.0
							0.1	1.65	10,929.6
							1.2	1.65	10,929.6
					66	-71	0	0	0.0
							0.1	1.69	11,194.6
							1.2	1.69	11,194.6
					68	-73	0	0	0.0
							0.1	1.93	12,784.3
							1.2	1.93	12,784.3
					70	-75	0	0	0.0
							0.1	1.97	13,049.3
							1.2	1.97	13,049.3
					72	-77	0	0	0.0
							0.1	2.02	13,380.5
							1.2	2.02	13,380.5
					74	-79	0	0	0.0
							0.1	2.06	13,645.4
							1.2	2.06	13,645.4
					76	-81	0	0	0.0
							0.1	2.1	13,910.4
							1.2	2.1	13,910.4
					78	-83	0	0	0.0
							0.1	2.15	14,241.6
							1.2	2.15	14,241.6
					80	-85	0	0	0.0
							0.1	2.19	14,506.6
							1.2	2.19	14,506.6

TABLE 2
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 15 AND 18

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					82	-87	0	0	0.0
							0.1	2.23	14,771.5
							1.2	2.23	14,771.5
					84	-89	0	0	0.0
							0.1	2.27	15,036.5
							1.2	2.27	15,036.5
					86	-91	0	0	0.0
							0.1	2.32	15,367.7
							1.2	2.32	15,367.7
					88	-93	0	0	0.0
							0.1	2.36	15,632.6
							1.2	2.36	15,632.6
					90	-95	0	0	0.0
							0.1	2.4	15,897.6
							1.2	2.4	15,897.6
					92	-97	0	0	0.0
							0.1	2.45	16,228.8
							1.2	2.45	16,228.8
					94	-99	0	0	0.0
							0.1	2.49	16,493.8
							1.2	2.49	16,493.8
					96	-101	0	0	0.0
							0.1	2.53	16,758.7
							1.2	2.53	16,758.7
					98	-103	0	0	0.0
							0.1	2.58	17,089.9
							1.2	2.58	17,089.9
					100	-105	0	0	0.0
							0.1	2.62	17,354.9
							1.2	2.62	17,354.9
					102	-107	0	0	0.0
							0.1	2.66	17,619.8
							1.2	2.66	17,619.8
					104	-109	0	0	0.0
							0.1	2.7	17,884.8
							1.2	2.7	17,884.8
					106	-111	0	0	0.0
							0.1	2.75	18,216.0
							1.2	2.75	18,216.0
					108	-113	0	0	0.0
							0.1	2.79	18,481.0
							1.2	2.79	18,481.0

TABLE 2
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 15 AND 18

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					110	-115	0	0	0.0
							0.1	2.83	18,745.9
							1.2	2.83	18,745.9
					112	-117	0	0	0.0
							0.1	2.88	19,077.1
							1.2	2.88	19,077.1
					114	-119	0	0	0.0
							0.1	2.92	19,342.1
							1.2	2.92	19,342.1
					116	-121	0	0	0.0
							0.1	2.73	18,083.5
							1.2	2.73	18,083.5
					118	-123	0	0	0.0
							0.1	2.76	18,282.2
							1.2	2.76	18,282.2
					120	-125	0	0	0.0
							0.1	2.8	18,547.2
							1.2	2.8	18,547.2
					122	-127	0	0	0.0
							0.1	2.83	18,745.9
							1.2	2.83	18,745.9
					124	-129	0	0	0.0
							0.1	2.87	19,010.9
							1.2	2.87	19,010.9
					126	-131	0	0	0.0
							0.1	2.9	19,209.6
							1.2	2.9	19,209.6
					128	-133	0	0	0.0
							0.1	2.94	19,474.6
							1.2	2.94	19,474.6
					130	-135	0	0	0.0
							0.1	2.97	19,673.3
							1.2	2.97	19,673.3
					132	-137	0	0	0.0
							0.1	3.73	24,707.5
							1.2	3.73	24,707.5
					134	-139	0	0	0.0
							0.1	3.78	25,038.7
							1.2	3.78	25,038.7
					136	-141	0	0	0.0
							0.1	3.83	25,369.9
							1.2	3.83	25,369.9

TABLE 2
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 15 AND 18

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					138	-143	0	0	0.0
							0.1	3.88	25,701.1
							1.2	3.88	25,701.1
					140	-145	0	0	0.0
							0.1	3.94	26,098.6
							1.2	3.94	26,098.6
					142	-147	0	0	0.0
							0.1	3.99	26,429.8
							1.2	3.99	26,429.8

NOTES:

[1] The reference bottom of footing and pile tip elevations are based on information included as part of scanned historical reports.

[2] Our recommended T-z curves are developed based on the Driven Pile Sand T-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019). The nominal (i.e., ultimate) unit side resistances used to develop the T-z curves are based on the results of test piles B-LTP, C-LTP, and D-LTP.

[3] lbf = pound-force and ksf = kips per square foot

TABLE 3
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 16 AND 17

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
16	36	-23.5	-140	-150	0	-23.5	0	0	0.0
							0.1	0.49	4,618.1
							1.8	0.49	4,618.1
					2	-25.5	0	0	0.0
							0.1	0.52	4,900.9
							1.8	0.52	4,900.9
					4	-27.5	0	0	0.0
							0.1	0.54	5,089.4
							1.8	0.54	5,089.4
					6	-29.5	0	0	0.0
							0.1	0.5	4,712.4
							1.8	0.5	4,712.4
					8	-31.5	0	0	0.0
							0.1	0.39	3,675.7
							1.8	0.39	3,675.7
					10	-33.5	0	0	0.0
							0.1	0.41	3,864.2
							1.8	0.41	3,864.2
					12	-35.5	0	0	0.0
							0.1	0.42	3,958.4
							1.8	0.42	3,958.4
					14	-37.5	0	0	0.0
							0.1	0.44	4,146.9
							1.8	0.44	4,146.9
					16	-39.5	0	0	0.0
							0.1	0.6	5,654.9
							1.8	0.6	5,654.9
					18	-41.5	0	0	0.0
							0.1	0.62	5,843.4
							1.8	0.62	5,843.4
					20	-43.5	0	0	0.0
							0.1	0.65	6,126.1
							1.8	0.65	6,126.1
					22	-45.5	0	0	0.0
							0.1	0.67	6,314.6
							1.8	0.67	6,314.6
					24	-47.5	0	0	0.0
							0.1	0.7	6,597.3
							1.8	0.7	6,597.3
					26	-49.5	0	0	0.0
							0.1	0.72	6,785.8
							1.8	0.72	6,785.8

TABLE 3
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 16 AND 17

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					28	-51.5	0	0	0.0
							0.1	0.74	6,974.3
							1.8	0.74	6,974.3
					30	-53.5	0	0	0.0
							0.1	0.77	7,257.1
							1.8	0.77	7,257.1
					32	-55.5	0	0	0.0
							0.1	0.79	7,445.6
							1.8	0.79	7,445.6
					34	-57.5	0	0	0.0
							0.1	0.82	7,728.3
							1.8	0.82	7,728.3
					36	-59.5	0	0	0.0
							0.1	0.84	7,916.8
							1.8	0.84	7,916.8
					38	-61.5	0	0	0.0
							0.1	0.79	7,445.6
							1.8	0.79	7,445.6
					40	-63.5	0	0	0.0
							0.1	0.81	7,634.1
							1.8	0.81	7,634.1
					42	-65.5	0	0	0.0
							0.1	0.83	7,822.6
							1.8	0.83	7,822.6
					44	-67.5	0	0	0.0
							0.1	0.85	8,011.1
							1.8	0.85	8,011.1
					46	-69.5	0	0	0.0
							0.1	0.87	8,199.6
							1.8	0.87	8,199.6
					48	-71.5	0	0	0.0
							0.1	0.89	8,388.1
							1.8	0.89	8,388.1
					50	-73.5	0	0	0.0
							0.1	0.91	8,576.5
							1.8	0.91	8,576.5
					52 to 60	-75.5 to -83.5	0	0	0.0
							0.1	2.14	20,169.0
							1.8	2.14	20,169.0
					62 to 96	-85.5 to -119.5	0	0	0.0
							0.1	4.5	42,411.5
							1.8	4.5	42,411.5

TABLE 3
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 16 AND 17

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					98 to 128	-121.5 to -151.5	0	0	0.0
							0.1	4.5	42,411.5
							1.8	4.5	42,411.5
17	36	-23.5	-185	-220	0	-23.5	0	0	0.0
							0.1	0.3	2,827.4
							1.8	0.3	2,827.4
					2	-25.5	0	0	0.0
							0.1	0.35	3,298.7
							1.8	0.35	3,298.7
					4	-27.5	0	0	0.0
							0.1	0.37	3,487.2
							1.8	0.37	3,487.2
					6	-29.5	0	0	0.0
							0.1	0.39	3,675.7
							1.8	0.39	3,675.7
					8	-31.5	0	0	0.0
							0.1	0.41	3,864.2
							1.8	0.41	3,864.2
					10	-33.5	0	0	0.0
							0.1	0.43	4,052.7
							1.8	0.43	4,052.7
					12	-35.5	0	0	0.0
							0.1	0.45	4,241.2
							1.8	0.45	4,241.2
					14	-37.5	0	0	0.0
							0.1	0.48	4,523.9
							1.8	0.48	4,523.9
					16	-39.5	0	0	0.0
							0.1	0.5	4,712.4
							1.8	0.5	4,712.4
					18	-41.5	0	0	0.0
							0.1	0.52	4,900.9
							1.8	0.52	4,900.9
					20	-43.5	0	0	0.0
							0.1	0.54	5,089.4
							1.8	0.54	5,089.4
					22	-45.5	0	0	0.0
							0.1	0.57	5,372.1
							1.8	0.57	5,372.1
					24	-47.5	0	0	0.0
							0.1	0.59	5,560.6
							1.8	0.59	5,560.6

TABLE 3
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 16 AND 17

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					26	-49.5	0	0	0.0
							0.1	0.61	5,749.1
							1.8	0.61	5,749.1
					28	-51.5	0	0	0.0
							0.1	0.63	5,937.6
							1.8	0.63	5,937.6
					30	-53.5	0	0	0.0
							0.1	0.74	6,974.3
							1.8	0.74	6,974.3
					32	-55.5	0	0	0.0
							0.1	0.76	7,162.8
							1.8	0.76	7,162.8
					34	-57.5	0	0	0.0
							0.1	0.79	7,445.6
							1.8	0.79	7,445.6
					36	-59.5	0	0	0.0
							0.1	0.81	7,634.1
							1.8	0.81	7,634.1
					38	-61.5	0	0	0.0
							0.1	0.84	7,916.8
							1.8	0.84	7,916.8
					40	-63.5	0	0	0.0
							0.1	0.86	8,105.3
							1.8	0.86	8,105.3
					42	-65.5	0	0	0.0
							0.1	0.89	8,388.1
							1.8	0.89	8,388.1
					44	-67.5	0	0	0.0
							0.1	0.91	8,576.5
							1.8	0.91	8,576.5
					46	-69.5	0	0	0.0
							0.1	0.94	8,859.3
							1.8	0.94	8,859.3
					48	-71.5	0	0	0.0
							0.1	0.96	9,047.8
							1.8	0.96	9,047.8
					50	-73.5	0	0	0.0
							0.1	0.99	9,330.5
							1.8	0.99	9,330.5
					52	-75.5	0	0	0.0
							0.1	1.01	9,519.0
							1.8	1.01	9,519.0

TABLE 3
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 16 AND 17

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					54	-77.5	0	0	0.0
							0.1	1.04	9,801.8
							1.8	1.04	9,801.8
					56	-79.5	0	0	0.0
							0.1	1.06	9,990.3
							1.8	1.06	9,990.3
					58	-81.5	0	0	0.0
							0.1	0.7	6,597.3
							1.8	0.7	6,597.3
					60	-83.5	0	0	0.0
							0.1	0.72	6,785.8
							1.8	0.72	6,785.8
					62	-85.5	0	0	0.0
							0.1	0.73	6,880.1
							1.8	0.73	6,880.1
					64	-87.5	0	0	0.0
							0.1	1.1	10,367.3
							1.8	1.1	10,367.3
					66	-89.5	0	0	0.0
							0.1	1.12	10,555.8
							1.8	1.12	10,555.8
					68	-91.5	0	0	0.0
							0.1	1.14	10,744.2
							1.8	1.14	10,744.2
					70	-93.5	0	0	0.0
							0.1	1.16	10,932.7
							1.8	1.16	10,932.7
					72	-95.5	0	0	0.0
							0.1	1.19	11,215.5
							1.8	1.19	11,215.5
					74	-97.5	0	0	0.0
							0.1	1.21	11,404.0
							1.8	1.21	11,404.0
					76	-99.5	0	0	0.0
							0.1	1.23	11,592.5
							1.8	1.23	11,592.5
					78	-101.5	0	0	0.0
							0.1	1.05	9,896.0
							1.8	1.05	9,896.0
					80	-103.5	0	0	0.0
							0.1	1.07	10,084.5
							1.8	1.07	10,084.5

TABLE 3
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 16 AND 17

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					82	-105.5	0	0	0.0
							0.1	1.09	10,273.0
							1.8	1.09	10,273.0
					84	-107.5	0	0	0.0
							0.1	1.11	10,461.5
							1.8	1.11	10,461.5
					86	-109.5	0	0	0.0
							0.1	1.12	10,555.8
							1.8	1.12	10,555.8
					88	-111.5	0	0	0.0
							0.1	1.14	10,744.2
							1.8	1.14	10,744.2
					90	-113.5	0	0	0.0
							0.1	1.16	10,932.7
							1.8	1.16	10,932.7
					92	-115.5	0	0	0.0
							0.1	1.18	11,121.2
							1.8	1.18	11,121.2
					94	-117.5	0	0	0.0
							0.1	1.2	11,309.7
							1.8	1.2	11,309.7
					96	-119.5	0	0	0.0
							0.1	1.21	11,404.0
							1.8	1.21	11,404.0
					98	-121.5	0	0	0.0
							0.1	1.23	11,592.5
							1.8	1.23	11,592.5
					100	-123.5	0	0	0.0
							0.1	1.25	11,781.0
							1.8	1.25	11,781.0
					102	-125.5	0	0	0.0
							0.1	1.27	11,969.5
							1.8	1.27	11,969.5
					104	-127.5	0	0	0.0
							0.1	1.6	15,079.6
							1.8	1.6	15,079.6
					106	-129.5	0	0	0.0
							0.1	1.63	15,362.4
							1.8	1.63	15,362.4
					108	-131.5	0	0	0.0
							0.1	1.65	15,550.9
							1.8	1.65	15,550.9

TABLE 3
SUMMARY OF RECOMMENDED T-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 16 AND 17

Summary of Foundation Geometry [1]					Recommended T-z Curves Beneath Reference Bottom of Footing Elevation [2]				
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Depth	Elevation	Pile Displacement, z	Mobilized Unit Side Resistance, T	
	(inches)	(feet)	(feet)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
					110	-133.5	0	0	0.0
							0.1	1.68	15,833.6
							1.8	1.68	15,833.6
					112	-135.5	0	0	0.0
							0.1	1.7	16,022.1
							1.8	1.7	16,022.1
					114	-137.5	0	0	0.0
							0.1	0.95	8,953.5
							1.8	0.95	8,953.5
					116	-139.5	0	0	0.0
							0.1	0.95	8,953.5
							1.8	0.95	8,953.5
					118 to 152	-141.5 to -175.5	0	0	0.0
							0.1	3.5	32,986.7
							1.8	3.5	32,986.7
					154 to 162	-177.5 to -185.5	0	0	0.0
							0.1	4	37,699.1
							1.8	4	37,699.1
					164 to 198	-187.5 to -221.5	0	0	0.0
							0.1	4	37,699.1
							1.8	4	37,699.1

NOTES:

[1] The reference bottom of footing and pile tip elevations are based on information included as part of scanned historical reports.

[2] Our recommended T-z curves are developed based on the Driven Pile Sand T-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019). The nominal (i.e., ultimate) unit side resistances used to develop the T-z curves are based on the results of test piles C-LTP and D-LTP.

[3] lbf = pound-force and ksf = kips per square foot

TABLE 4
SUMMARY OF RECOMMENDED Q-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 15 AND 18

Summary of Foundation Geometry [1]					Recommended Q-z Curves [2]		
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Pile Displacement, z	Mobilized Unit Base Resistance, Q	
	(inches)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
15	24	-11	-66	-66	0	0	0.0
					0.048	15	49,654.4
					0.312	30	99,308.8
					1.008	45	148,963.2
					1.752	54	178,755.8
					2.4	60	198,617.5
					3	60	198,617.5
18	24	-5	-147	-147	0	0	0.0
					0.048	12.5	41,378.7
					0.312	25	82,757.3
					1.008	37.5	124,136.0
					1.752	45	148,963.2
					2.4	50	165,514.6
					3	50	165,514.6

NOTES:

[1] The reference bottom of footing and pile tip elevations are based on information included as part of scanned historical reports.

[2] Our recommended Q-z curves are developed based on the Driven Pile Sand and Clay Q-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019). The nominal (i.e., ultimate) unit base resistances used to develop the Q-z curves are based on the results of test piles B-LTP, C-LTP, and D-LTP.

[3] lbf = pound-force and ksf = kips per square foot

TABLE 5
SUMMARY OF RECOMMENDED Q-Z CURVES FOR DRIVEN PILE DESIGN - PIERS 16 AND 17

Summary of Foundation Geometry [1]					Recommended Q-z Curves [2]		
Pier No.	Pile Diameter / Width	Reference Bottom of Footing Elevation	Maximum Pile Tip Elevation	Minimum Pile Tip Elevation	Pile Displacement, z	Mobilized Unit Base Resistance, Q	
	(inches)	(feet)	(feet)	(feet)	(inches)	(ksf)	(lbf)
16	36	-23.5	-140	-150	0	0	0.0
					0.072	20	141,371.7
					0.468	40	282,743.3
					1.512	60	424,115.0
					2.628	72	508,938.0
					3.6	80	565,486.7
					4.5	80	565,486.7
17	36	-23.5	-185	-220	0	0	0.0
					0.072	20	141,371.7
					0.468	40	282,743.3
					1.512	60	424,115.0
					2.628	72	508,938.0
					3.6	80	565,486.7
					4.5	80	565,486.7

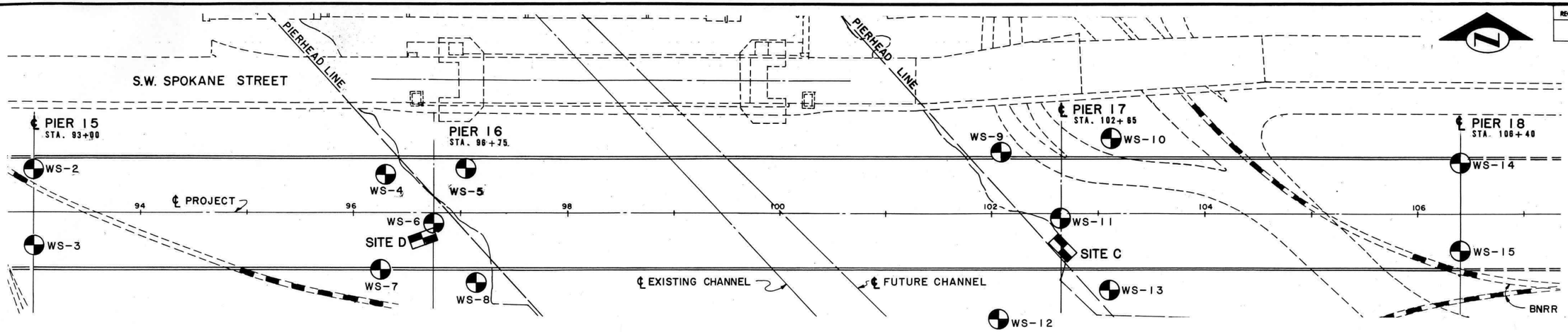
NOTES:

[1] The reference bottom of footing and pile tip elevations are based on information included as part of scanned historical reports.

[2] Our recommended Q-z curves are developed based on the Driven Pile Sand and Clay Q-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019). The nominal (i.e., ultimate) unit base resistances used to develop the Q-z curves are based on the results of test piles C-LTP and D-LTP.

[3] lbf = pound-force and ksf = kips per square foot

REGION NO.	STATE	FEDERAL AID PROJECT NO.	SHEET NO.
10	WA	ER-134(1)	96



PLAN

LEGEND

WS-2 BORING LOCATION AND DESIGNATION

SITE C PILE TEST LOCATION AND DESIGNATION

NOTES

1. PLAN FURNISHED BY THE DESIGN TEAM.
2. BORING AND PILE TEST LOCATIONS ARE APPROXIMATE.

0 50 100 150
SCALE IN FEET

V

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

WEST SEATTLE BRIDGE DESIGN TEAM
Seattle, Washington

Andersen • Bjornstad • Kane • Jacobs, Inc.
Kramer, Chin & Mayo, Inc.

Parsons, Brinckerhoff
Tudor Engineering Co.

THOMAS A. KANE STATE OF WASHINGTON REGISTERED PROFESSIONAL ENGINEER

APPROVED BY THE BOARD OF PUBLIC WORKS FOR ADVERTISING
SEATTLE, WASHINGTON ..... 19 .....

ATTEST: ..... CHAIRMAN

EXECUTIVE SECRETARY ..... PROJECT NO.

DESIGNED: M.G. Wu 5/8/80
DRAWN: T. Takemura 5/12/80
CHECKED: H. Dunleavy 5/27/80
REVIEWED:
FINAL CONST. CHECKED

APPROVED:
APPROVED:
APPROVED:
APPROVED:

Work done under standard specifications for Municipal Public Works Construction prepared by The Washington State Chapter American Public Works Association as modified by the City Of Seattle Standard Plans and Specifications, 10th Ed., Supplemented by Special Specifications.

File No. 733-14

THE CITY OF SEATTLE
DEPARTMENT OF ENGINEERING

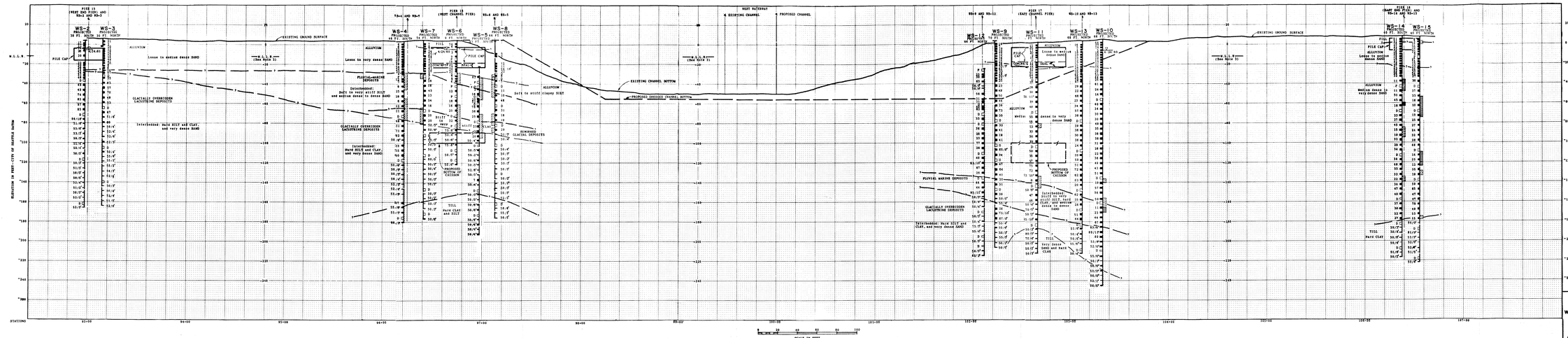
ORDINANCE NO. 109101 APPROVED JUNE 18, 1980
FUND:
SCALE: H. V. INSPECTOR'S BOOK NO.

WEST SEATTLE FREEWAY BRIDGE REPLACEMENT
MAIN SPAN SUBSTRUCTURE
SOIL EXPLORATION LOGS
PLAN AND LOG OF BORING WS-2

CONTRACT NO. 2
W/A NO.
PC S 0000 A
R/W S 0000 B
CO S 0000 C
REF. NO. B-1
CITY ENGR'S DWG. NO. 782-137
SHEET 96 OF 107

PERIOR REPROGRAPHICS TB422

FIG. 1



LEGEND

WS-11 — BORING LOCATION AND DESIGNATION, PROJECTED LESS THAN 15 FEET

-15.60 — WATER LEVEL DEPTH AND DATE MEASURED

SPT — SPLIT SPOON SAMPLE SHOWING N-VALUE

P — SHELBY TUBE SAMPLE

D — PITCHER SAMPLE

O — CUTTERHEAD SAMPLE

V — SAMPLE

BOTTOM OF BORING

WS-15 — BORING LOCATION AND DESIGNATION, PROJECTED MORE THAN 15 FEET

Loose to medium dense SILT, sandy SILT and silty fine SAND (fine-grained noncohesive soils)

Very soft to very stiff clayey SILT, silty CLAY, clayey sandy SILT and sandy silty CLAY (cohesive soils)

NOTES

1. SEE FIG. 3 FOR BORING, SECTION LINE AND PIER LOCATIONS.
2. THIS PROFILE IS GENERALIZED FROM MATERIALS ENCOUNTERED IN THE BORINGS. VARIATIONS BETWEEN THE PROFILE AND ACTUAL CONDITIONS MAY EXIST.
3. M.L.L.W. (MEAN LOWER LOW WATER). INDICATES THE APPROXIMATE ELEVATION OF THE GROUND SURFACE PRIOR TO PLACEMENT OF HYDRAULIC FILL FOR PIER 15.
4. BORINGS PROJECTED NORMALLY ONTO THE ALIGNMENT CENTERLINE ARE SHOWN TO INDICATE STANDARD PENETRATION TEST BLOW-COUNTS (N-VALUES). STRATIFICATION LINES SHOWN ON THE PROFILE THROUGH THESE PROJECTED BORINGS HAVE BEEN INFERRED.

CITY OF SEATTLE
WEST SEATTLE FREEWAY BRIDGE REPLACEMENT

GENERALIZED SUBSURFACE PROFILE ALONG PROPOSED ALIGNMENT

MAIN SPAN SUBSTRUCTURE
STA. 92+00 TO 107+00
JUNE 1980

W-3476-20

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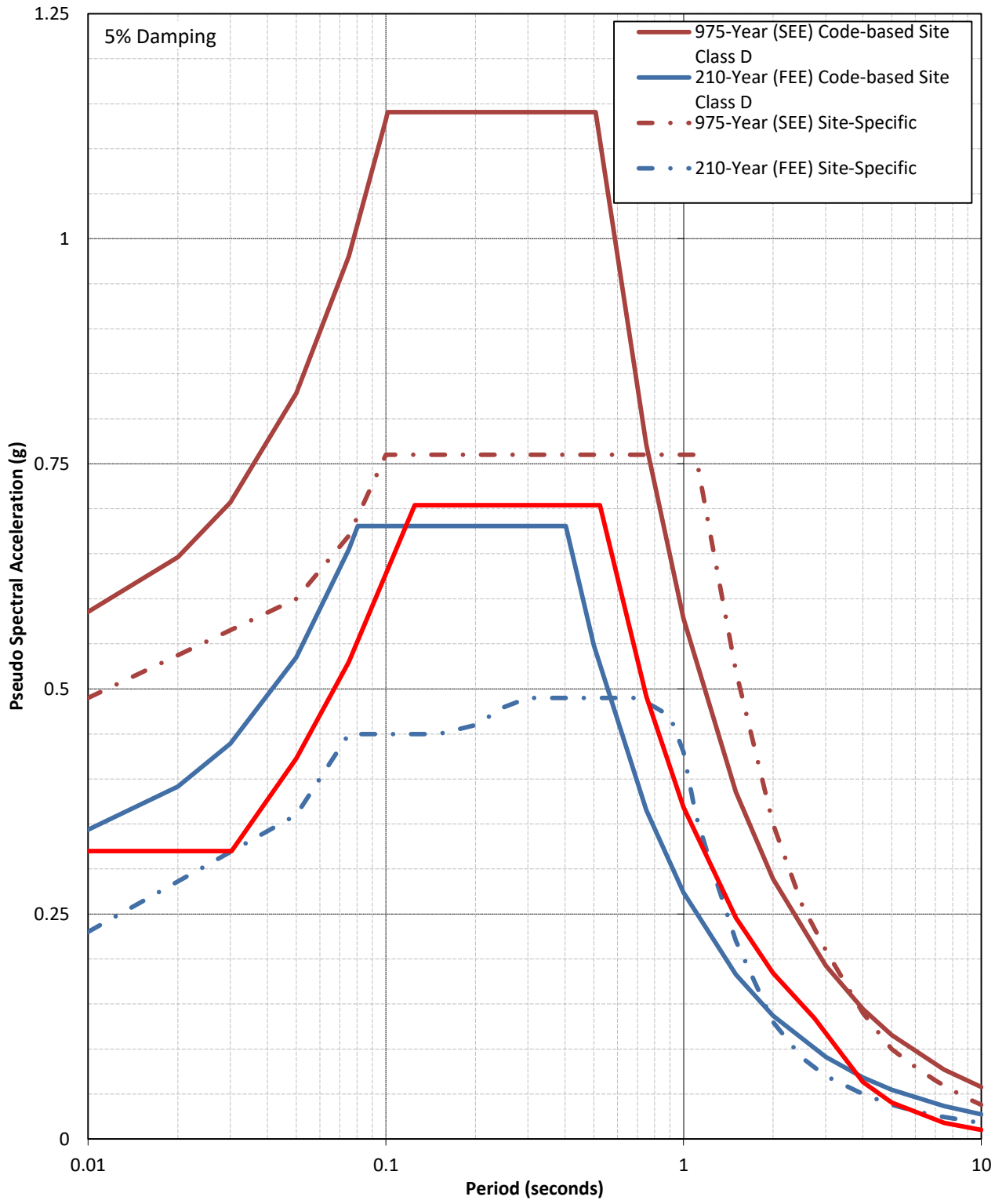
GEOTECHNICAL CONSULTANTS

FIG. 2



NOTES

FEE = functional evaluation earthquake; g = standard gravitational acceleration; SDOT = Seattle department of transportation; SEE = safety evaluation earthquake

SDOT
West Seattle High-Rise Bridge
Seattle, Washington

RECOMMENDED SITE-SPECIFIC HORIZONTAL DESIGN SPECTRA

October 2020 105492-001

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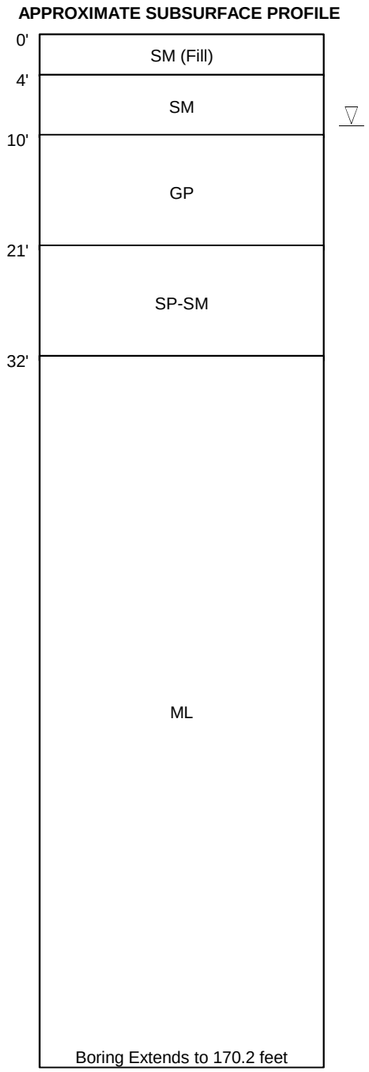
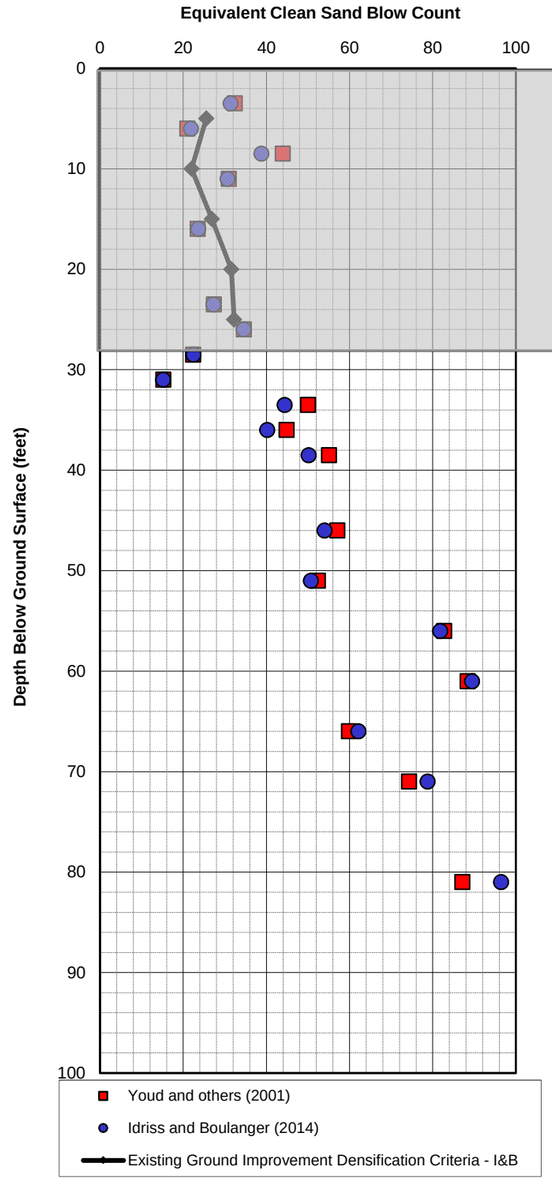
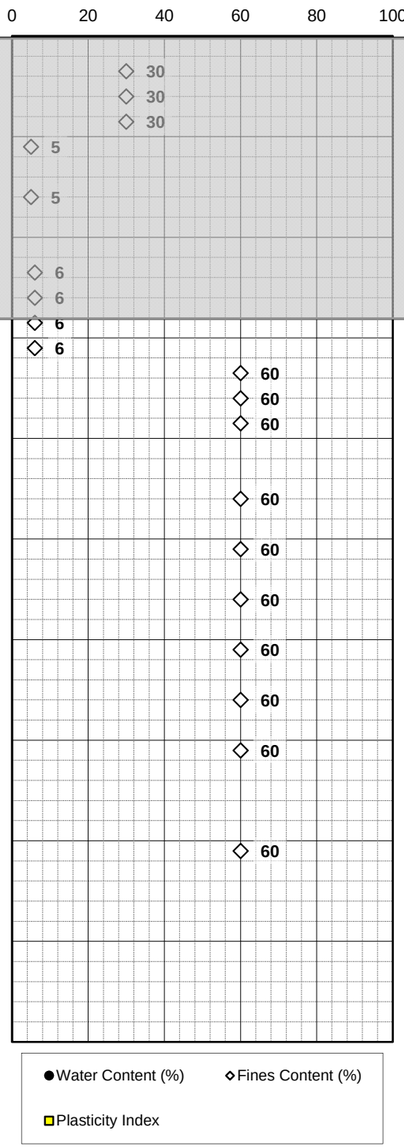
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

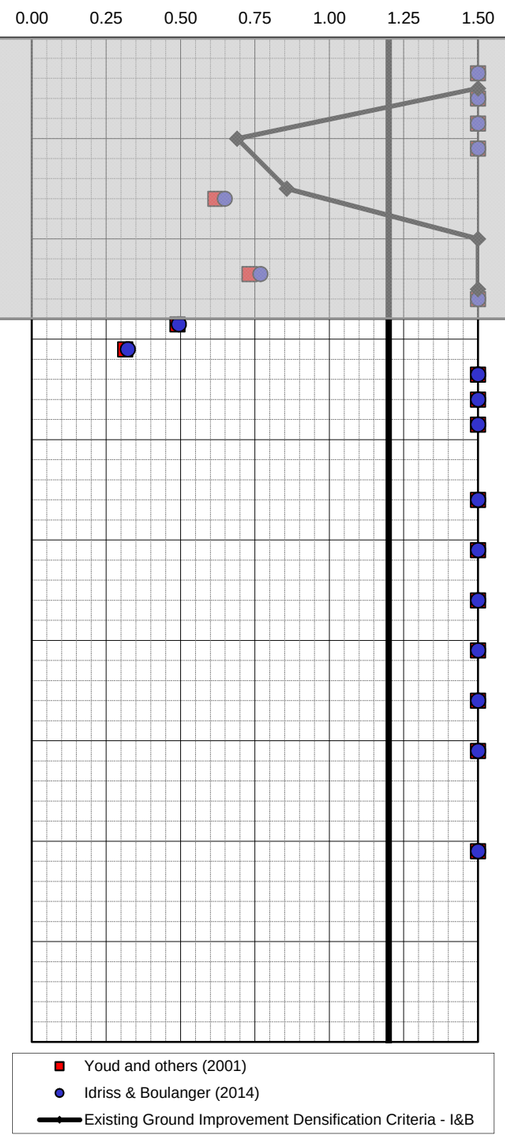
FIG. 3

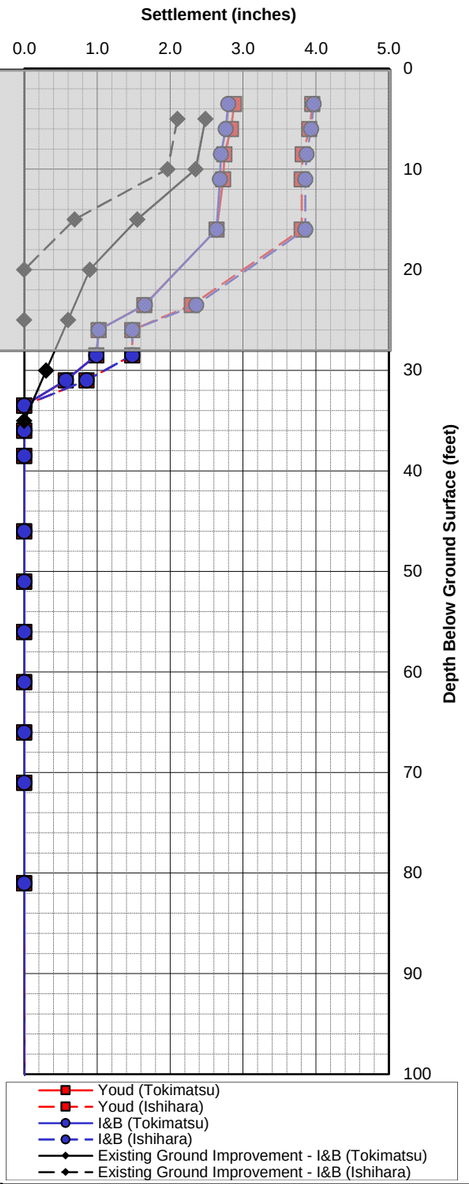
JLR











NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.
3. We estimated the seismic settlement below the water table using Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). We estimated the settlement above the water table using Pradel (1998).
4. Average shear wave velocity in upper 30 feet = 688 fps
   Hammer energy ratio = 60%
   Groundwater table depth = 9 feet
5. pcf = pounds per cubic foot
   g = gravitational acceleration
   PGA = peak ground acceleration
6. Liquefaction analyses are presented for both in situ pre-construction conditions and post-construction ground improved conditions.
7. Approximate depth of existing ground improvement is shaded in grey.

SDOT
West Seattle High-Rise Bridge
Seattle, Washington

LIQUEFACTION ANALYSIS SUMMARY
BORING WS-2 (PIER 15N)
M = 7.6, PGA = 0.51g

August 2020 105492

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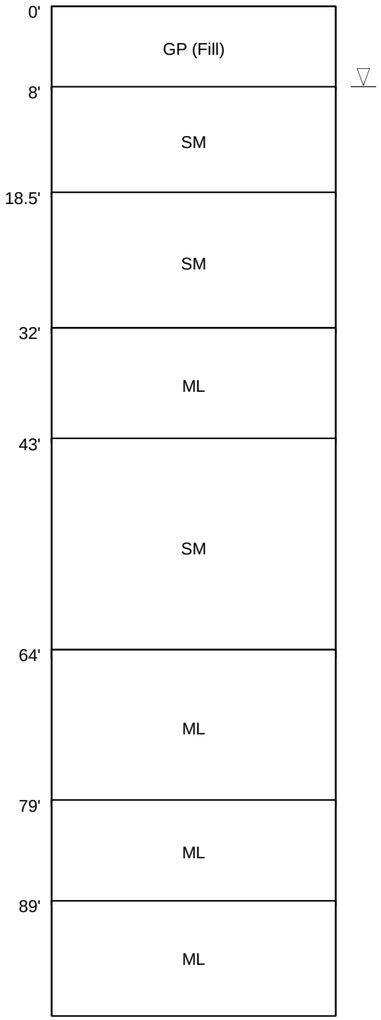
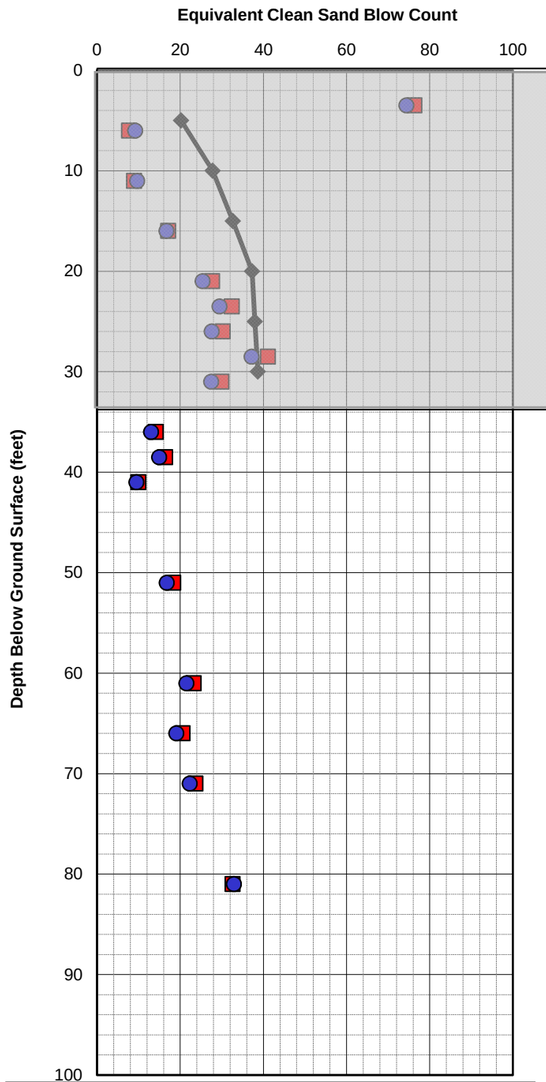
Geotechnical and Environmental Consultants

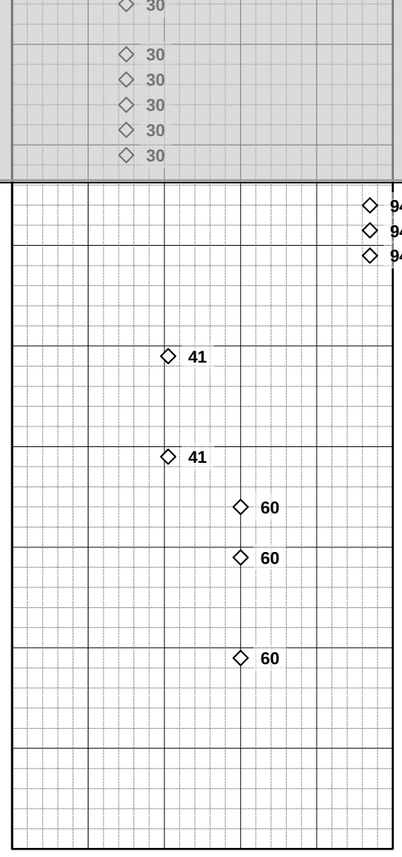
FIG. 4
Sheet 1 of 4

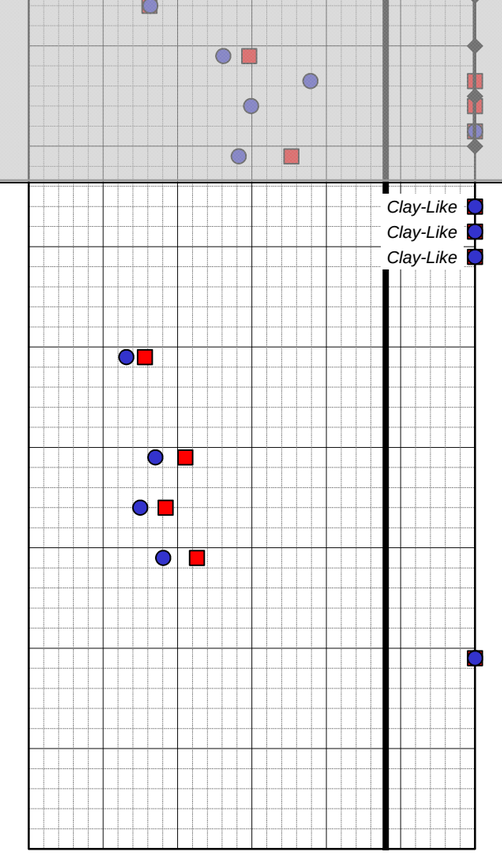
APPROXIMATE SUBSURFACE PROFILE

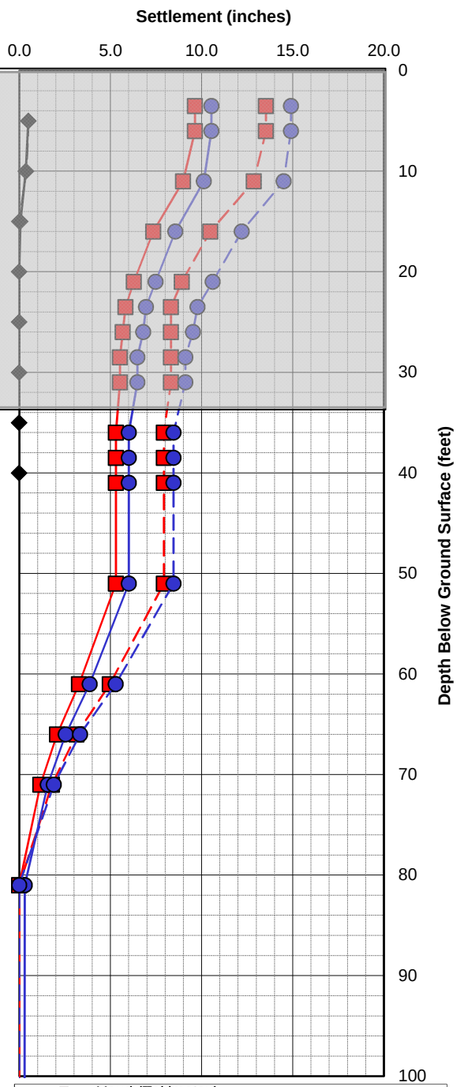


Boring Extends to 125.3 feet









NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.
3. We estimated the seismic settlement below the water table using Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). We estimated the settlement above the water table using Pradel (1998).
4. Average shear wave velocity in upper 30 feet = 574 fps
   Hammer energy ratio = 60%
   Groundwater table depth = 8 feet
5. pcf = pounds per cubic foot
   g = gravitational acceleration
   PGA = peak ground acceleration
6. Liquefaction analyses are presented for both in situ pre-construction conditions and post-construction ground improved conditions.
7. Approximate depth of existing ground improvement is shaded in grey.

SDOT
West Seattle High-Rise Bridge
Seattle, Washington

LIQUEFACTION ANALYSIS SUMMARY
BORING WS-6 (PIER 16C)
M = 7.6, PGA = 0.51g

August 2020 105492

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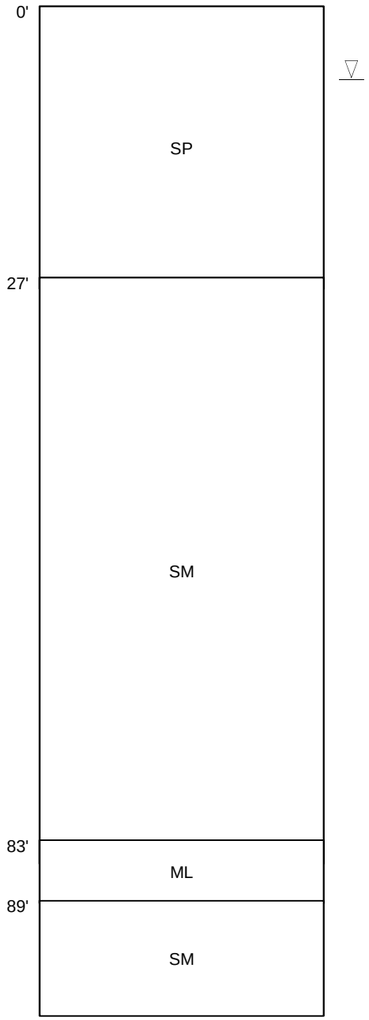
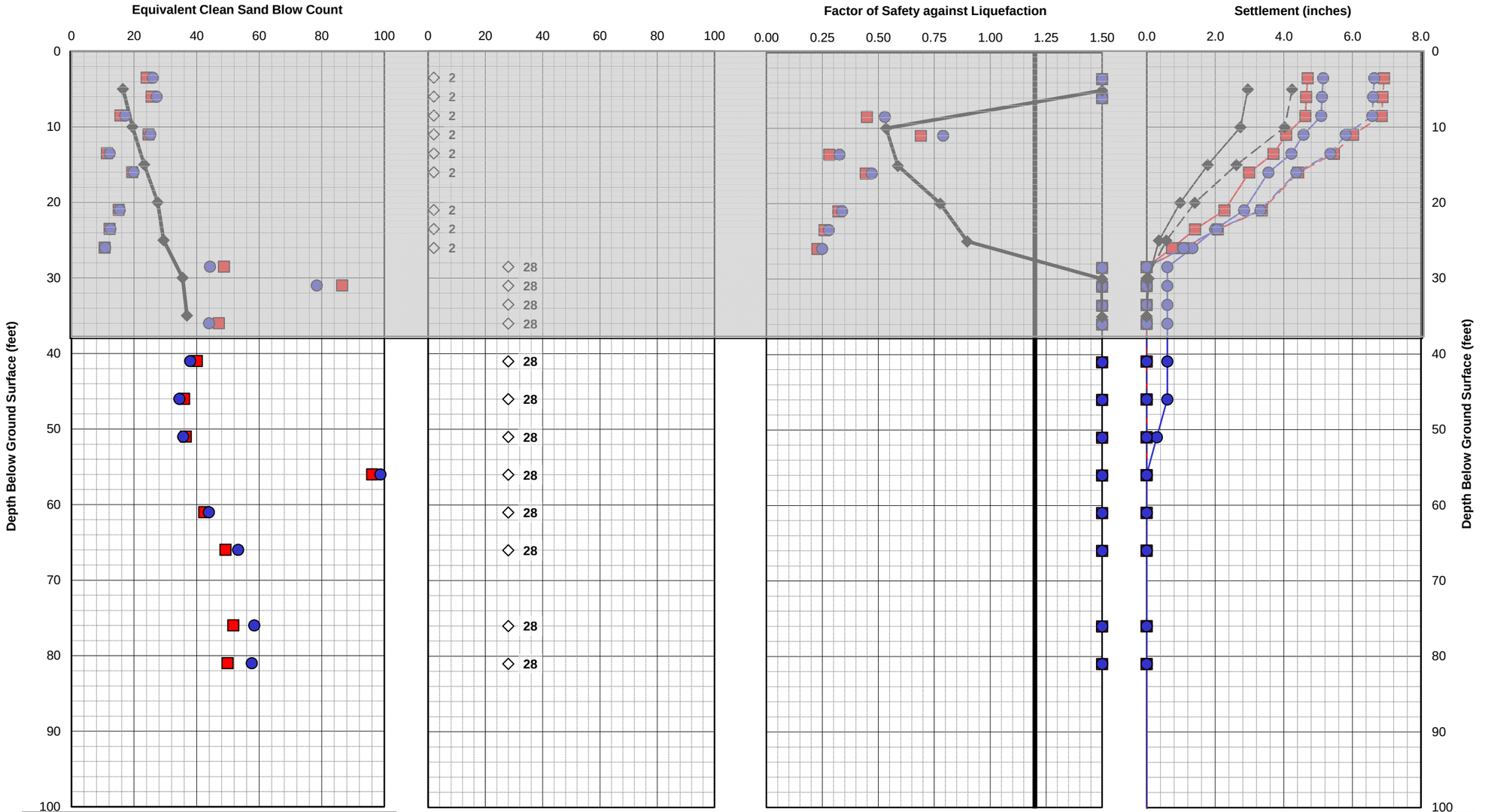
Geotechnical and Environmental Consultants

FIG. 4
Sheet 2 of 4

APPROXIMATE SUBSURFACE PROFILE



Boring Extends to 215.2 feet



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.
3. We estimated the seismic settlement below the water table using Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). We estimated the settlement above the water table using Pradel (1998).
4. Average shear wave velocity in upper 30 feet = 656 fps
   Hammer energy ratio = 60%
   Groundwater table depth = 7 feet
5. pcf = pounds per cubic foot
   g = gravitational acceleration
   PGA = peak ground acceleration
6. Liquefaction analyses are presented for both in situ pre-construction conditions and post-construction ground improved conditions.
7. Approximate depth of existing ground improvement is shaded in grey.

SDOT
West Seattle High-Rise Bridge
Seattle, Washington

LIQUEFACTION ANALYSIS SUMMARY
BORING WS-11 (PIER 17C)
M = 7.6, PGA = 0.51g

August 2020 105492

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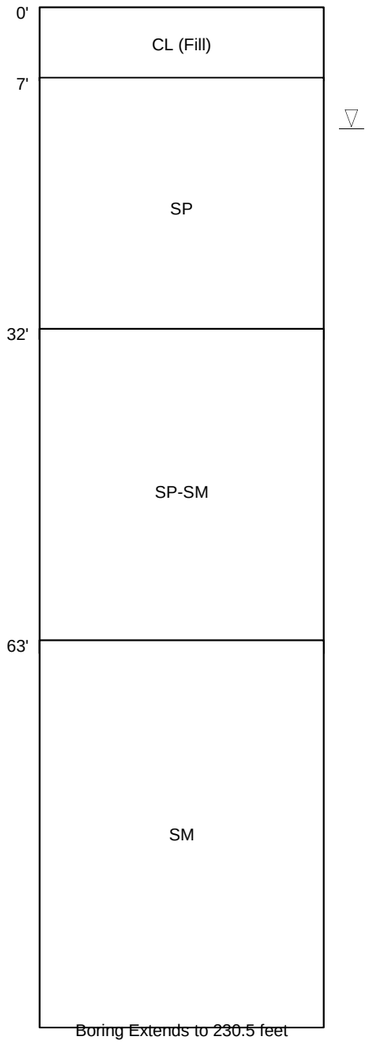
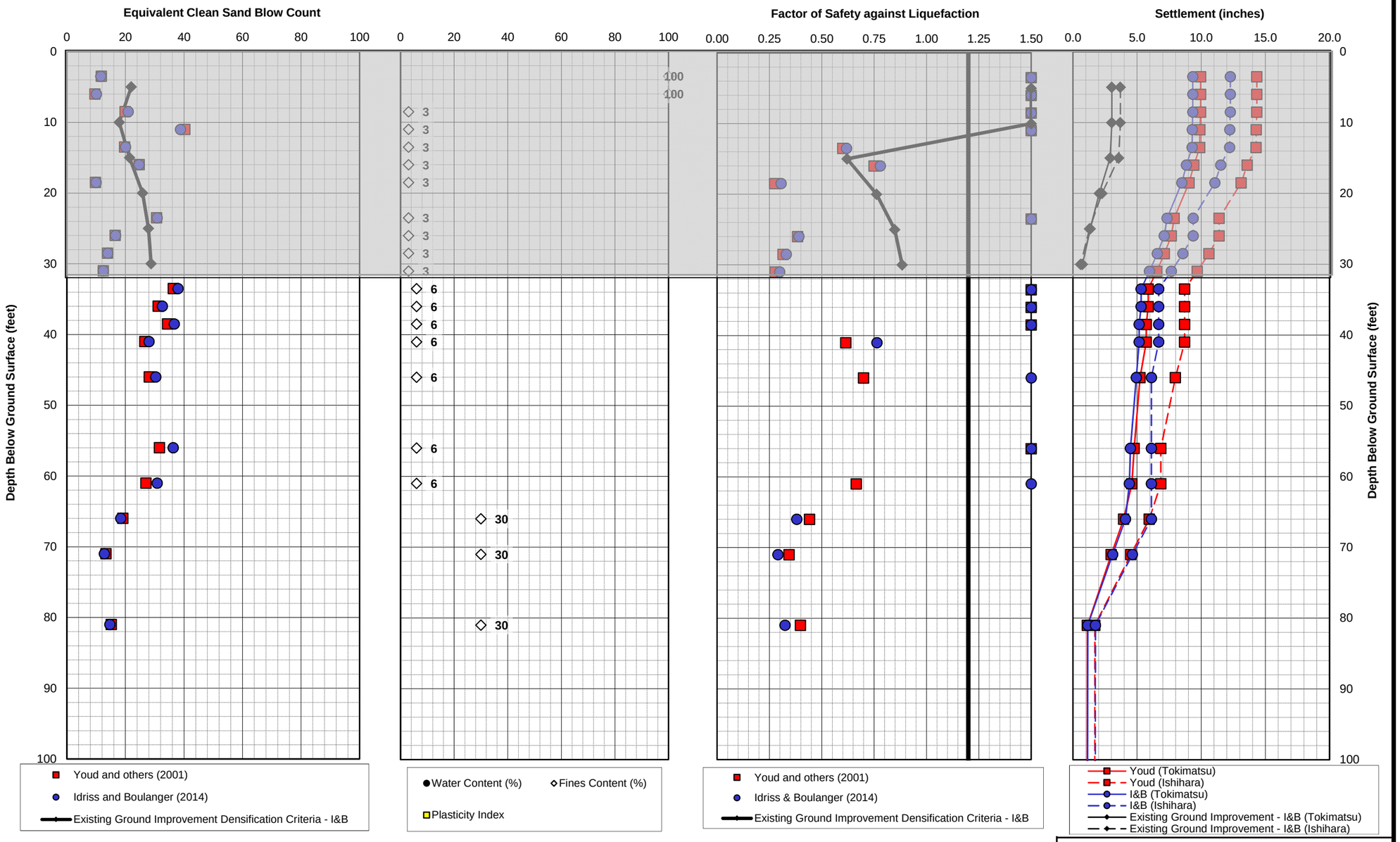
FIG. 4
Sheet 3 of 4

APPROXIMATE SUBSURFACE PROFILE





NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.
3. We estimated the seismic settlement below the water table using Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). We estimated the settlement above the water table using Pradel (1998).
4. Average shear wave velocity in upper 30 feet = 626 fps
   Hammer energy ratio = 60%
   Groundwater table depth = 12 feet
5. pcf = pounds per cubic foot
   g = gravitational acceleration
   PGA = peak ground acceleration
6. Liquefaction analyses are presented for both in situ pre-construction conditions and post-construction ground improved conditions.
7. Approximate depth of existing ground improvement is shaded in grey.

SDOT
West Seattle High-Rise Bridge
Seattle, Washington

LIQUEFACTION ANALYSIS SUMMARY
BORING WS-15 (PIER 18S)
M = 7.6, PGA = 0.51g

August 2020 105492

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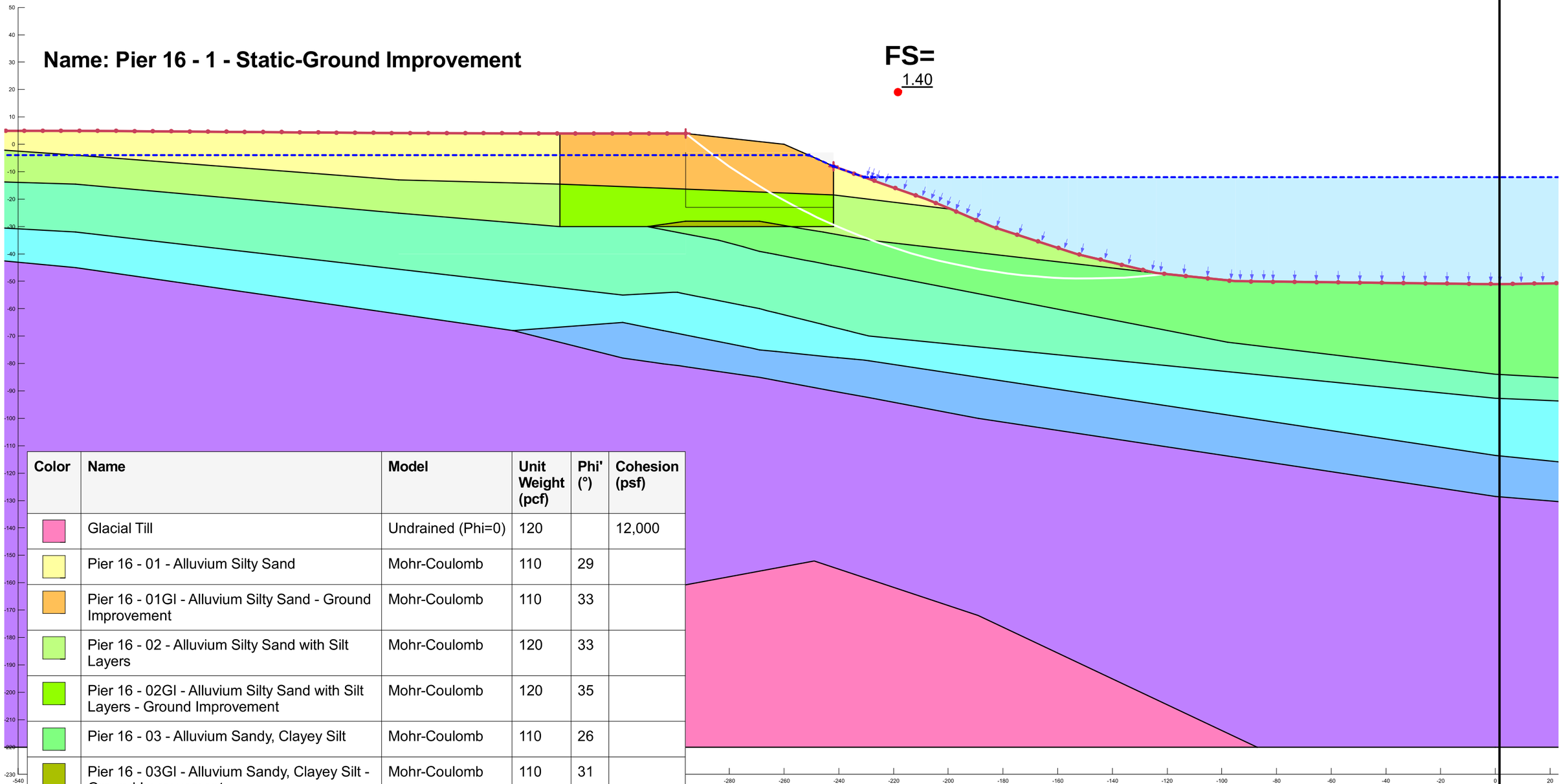
Geotechnical and Environmental Consultants

FIG. 4
Sheet 4 of 4



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 16 - 01 - Alluvium Silty Sand	Mohr-Coulomb	110	29	
	Pier 16 - 01GI - Alluvium Silty Sand - Ground Improvement	Mohr-Coulomb	110	33	
	Pier 16 - 02 - Alluvium Silty Sand with Silt Layers	Mohr-Coulomb	120	33	
	Pier 16 - 02GI - Alluvium Silty Sand with Silt Layers - Ground Improvement	Mohr-Coulomb	120	35	
	Pier 16 - 03 - Alluvium Sandy, Clayey Silt	Mohr-Coulomb	110	26	
	Pier 16 - 03GI - Alluvium Sandy, Clayey Silt - Ground Improvement	Mohr-Coulomb	110	31	
	Pier 16 - 04 - Fluvial-Marine - Silty Sand	Mohr-Coulomb	120	31	
	Pier 16 - 05 - Fluvial-Marine - Clayey, Sandy Silt	Mohr-Coulomb	120	29	
	Pier 16 - 06 - Glacial Lacustrine - Clayey Sandy Silt	Undrained (Phi=0)	120		4,000
	Pier 16 - 07 - Glacial Lacustrine - Sandy Clayey Silt and Silty Clay	Undrained (Phi=0)	120		8,000

SDOT
West Seattle High Bridge
Seattle, Washington

SLOPE STABILITY ANLAYSIS
PIER 16

August 2020 105492

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Geotechnical and Environmental Consultants

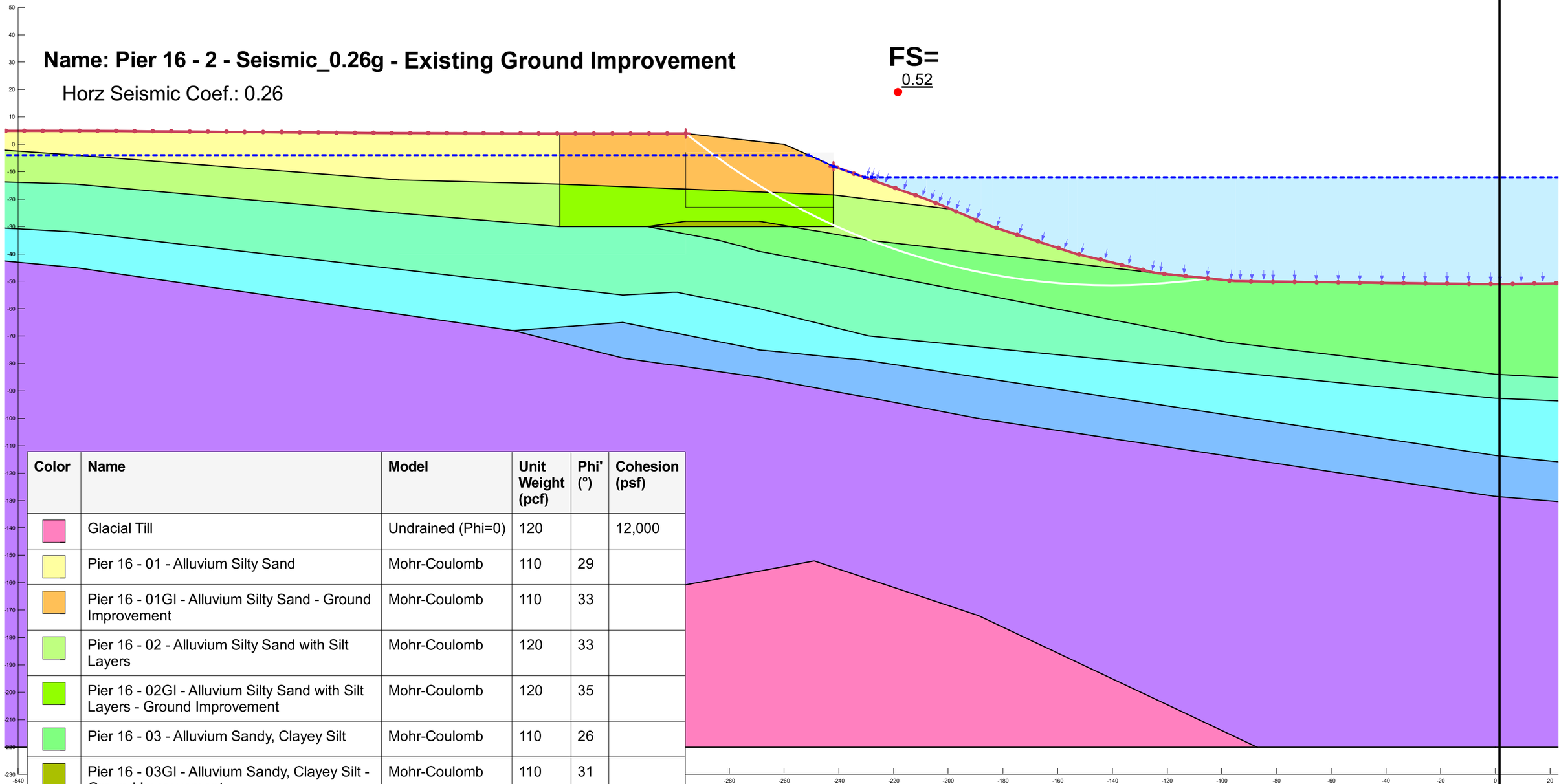
FIG. 5
Sheet 1 of 10

R:\EFS\EA\105000s\105492.SDOT, West Seattle High-Rise Bridge\Proposal\Analysis\Slope Stability\Pier 16.gsz

Name: Pier 16 - 2 - Seismic_0.26g - Existing Ground Improvement

Horz Seismic Coef.: 0.26

FS=
0.52



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 16 - 01 - Alluvium Silty Sand	Mohr-Coulomb	110	29	
	Pier 16 - 01GI - Alluvium Silty Sand - Ground Improvement	Mohr-Coulomb	110	33	
	Pier 16 - 02 - Alluvium Silty Sand with Silt Layers	Mohr-Coulomb	120	33	
	Pier 16 - 02GI - Alluvium Silty Sand with Silt Layers - Ground Improvement	Mohr-Coulomb	120	35	
	Pier 16 - 03 - Alluvium Sandy, Clayey Silt	Mohr-Coulomb	110	26	
	Pier 16 - 03GI - Alluvium Sandy, Clayey Silt - Ground Improvement	Mohr-Coulomb	110	31	
	Pier 16 - 04 - Fluvial-Marine - Silty Sand	Mohr-Coulomb	120	31	
	Pier 16 - 05 - Fluvial-Marine - Clayey, Sandy Silt	Mohr-Coulomb	120	29	
	Pier 16 - 06 - Glacial Lacustrine - Clayey Sandy Silt	Undrained (Phi=0)	120		4,000
	Pier 16 - 07 - Glacial Lacustrine - Sandy Clayey Silt and Silty Clay	Undrained (Phi=0)	120		8,000

SDOT
West Seattle High Bridge
Seattle, Washington

SLOPE STABILITY ANLAYSIS
PIER 16

August 2020 105492

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Geotechnical and Environmental Consultants

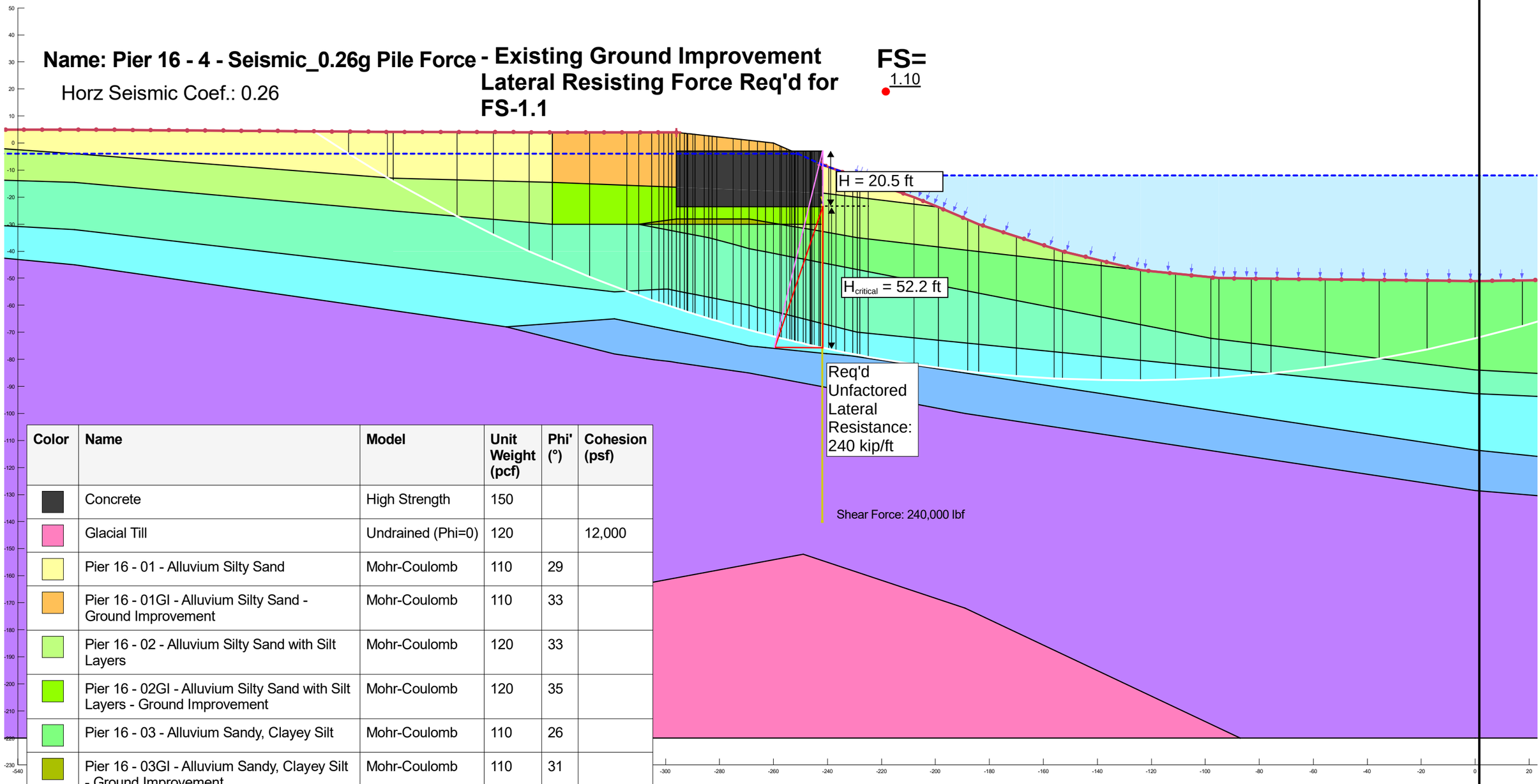
FIG. 5

R:\EFS\EA\105000s\105492.SDOT, West Seattle High-Rise Bridge\Proposal\Analysis\Slope Stability\Pier 16.gsz

Name: Pier 16 - 4 - Seismic_0.26g Pile Force - Existing Ground Improvement Lateral Resisting Force Req'd for FS-1.1

Horz Seismic Coef.: 0.26

FS= 1.10



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Concrete	High Strength	150		
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 16 - 01 - Alluvium Silty Sand	Mohr-Coulomb	110	29	
	Pier 16 - 01GI - Alluvium Silty Sand - Ground Improvement	Mohr-Coulomb	110	33	
	Pier 16 - 02 - Alluvium Silty Sand with Silt Layers	Mohr-Coulomb	120	33	
	Pier 16 - 02GI - Alluvium Silty Sand with Silt Layers - Ground Improvement	Mohr-Coulomb	120	35	
	Pier 16 - 03 - Alluvium Sandy, Clayey Silt	Mohr-Coulomb	110	26	
	Pier 16 - 03GI - Alluvium Sandy, Clayey Silt - Ground Improvement	Mohr-Coulomb	110	31	
	Pier 16 - 04 - Fluvial-Marine - Silty Sand	Mohr-Coulomb	120	31	
	Pier 16 - 05 - Fluvial-Marine - Clayey, Sandy Silt	Mohr-Coulomb	120	29	
	Pier 16 - 06 - Glacial Lacustrine - Clayey Sandy Silt	Undrained (Phi=0)	120		4,000
	Pier 16 - 07 - Glacial Lacustrine - Sandy Clayey Silt and Silty Clay	Undrained (Phi=0)	120		8,000

SDOT
West Seattle High Bridge
Seattle, Washington

SLOPE STABILITY ANLAYSIS
PIER 16

August 2020 105492

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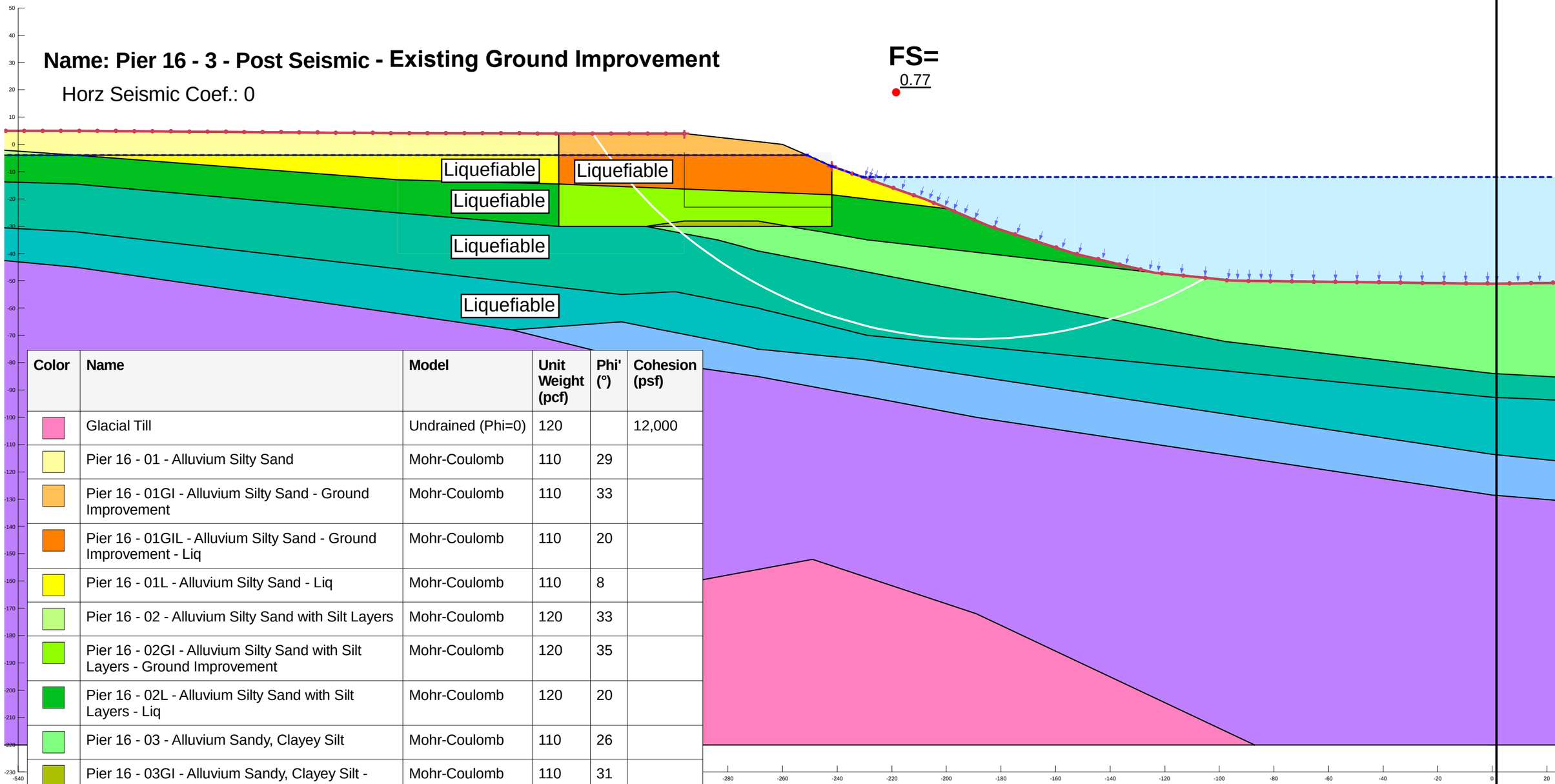
FIG. 5

\\EFNSEA\105000s\105492 SDOT\West Seattle High-Rise Bridge\Proposal\Analysis\Slope Stability\Pier 16_082520.gsz

Name: Pier 16 - 3 - Post Seismic - Existing Ground Improvement

Horz Seismic Coef.: 0

FS=
0.77



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 16 - 01 - Alluvium Silty Sand	Mohr-Coulomb	110	29	
	Pier 16 - 01GI - Alluvium Silty Sand - Ground Improvement	Mohr-Coulomb	110	33	
	Pier 16 - 01GIL - Alluvium Silty Sand - Ground Improvement - Liq	Mohr-Coulomb	110	20	
	Pier 16 - 01L - Alluvium Silty Sand - Liq	Mohr-Coulomb	110	8	
	Pier 16 - 02 - Alluvium Silty Sand with Silt Layers	Mohr-Coulomb	120	33	
	Pier 16 - 02GI - Alluvium Silty Sand with Silt Layers - Ground Improvement	Mohr-Coulomb	120	35	
	Pier 16 - 02L - Alluvium Silty Sand with Silt Layers - Liq	Mohr-Coulomb	120	20	
	Pier 16 - 03 - Alluvium Sandy, Clayey Silt	Mohr-Coulomb	110	26	
	Pier 16 - 03GI - Alluvium Sandy, Clayey Silt - Ground Improvement	Mohr-Coulomb	110	31	
	Pier 16 - 04L - Fluvial-Marine - Silty Sand - Liq	Mohr-Coulomb	120	10	
	Pier 16 - 05L - Fluvial-Marine - Clayey, Sandy Silt - Liq	Mohr-Coulomb	120	10	
	Pier 16 - 06 - Glacial Lacustrine - Clayey Sandy Silt	Undrained (Phi=0)	120		4,000
	Pier 16 - 07 - Glacial Lacustrine - Sandy Clayey Silt and Silty Clay	Undrained (Phi=0)	120		8,000

Liquefiable layer friction angles based on S&W spreadsheet

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SLOPE STABILITY ANLAYSIS
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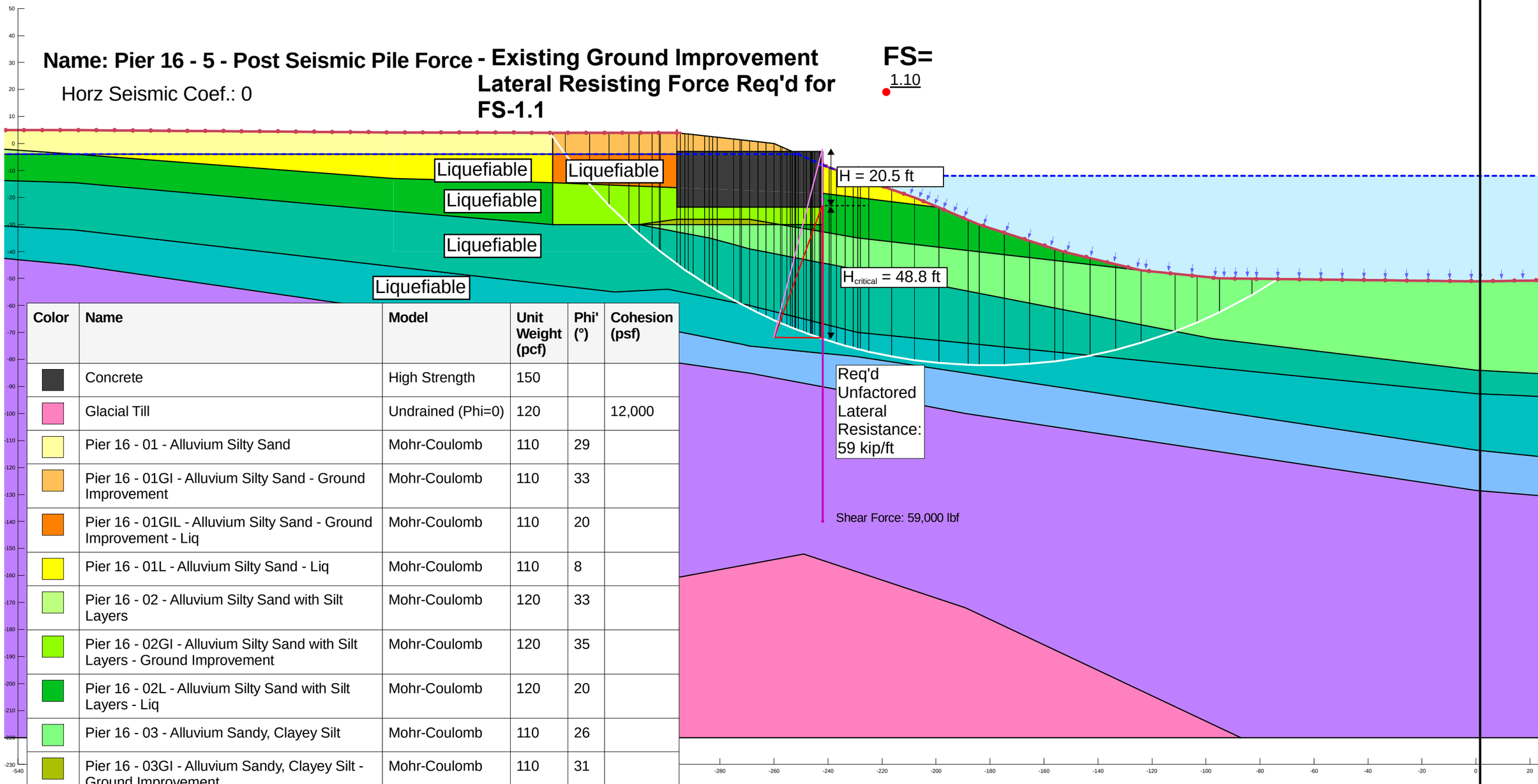
FIG. 5
Sheet 4 of 10

A:\E\SEA\105000\105492.SDOT, West Seattle High Rise Bridge\Proposal\Analysis\Slope Stability\Pier 16_082520.gsz

Name: Pier 16 - 5 - Post Seismic Pile Force - Existing Ground Improvement Lateral Resisting Force Req'd for FS-1.1

Horz Seismic Coef.: 0

FS= 1.10



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Concrete	High Strength	150		
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 16 - 01 - Alluvium Silty Sand	Mohr-Coulomb	110	29	
	Pier 16 - 01GI - Alluvium Silty Sand - Ground Improvement	Mohr-Coulomb	110	33	
	Pier 16 - 01GIL - Alluvium Silty Sand - Ground Improvement - Liq	Mohr-Coulomb	110	20	
	Pier 16 - 01L - Alluvium Silty Sand - Liq	Mohr-Coulomb	110	8	
	Pier 16 - 02 - Alluvium Silty Sand with Silt Layers	Mohr-Coulomb	120	33	
	Pier 16 - 02GI - Alluvium Silty Sand with Silt Layers - Ground Improvement	Mohr-Coulomb	120	35	
	Pier 16 - 02L - Alluvium Silty Sand with Silt Layers - Liq	Mohr-Coulomb	120	20	
	Pier 16 - 03 - Alluvium Sandy, Clayey Silt	Mohr-Coulomb	110	26	
	Pier 16 - 03GI - Alluvium Sandy, Clayey Silt - Ground Improvement	Mohr-Coulomb	110	31	
	Pier 16 - 04L - Fluvial-Marine - Silty Sand - Liq	Mohr-Coulomb	120	10	
	Pier 16 - 05L - Fluvial-Marine - Clayey, Sandy Silt - Liq	Mohr-Coulomb	120	10	
	Pier 16 - 06 - Glacial Lacustrine - Clayey Sandy Silt	Undrained (Phi=0)	120		4,000
	Pier 16 - 07 - Glacial Lacustrine - Sandy Clayey Silt and Silty Clay	Undrained (Phi=0)	120		8,000

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SLOPE STABILITY ANLAYSIS PIER 16

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FIG. 5
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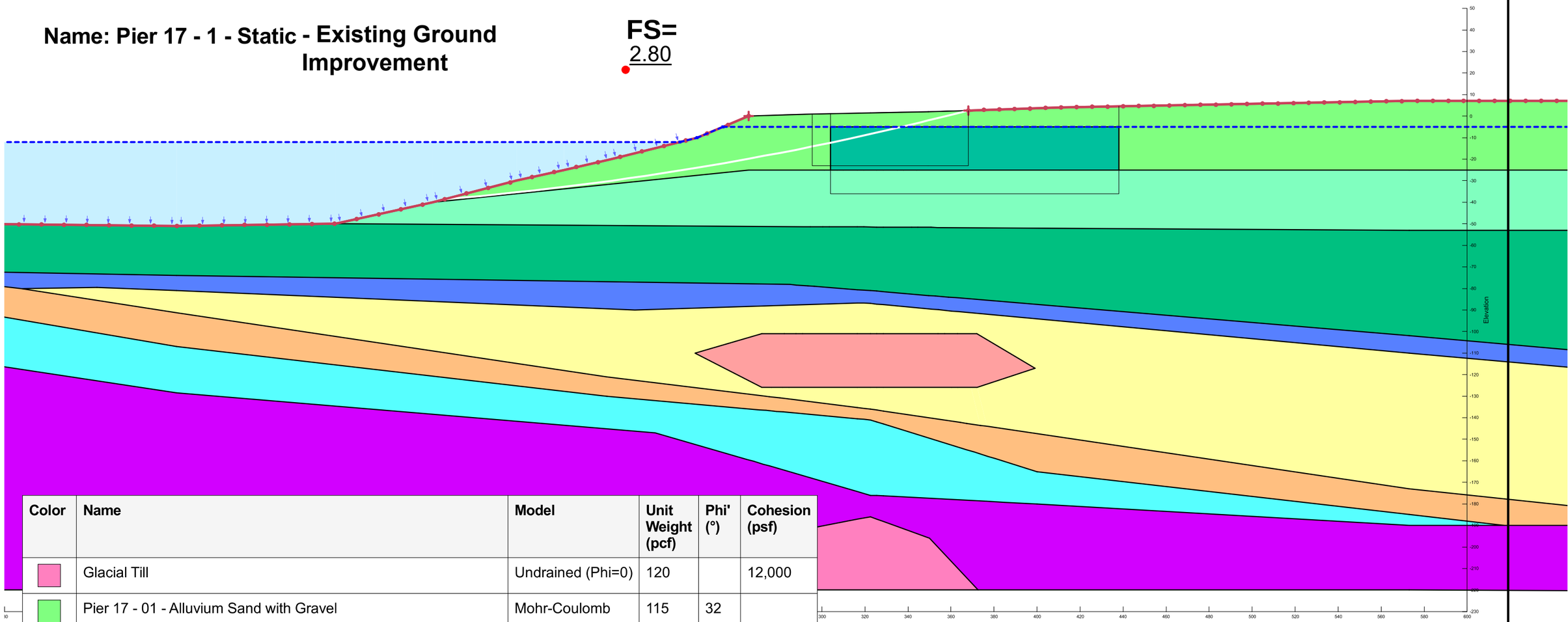
R:\EF\SEA\105000s\105492 SDOT, West Seattle High+Rise Bridge\Proposal\Analysis\Slope Stability\Pier 16_082520.gsz

Name: Pier 17 - 1 - Static - Existing Ground Improvement

FS=
2.80



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 17 - 01 - Alluvium Sand with Gravel	Mohr-Coulomb	115	32	
	Pier 17 - 01GI - Alluvium Sand with Gravel - Ground Improvement	Mohr-Coulomb	115	33	
	Pier 17 - 02 - Alluvium Silty Sand with Clayey Silt	Mohr-Coulomb	125	36	
	Pier 17 - 03 - Alluvium Silty Sand	Mohr-Coulomb	125	38	
	Pier 17 - 04 - Alluvium Clayey, Sandy Silt	Mohr-Coulomb	115	27	
	Pier 17 - 05 - Alluvium Silty Sand 2	Mohr-Coulomb	125	32	
	Pier 17 - 06 - Alluvium Interbedded Silty Clay and Sandy Silt	Mohr-Coulomb	115	34	
	Pier 17 - 08 - Alluvium Clayey, Sandy Silt with Silty Sand layers	Mohr-Coulomb	105	24	
	Pier 17 - 09 - Fluvial-Marine - Clayey Silt with Sand	Mohr-Coulomb	110	39	
	Pier 17 - 11 - Glacial Lacustrine - Silty Clay with Sand and Gravel	Undrained (Phi=0)	115		8,000

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SLOPE STABILITY ANLAYSIS
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FIG. 5
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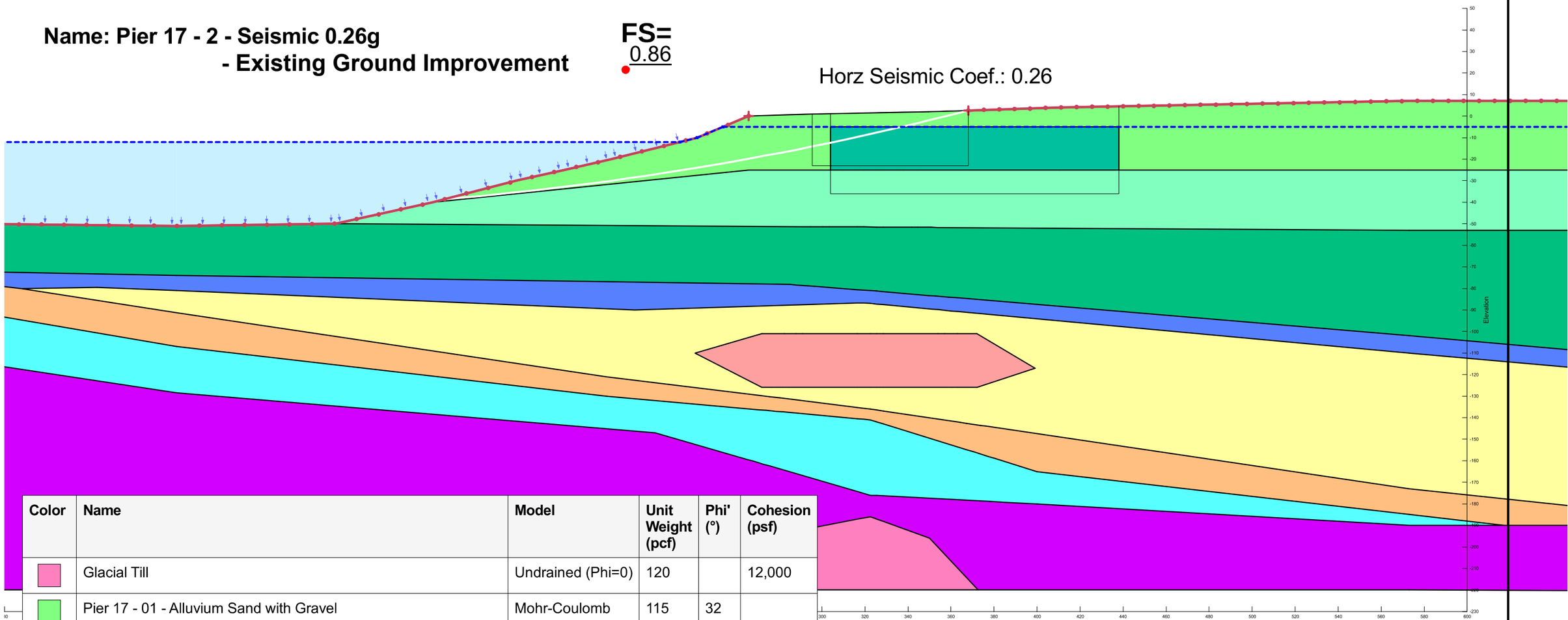
I:\EPSEA\105000s\105492 SDOT West Seattle High-Rise Bridge\Proposal\Analysis\Slope Stability\Pier 17.gsz

Name: Pier 17 - 2 - Seismic 0.26g
- Existing Ground Improvement

FS=
0.86



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 17 - 01 - Alluvium Sand with Gravel	Mohr-Coulomb	115	32	
	Pier 17 - 01GI - Alluvium Sand with Gravel - Ground Improvement	Mohr-Coulomb	115	33	
	Pier 17 - 02 - Alluvium Silty Sand with Clayey Silt	Mohr-Coulomb	125	36	
	Pier 17 - 03 - Alluvium Silty Sand	Mohr-Coulomb	125	38	
	Pier 17 - 04 - Alluvium Clayey, Sandy Silt	Mohr-Coulomb	115	27	
	Pier 17 - 05 - Alluvium Silty Sand 2	Mohr-Coulomb	125	32	
	Pier 17 - 06 - Alluvium Interbedded Silty Clay and Sandy Silt	Mohr-Coulomb	115	34	
	Pier 17 - 08 - Alluvium Clayey, Sandy Silt with Silty Sand layers	Mohr-Coulomb	105	24	
	Pier 17 - 09 - Fluvial-Marine - Clayey Silt with Sand	Mohr-Coulomb	110	39	
	Pier 17 - 11 - Glacial Lacustrine - Silty Clay with Sand and Gravel	Undrained (Phi=0)	115		8,000

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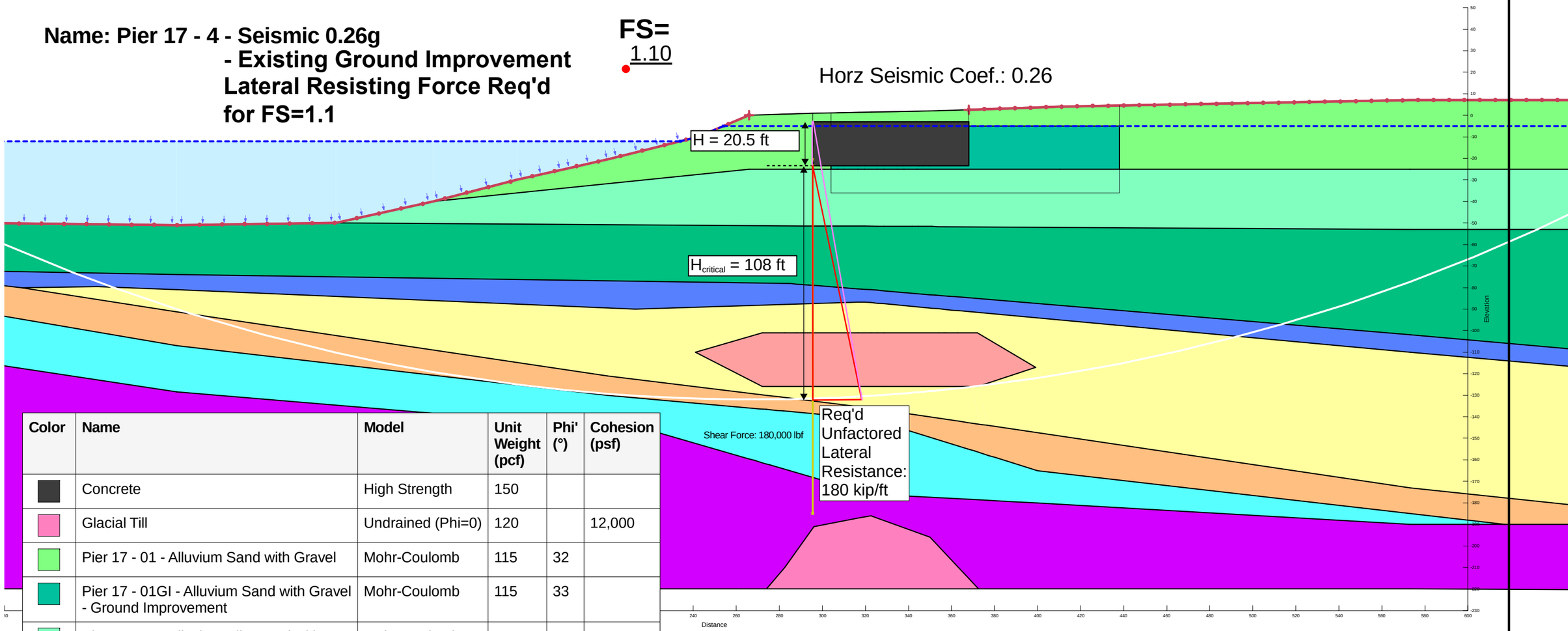
FIG. 5
Sheet 7 of 10

I:\EPSEA\105000s\105492 SDOT West Seattle High-Rise Bridge\Proposal\Analysis\Slope Stability\Pier 17.gsz

Name: Pier 17 - 4 - Seismic 0.26g
- Existing Ground Improvement
Lateral Resisting Force Req'd
for FS=1.1

FS=
1.10



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Concrete	High Strength	150		
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 17 - 01 - Alluvium Sand with Gravel	Mohr-Coulomb	115	32	
	Pier 17 - 01GI - Alluvium Sand with Gravel - Ground Improvement	Mohr-Coulomb	115	33	
	Pier 17 - 02 - Alluvium Silty Sand with Clayey Silt	Mohr-Coulomb	125	36	
	Pier 17 - 03 - Alluvium Silty Sand	Mohr-Coulomb	125	38	
	Pier 17 - 04 - Alluvium Clayey, Sandy Silt	Mohr-Coulomb	115	27	
	Pier 17 - 05 - Alluvium Silty Sand 2	Mohr-Coulomb	125	32	
	Pier 17 - 06 - Alluvium Interbedded Silty Clay and Sandy Silt	Mohr-Coulomb	115	34	
	Pier 17 - 08 - Alluvium Clayey, Sandy Silt with Silty Sand layers	Mohr-Coulomb	105	24	
	Pier 17 - 09 - Fluvial-Marine - Clayey Silt with Sand	Mohr-Coulomb	110	39	
	Pier 17 - 11 - Glacial Lacustrine - Silty Clay with Sand and Gravel	Undrained (Phi=0)	115		8,000

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SLOPE STABILITY ANLAYSIS
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FIG. 5
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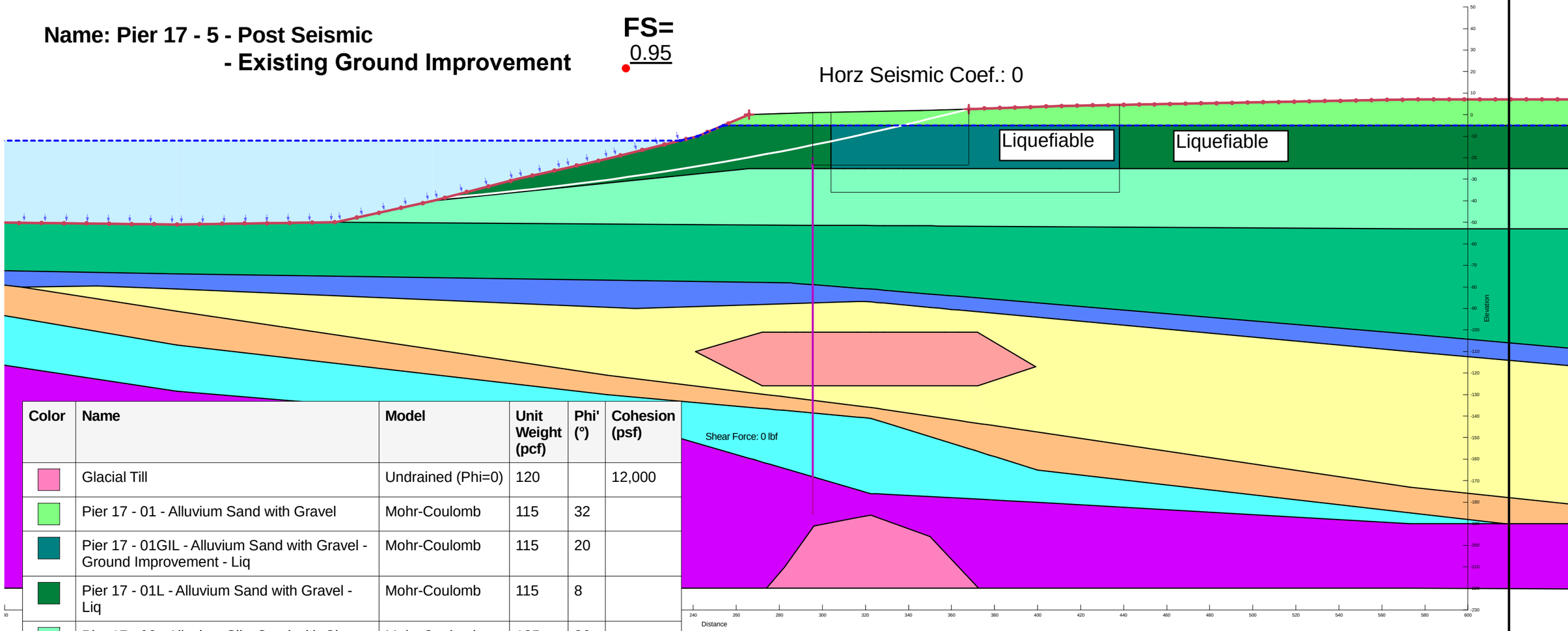
I:\EF\SEA\105000s\105492.SDOT.West Seattle High+Rise Bridge\Proposal\Analysis\Slope Stability\Pier 17_082520.gsz

Name: Pier 17 - 5 - Post Seismic
- Existing Ground Improvement

FS=
0.95



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 17 - 01 - Alluvium Sand with Gravel	Mohr-Coulomb	115	32	
	Pier 17 - 01GIL - Alluvium Sand with Gravel - Ground Improvement - Liq	Mohr-Coulomb	115	20	
	Pier 17 - 01L - Alluvium Sand with Gravel - Liq	Mohr-Coulomb	115	8	
	Pier 17 - 02 - Alluvium Silty Sand with Clayey Silt	Mohr-Coulomb	125	36	
	Pier 17 - 03 - Alluvium Silty Sand	Mohr-Coulomb	125	38	
	Pier 17 - 04 - Alluvium Clayey, Sandy Silt	Mohr-Coulomb	115	27	
	Pier 17 - 05 - Alluvium Silty Sand 2	Mohr-Coulomb	125	32	
	Pier 17 - 06 - Alluvium Interbedded Silty Clay and Sandy Silt	Mohr-Coulomb	115	34	
	Pier 17 - 08 - Alluvium Clayey, Sandy Silt with Silty Sand layers	Mohr-Coulomb	105	24	
	Pier 17 - 09 - Fluvial-Marine - Clayey Silt with Sand	Mohr-Coulomb	110	39	
	Pier 17 - 11 - Glacial Lacustrine - Silty Clay with Sand and Gravel	Undrained (Phi=0)	115		8,000

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SLOPE STABILITY ANLAYSIS
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FIG. 5
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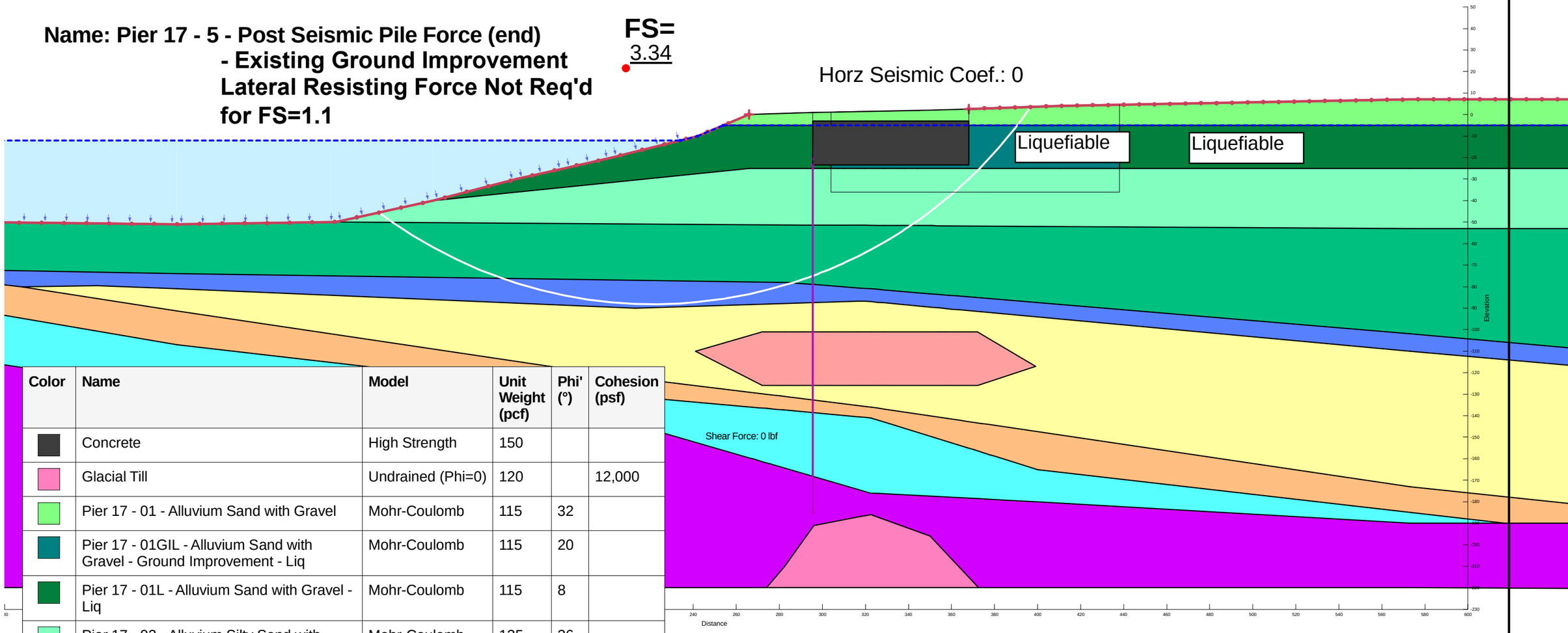
I:\EF\SEA\105000s\105492 SDOT, West Seattle High-Rise Bridge\Proposal\Analysis\Slope Stability\Per 17_082520.gsz

Name: Pier 17 - 5 - Post Seismic Pile Force (end) - Existing Ground Improvement Lateral Resisting Force Not Req'd for FS=1.1

FS= 3.34



Color	Name	Model	Unit Weight (pcf)	Phi' (°)	Cohesion (psf)
	Concrete	High Strength	150		
	Glacial Till	Undrained (Phi=0)	120		12,000
	Pier 17 - 01 - Alluvium Sand with Gravel	Mohr-Coulomb	115	32	
	Pier 17 - 01GIL - Alluvium Sand with Gravel - Ground Improvement - Liq	Mohr-Coulomb	115	20	
	Pier 17 - 01L - Alluvium Sand with Gravel - Liq	Mohr-Coulomb	115	8	
	Pier 17 - 02 - Alluvium Silty Sand with Clayey Silt	Mohr-Coulomb	125	36	
	Pier 17 - 03 - Alluvium Silty Sand	Mohr-Coulomb	125	38	
	Pier 17 - 04 - Alluvium Clayey, Sandy Silt	Mohr-Coulomb	115	27	
	Pier 17 - 05 - Alluvium Silty Sand 2	Mohr-Coulomb	125	32	
	Pier 17 - 06 - Alluvium Interbedded Silty Clay and Sandy Silt	Mohr-Coulomb	115	34	
	Pier 17 - 08 - Alluvium Clayey, Sandy Silt with Silty Sand layers	Mohr-Coulomb	105	24	
	Pier 17 - 09 - Fluvial-Marine - Clayey Silt with Sand	Mohr-Coulomb	110	39	
	Pier 17 - 11 - Glacial Lacustrine - Silty Clay with Sand and Gravel	Undrained (Phi=0)	115		8,000

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SLOPE STABILITY ANLAYSIS
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FIG. 5
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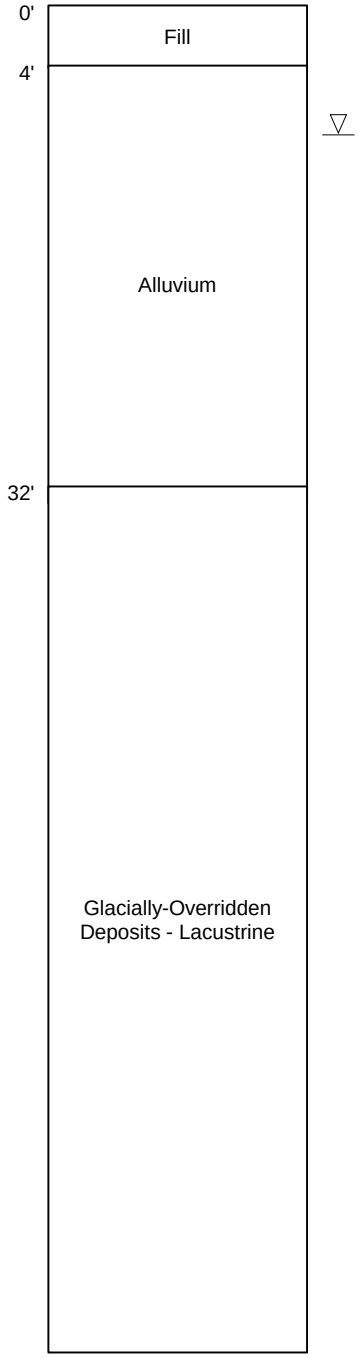
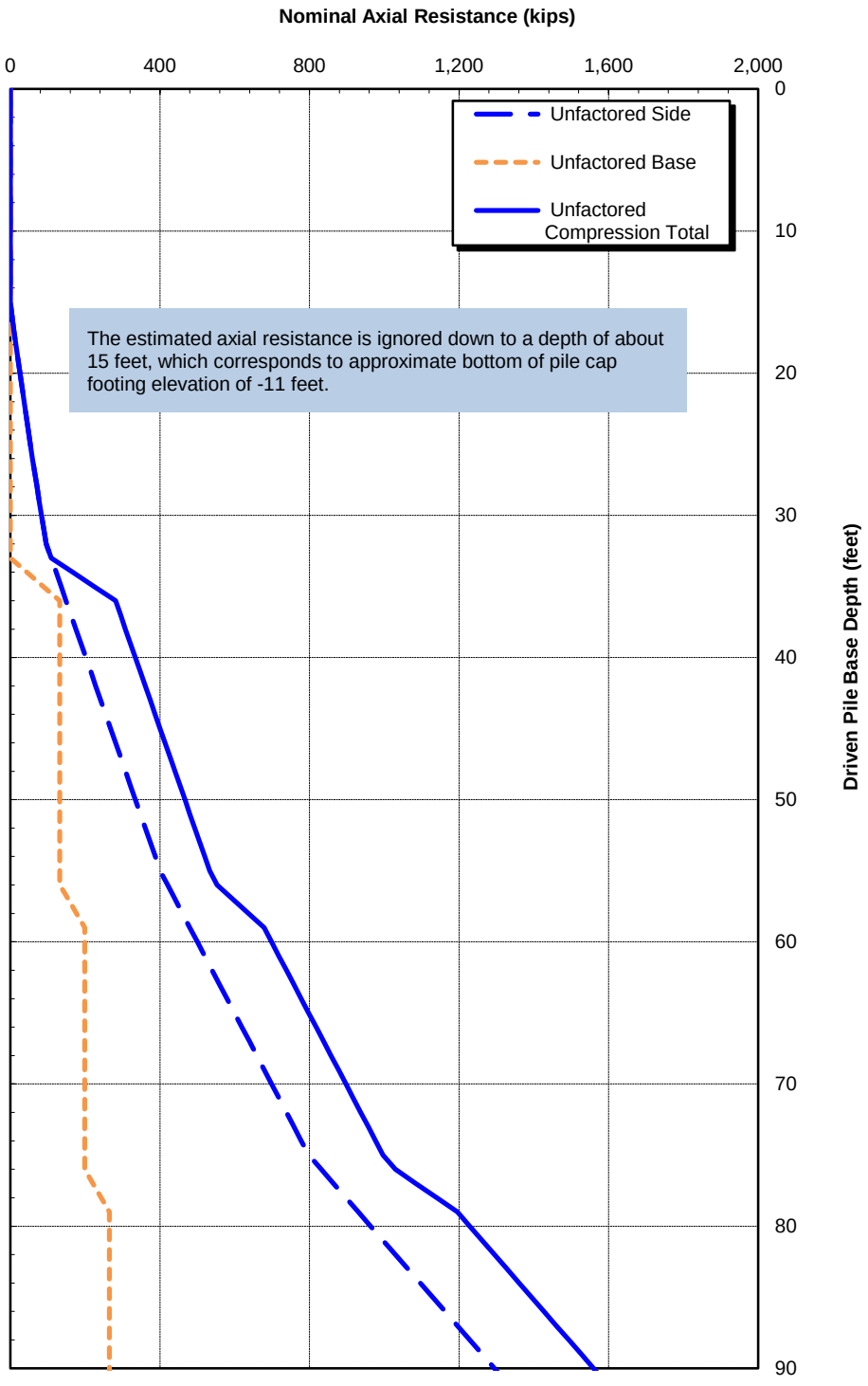
A:\E-Mail\Attachments\105492 SDOT West Seattle High-Rise Bridge\Proposal\Analysis\Slope Stability\Pier 17_082520.gsz

ASSUMED SUBSURFACE PROFILE

Based on nearby explorations:
WS-2
(Reference Grade Elev. = 4 ft.)





NOTES

1. The analyses were performed for a single shaft based on guidelines included in AASHTO, and our experience.
2. The factored total shaft resistance is determined by adding the unfactored side and base resistances, multiplied by appropriate resistance factors per AASHTO.

West Seattle High Level Bridge
Seattle, Washington

ESTIMATED AXIAL RESISTANCE OF 24-IN DRIVEN CONCRETE PILE PIER 15

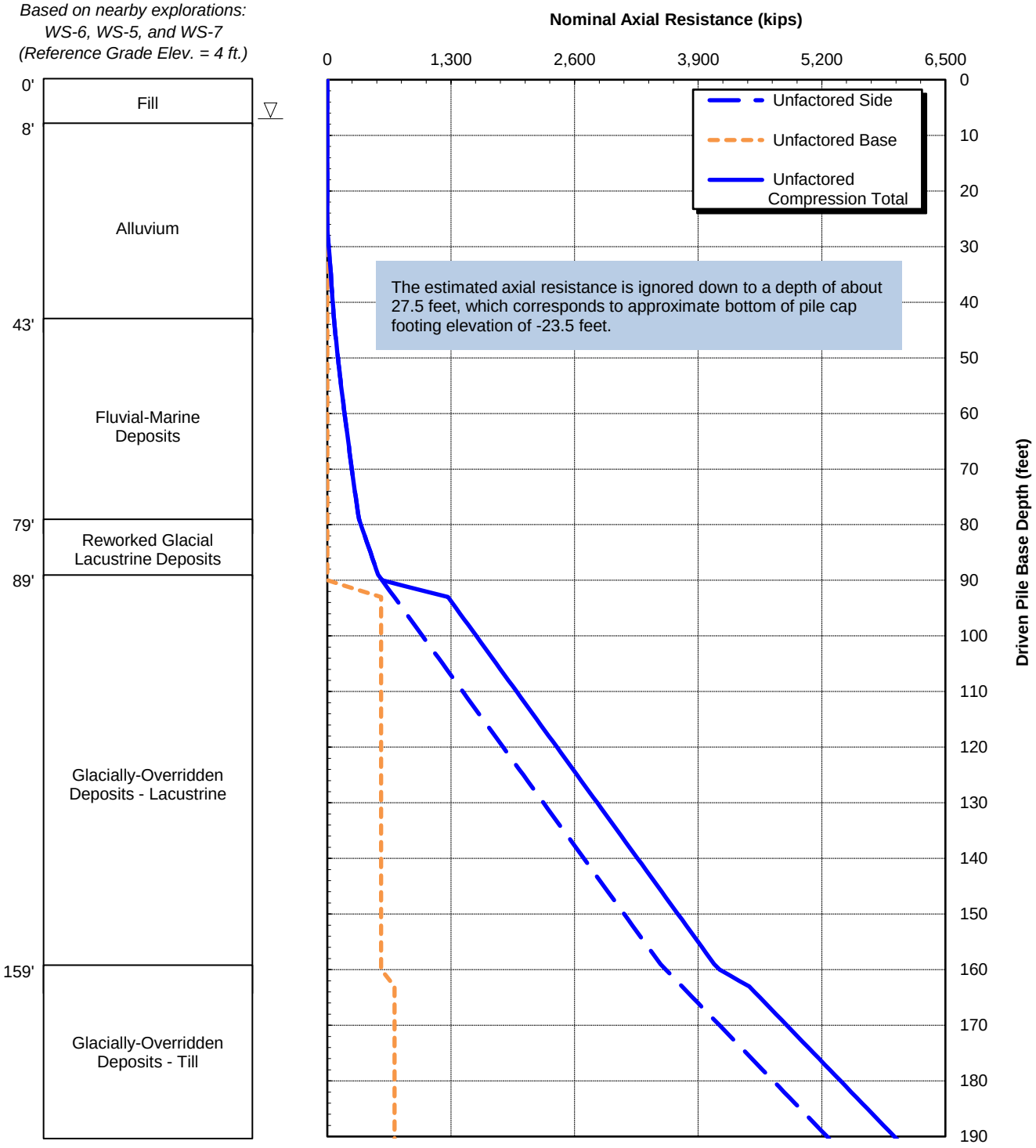
September 2020 105492-001

SHANNON & WILSON

FIG. 6

ASSUMED SUBSURFACE PROFILE

Based on nearby explorations: WS-6, WS-5, and WS-7 (Reference Grade Elev. = 4 ft.)



NOTES

1. The analyses were performed for a single pile based on guidelines included in AASHTO, and our experience.
2. The factored total shaft resistance is determined by adding the unfactored side and base resistances, multiplied by appropriate resistance factors per AASHTO.

West Seattle High Level Bridge
Seattle, Washington

ESTIMATED AXIAL RESISTANCE OF 36-IN DRIVEN STEEL PIPE PILE PIER 16

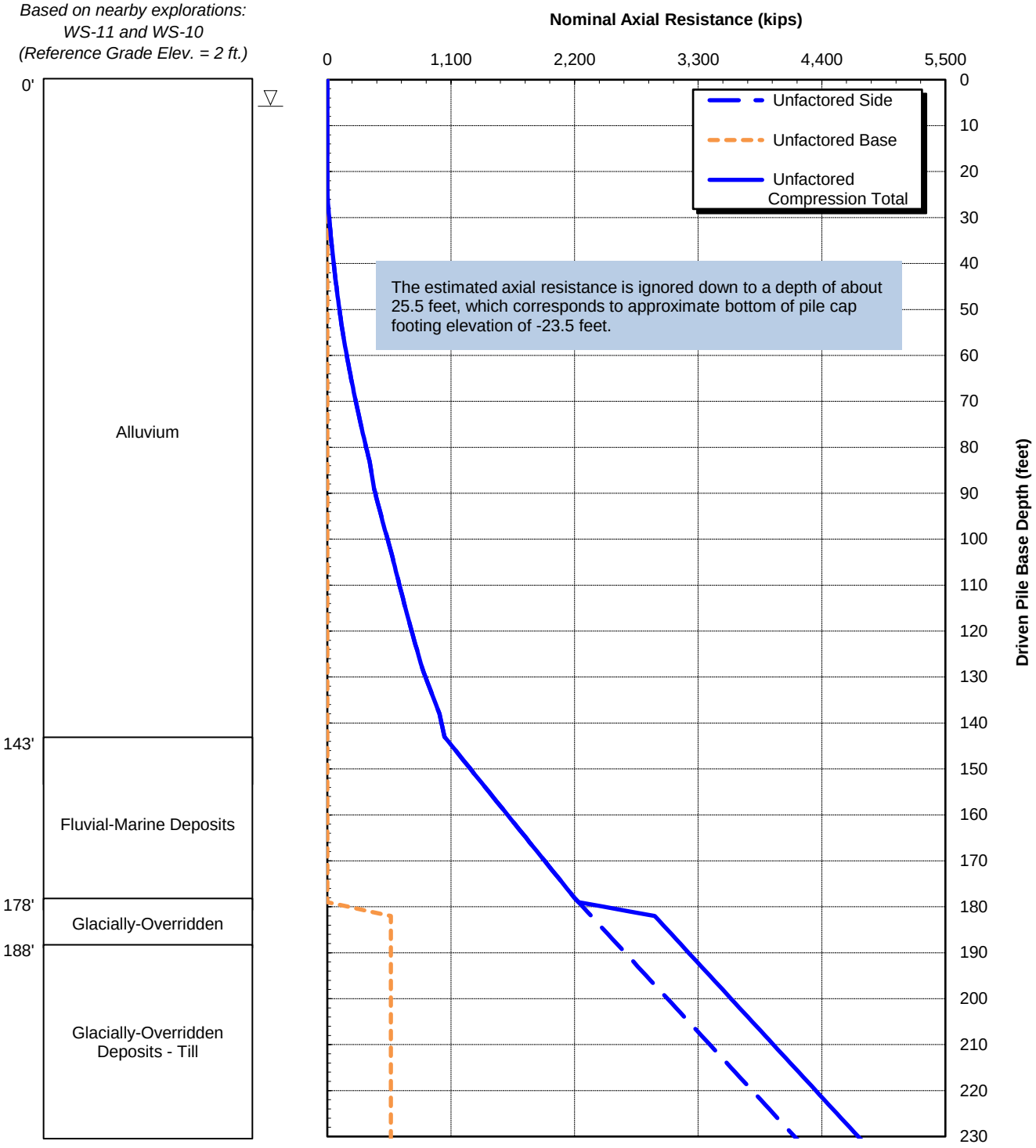
September 2020 105492-001

SHANNON & WILSON

FIG. 7

ASSUMED SUBSURFACE PROFILE

Based on nearby explorations:
WS-11 and WS-10
(Reference Grade Elev. = 2 ft.)



NOTES

1. The analyses were performed for a single pile based on guidelines included in AASHTO, and our experience.
2. The factored total shaft resistance is determined by adding the unfactored side and base resistances, multiplied by appropriate resistance factors per AASHTO.

West Seattle High Level Bridge
Seattle, Washington

ESTIMATED AXIAL RESISTANCE OF 36-IN DRIVEN STEEL PIPE PILE PIER 17

September 2020 105492-001

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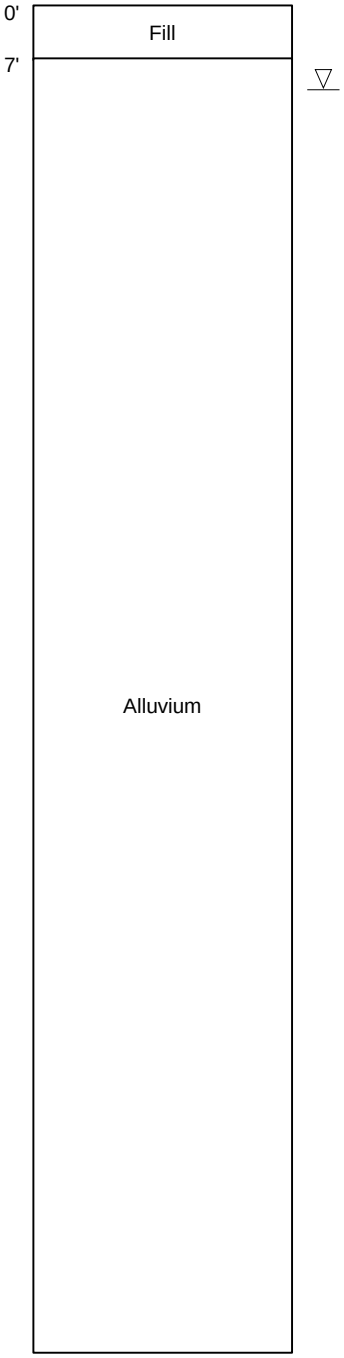
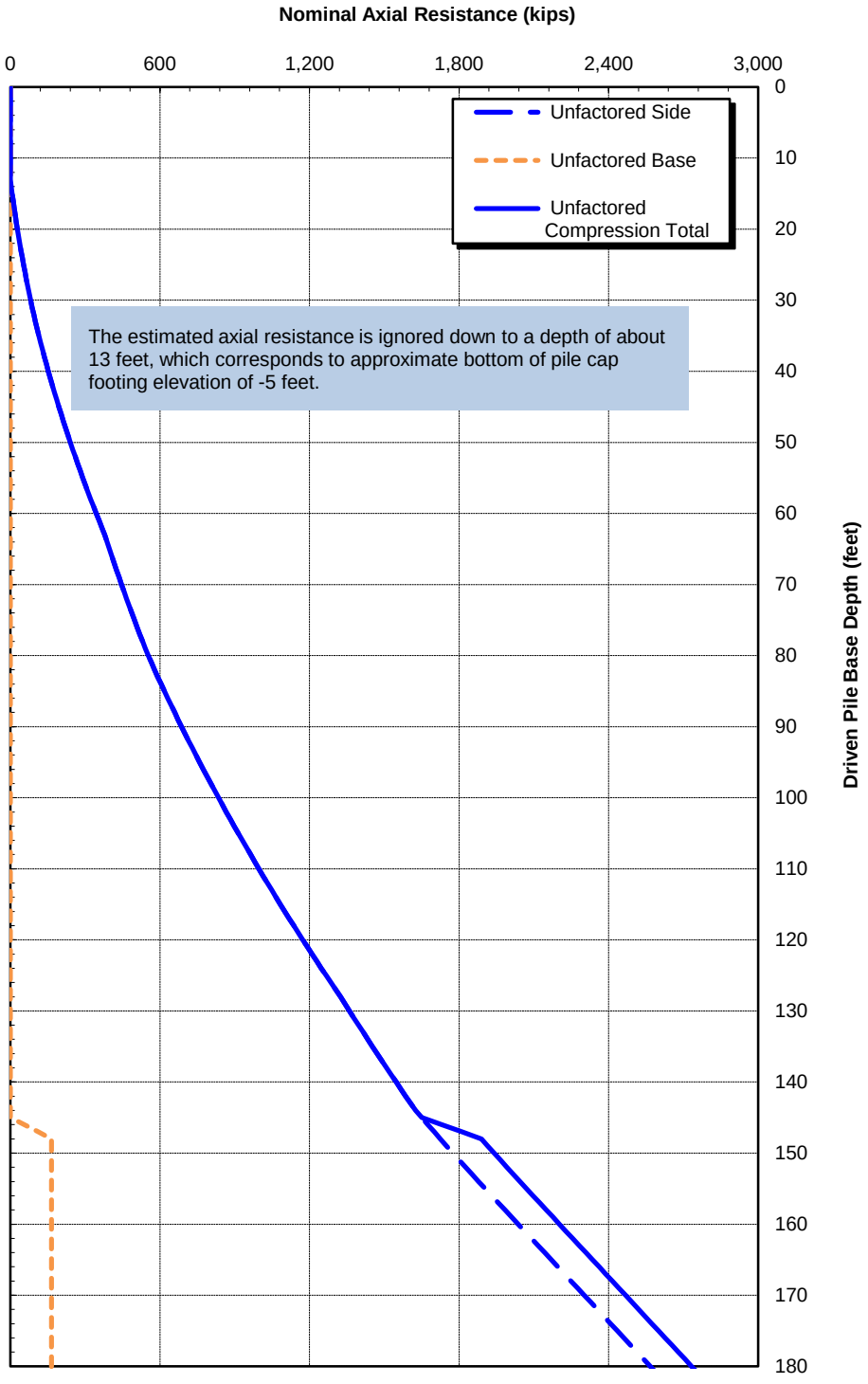
FIG. 8

ASSUMED SUBSURFACE PROFILE

Based on nearby explorations:
WS-15
(Reference Grade Elev. = 8 ft.)





NOTES

1. The analyses were performed for a single shaft based on guidelines included in AASHTO, and our experience.
2. The factored total shaft resistance is determined by adding the unfactored side and base resistances, multiplied by appropriate resistance factors per AASHTO.

West Seattle High Level Bridge
Seattle, Washington

ESTIMATED AXIAL RESISTANCE OF 24-IN DRIVEN CONCRETE PILE PIER 18

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FIG. 9

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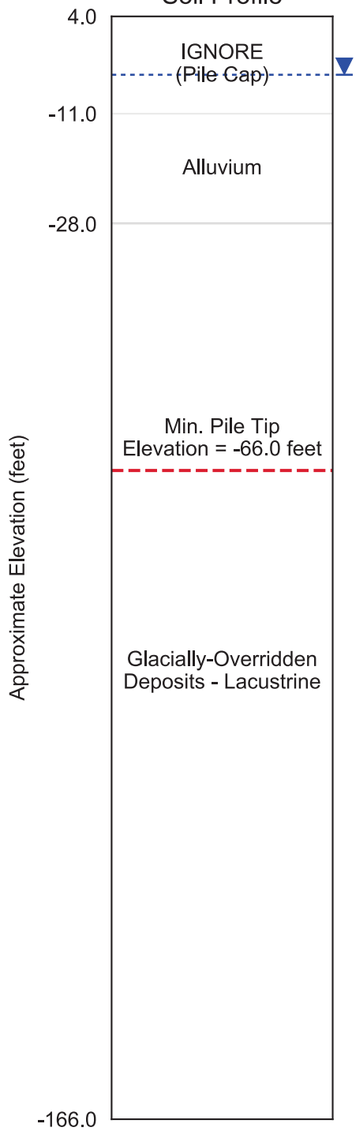
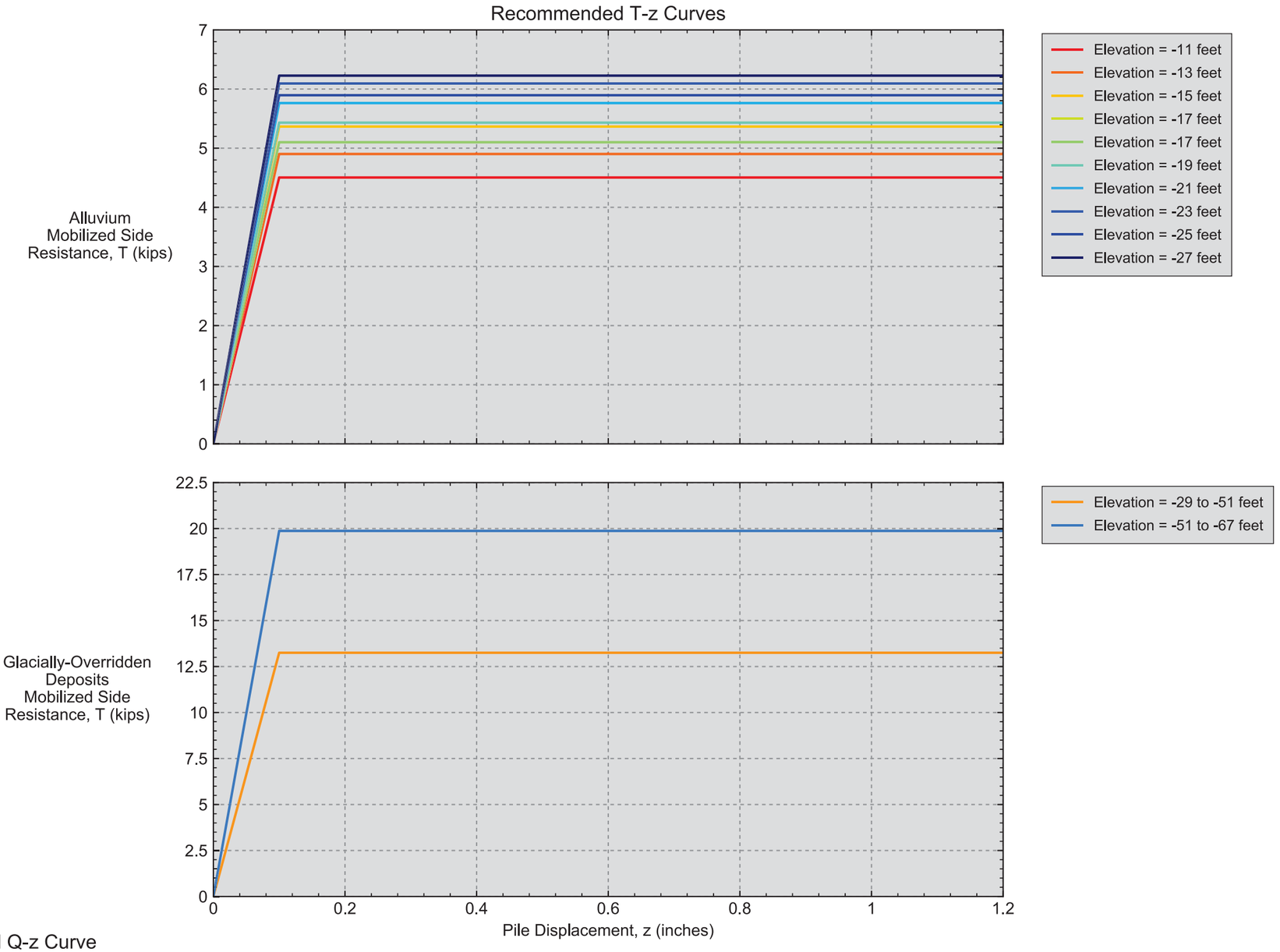
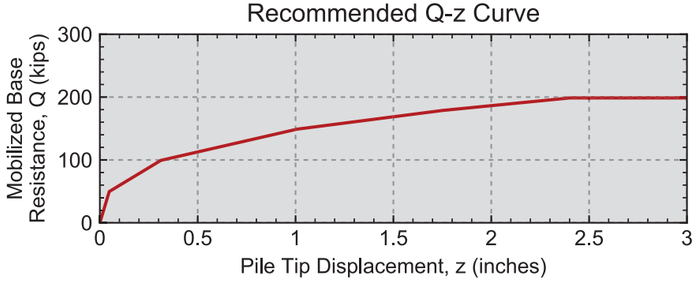
Generalized Soil Profile



Recommended T-z Curves



Recommended Q-z Curve



Notes:

1. The bottom of pile cap and minimum pile tip elevations are based on information included as part of scanned historical reports and as-builts.
2. Our recommended T-z curves are developed based on the Driven Pile Sand T-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019).
3. Our recommended Q-z curves are developed based on the Driven Pile Sand and Clay Q-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019).
4. The nominal unit side and base resistances used to develop the T-z and Q-z curves are based on the results of test piles B-LTP, C-LTP, and D-LTP, published correlations, and our experience in similar soil and project conditions.
5. The generalized soil profile is based on the subsurface conditions encountered in nearby boring WS-2.

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RECOMMENDED DRIVEN PILE T-Z AND Q-Z CURVES PIER 15

October 2020 105492-001

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FIG. 10

I:\EIS\EA\105000s\105492-SDOT_West Seattle High-Rise Bridge\Proposal\Analysis\Axial Capacity\T-z and Q-z Curves

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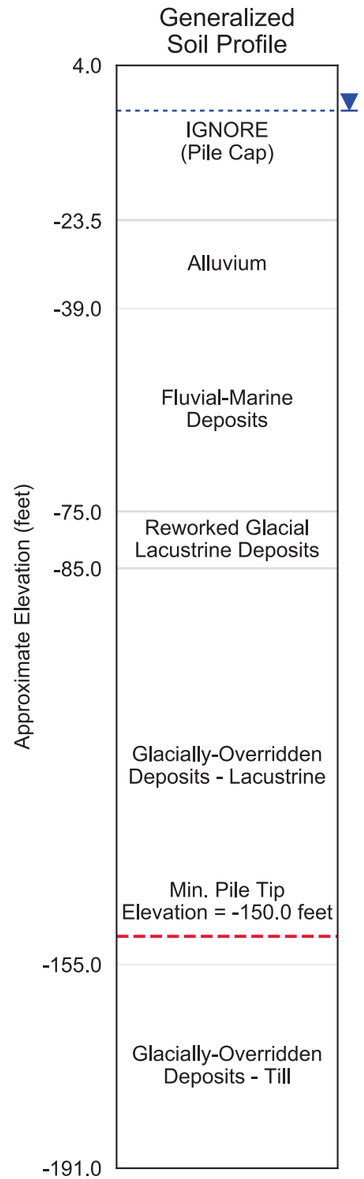

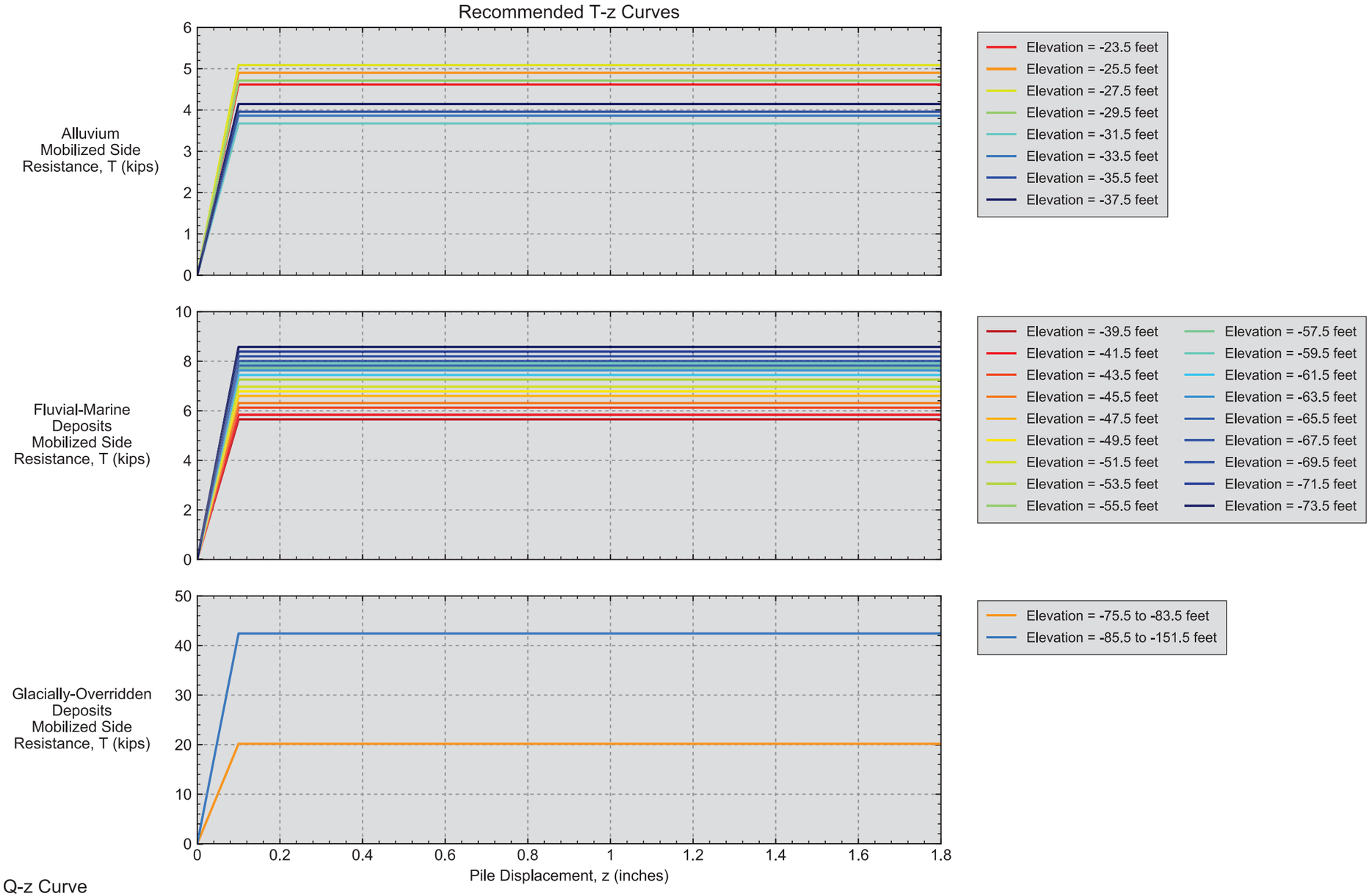

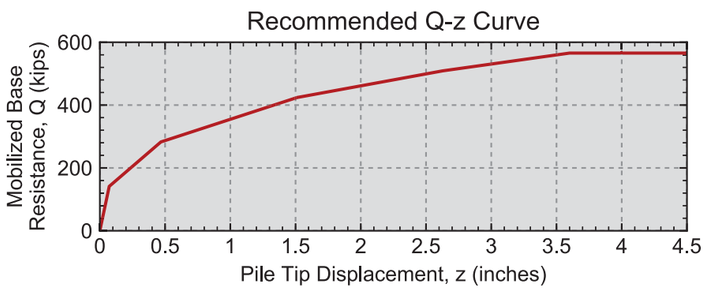

Notes:
1. The bottom of pile cap and minimum pile tip elevations are based on information included as part of scanned historical reports and as-builts.
2. Our recommended T-z curves are developed based on the Driven Pile Sand T-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019).
3. Our recommended Q-z curves are developed based on the Driven Pile Sand and Clay Q-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019).
4. The nominal unit side and base resistances used to develop the T-z and Q-z curves are based on the results of test piles C-LTP and D-LTP, published correlations, and our experience in similar soil and project conditions.
5. The generalized soil profile is based on the subsurface conditions encountered in nearby borings WS-6, WS-5, and WS-7.

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Seattle, Washington

RECOMMENDED DRIVEN PILE T-Z AND Q-Z CURVES PIER 16

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FIG. 11

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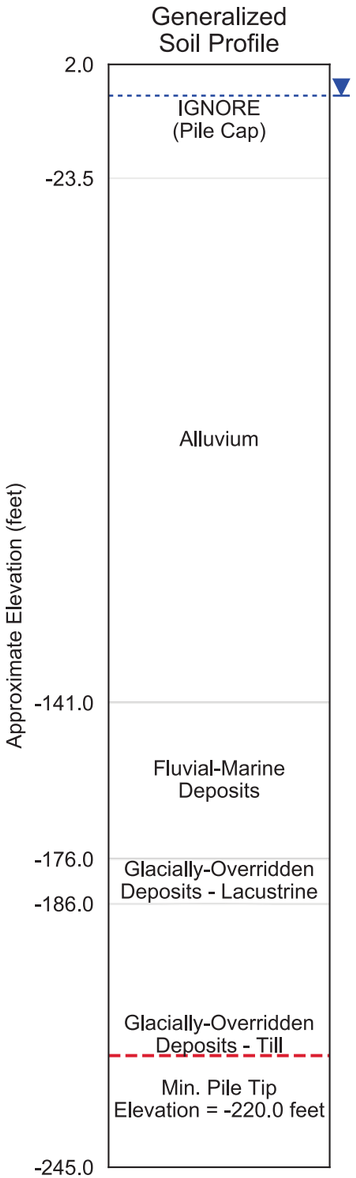

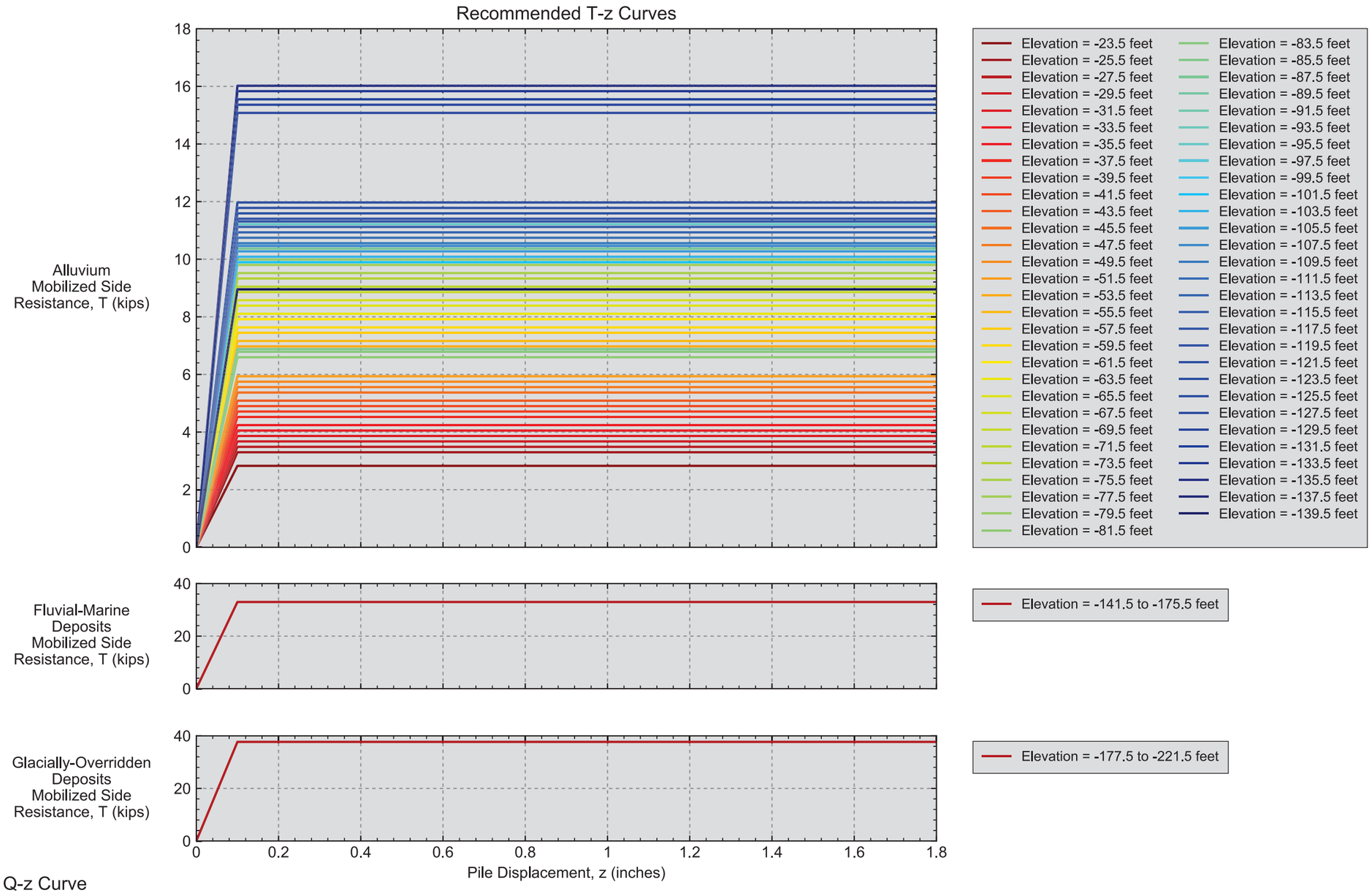

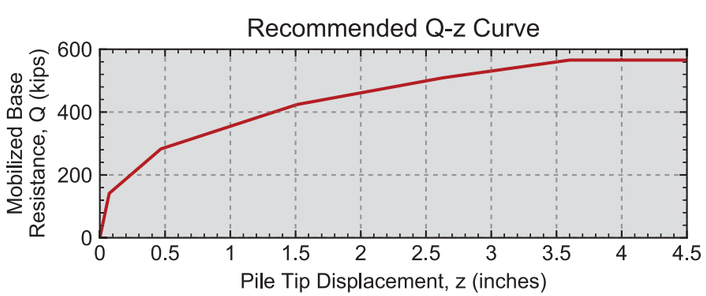

Notes:
1. The bottom of pile cap and minimum pile tip elevations are based on information included as part of scanned historical reports and as-builts.
2. Our recommended T-z curves are developed based on the Driven Pile Sand T-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019).
3. Our recommended Q-z curves are developed based on the Driven Pile Sand and Clay Q-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019).
4. The nominal unit side and base resistances used to develop the T-z and Q-z curves are based on the results of test piles C-LTP and D-LTP, published correlations, and our experience in similar soil and project conditions.
5. The generalized soil profile is based on the subsurface conditions encountered in nearby borings WS-11 and WS-10.

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RECOMMENDED DRIVEN PILE T-Z AND Q-Z CURVES PIER 17

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FIG. 12

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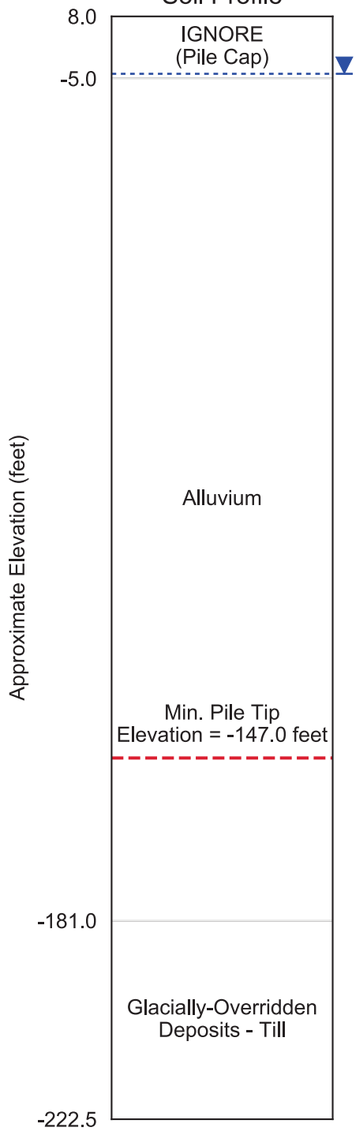
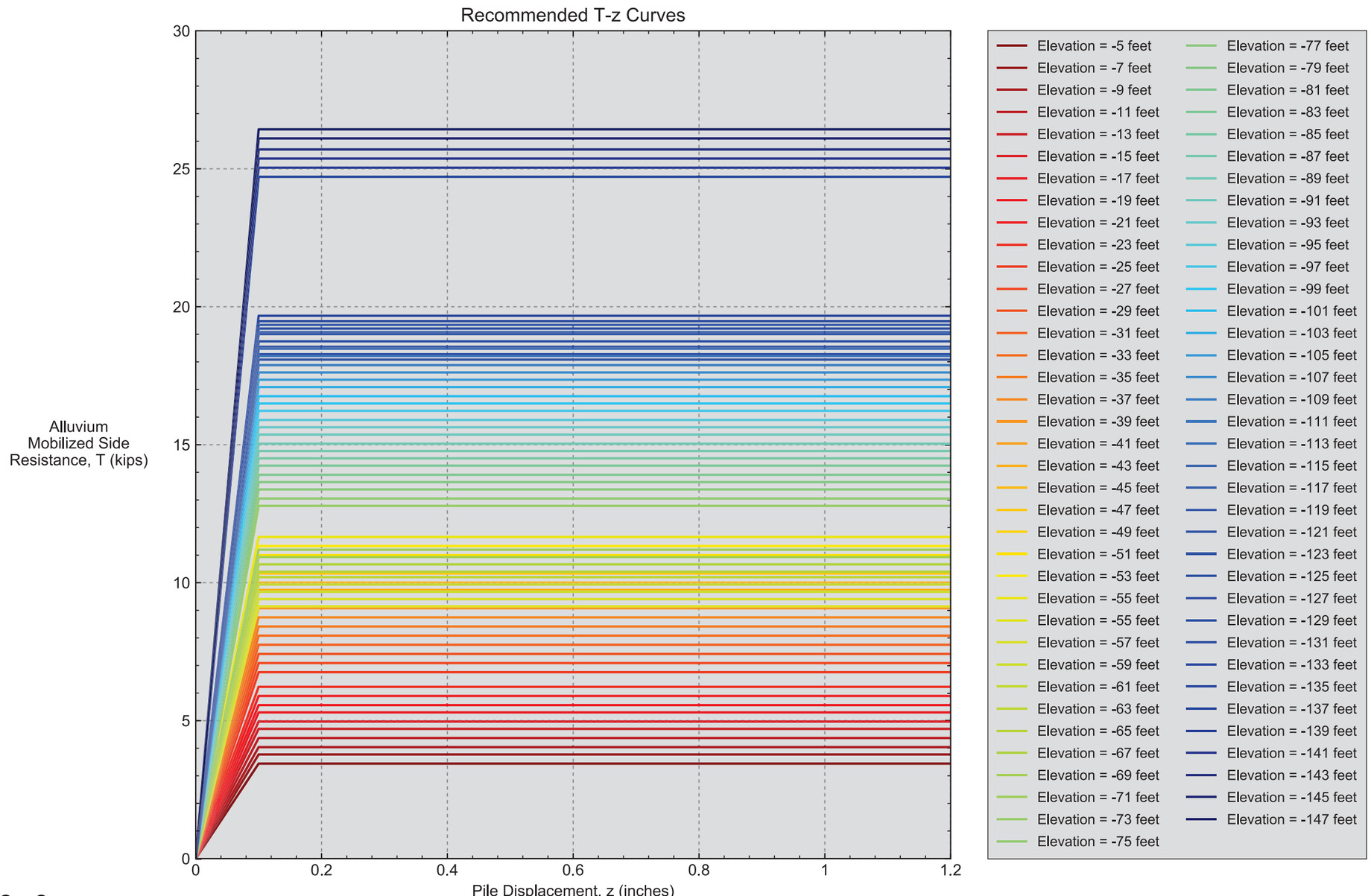
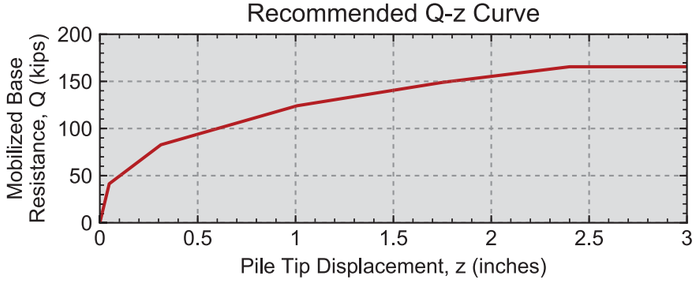
Generalized Soil Profile



Recommended T-z Curves



Recommended Q-z Curve



Notes:

1. The bottom of pile cap and minimum pile tip elevations are based on information included as part of scanned historical reports and as-builts.
2. Our recommended T-z curves are developed based on the Driven Pile Sand T-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019).
3. Our recommended Q-z curves are developed based on the Driven Pile Sand and Clay Q-z curve model recommended in the American Petroleum Institute Publication: RP 2A-LRFD (2nd edition, 2019).
4. The nominal unit side and base resistances used to develop the T-z and Q-z curves are based on the results of test piles B-LTP, C-LTP, and D-LTP, published correlations, and our experience in similar soil and project conditions.
5. The generalized soil profile is based on the subsurface conditions encountered in nearby boring WS-15.

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FIG. 13

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APPENDIX A: SITE-SPECIFIC SITE RESPONSE ANALYSIS

Appendix A

Site-Specific Site Response Analysis

West Seattle High Level Bridge

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APPENDIX A: SITE-SPECIFIC SITE RESPONSE ANALYSIS

A.1 INTRODUCTION

This appendix details the procedure we used to perform the site-specific site response analysis for the West Seattle High Level Bridge project. Our analysis followed the requirements of the American Association of State Highway and Transportation Officials (AASHTO) LRFD bridge design specifications (AASHTO, 2020) and the Washington State Department of Transportation (WSDOT) Geotechnical Design Manual (GDM, 2019) for the site response analysis. These codes are referred to herein as AASHTO and the GDM, respectively.

Based on historical subsurface exploration and laboratory testing program the seismic site condition is preliminarily characterized as Site Class D. The Site Class D condition corresponds to a time-averaged shear wave velocity for the 100 feet below ground surface (V_{S30}) of between 600 feet per second (fps) and 1,200 fps. Our Standard Penetration Test (SPT) -based liquefaction hazard analyses indicate that the subsurface conditions at the site include liquefiable soils. We performed a site-specific site response analysis for the project because:

- a site-specific site response analysis provides a more accurate evaluation of the design ground motions. For some site conditions, the site-specific evaluation can reduce the seismic design demand as compared to standard code-based response spectrum, and
- to provide a more detailed evaluation of the liquefied-induced effects (e.g., liquefaction-induced settlement) considering variation and extension depth of the liquefied soil along the bridge.

We performed a one-dimensional (1D) site response analysis to evaluate the design ground motions and seismic hazard at the site. The 1D site response analysis models a horizontally layered column of soil. An input motion is applied at the base of the column and the vertical wave propagation is computed based on models that approximate the soil's response to cyclic loading. As such, the inputs for our site-specific site response analysis include the input base motions and the soil model types and input parameters. The following sections describe the development of the input base motions, the input parameters, and results of our site-specific site response analysis. The effects of liquefaction (such as loss of shear strength, reduction in pile resistance, settlement, and lateral spreading) and our corresponding liquefaction related design recommendations are discussed in the main text of the report.

A.2 INPUT BASE MOTIONS

We developed our input base motion time histories based on the guidance provided in AASHTO and the GDM. To develop the input base motions for our site-specific site response analysis we:

- Evaluated the site conditions at the base of the model,
- Developed a Safety Evaluation Earthquake (SEE) and Functional Evaluation Earthquake (FEE) target response spectra that are consistent with the site conditions at the base of our model, and
- Selected one suite of the time histories for each of the SEE and FEE ground motions that were consistent with the seismic hazard at our site and scaled and then spectrally matched them so that the response spectrum of our selected motions suite was, on average, consistent with the SEE and FEE target design spectra.

Each of these tasks are described in the following sections.

A.2.1 Base Site Conditions

Shear wave velocity measurements were not performed as part of this study. The shear wave velocity measurement tests were conducted as part of the 1984 geophysical subsurface investigation program (Shannon & Wilson, 1984). Two crosshole and two downhole tests were identified at the east and west banks of the Duwamish west waterway, west of the existing West Seattle High Level Bridge. The crosshole shear wave velocity measurements included borings WSB-2 (near Pier 17) and WSB-4 (near Pier 16) and downhole shear wave velocity measurements included borings W-4 (near Pier 17) and W-7 (near Pier 16). Based on our evaluation of the available shear wave velocity data, the site conditions at the base of the profile corresponds to Site Class C. Per AASHTO, the Site Class C conditions corresponds to a V_{S30} of 1,200 fps to 2,500 fps. We evaluated the base motion response spectra for SEE and FEE using the average V_{S30} for Site Class C conditions of approximately 1,850 fps).

A.2.2 Seismic Design Spectra for Base of Soil Profile

Following the GDM seismic design guidelines, the design SEE and FEE spectra for base of the soil profile were developed from probabilistic 5% damping mean uniform hazard spectrum for return periods of 975- and 210-year, respectively. These return periods are approximately equivalent to 7% and 30% probability of exceedance in 75 years for SEE and FEE spectra, respectively. Deterministic hazards were not considered in our analyses. We obtained spectral acceleration values consistent with Site Class C conditions from the

existing, publicly available, U.S. Geological Survey (USGS) Probabilistic Seismic Hazard Analysis (PSHA) data. We used the USGS 2018 National Seismic Hazard Model (NSHM) (Petersen and others, 2020) to obtain the SEE and FEE spectral acceleration values. The spectral values were calculated from the USGS 2018 NSHM hazard curves available at https://www.sciencebase.gov/catalog/item/5cbf47c4e4b0c3b00664fdef for site location at 47.5709°N and 122.3503°W.

The bridge is located within Puget Lowland basin area which was considered in the USGS 2018 NSHM. The basin can amplify ground motions at large periods, typically greater than 1.0 second. The USGS 2018 NSHM site parameters that are used to evaluate basin effects include the depth to 2.5 kilometers (km) per second shear wave velocity ($Z_{2.5}$) and the effective depth to 1.0 km per second shear wave velocity consistent with the $Z_{2.5}$ ($Z_{1.0, eff}$). Using Stephenson and others (2017) community velocity model, we estimated the value of $Z_{2.5}$ at the project location to be approximately 5.9 km. The corresponding $Z_{1.0, eff}$ value was also estimated 0.82 km from two equally weighed $Z_{1.0}$-$Z_{2.5}$ regressions used in the USGS 2018 NSHM.

The ground motion models used in the USGS 2018 NSHM does not include the Puget Lowland basin effects for periods less than 0.75 second but already includes the basin effects for periods greater than 0.75 second, so no additional measures were needed to include the basin effects on the spectral values obtained from the USGS 2018 NSHM.

The SEE and FEE base motion acceleration spectra are shown in Figure A-1. These spectra served as the target spectra in our development of the base input time histories described in the following sections.

A.2.3 Development of Time Histories

We developed two suites of base input time histories for our site response analyses. We developed a suite of seven base input time histories for each of SEE and FEE design levels. The motions were selected from recorded earthquake records from events that were considered representative of the seismic hazards and source characteristics of the project site. The recorded motions were then scaled so that the motion suite produced a response spectrum that was, on average, consistent with the target SEE or FEE spectra that were obtained previously. Finally, to lower variability of the structural response to individual time histories, each time history was spectrally matched to the corresponding target spectrum. The following sections describe our time history development procedure including the seismic source characteristics of the site, time history selection, time history scaling, and time history spectral matching.

A.2.3.1 Distribution of Time Histories in Suite of Ground Motions

To evaluate the seismic hazard and source characteristics at the site, we evaluated the deaggregation data from the USGS hazard analysis. However, the USGS 2018 NSHM deaggregation data was not publicly available at the time of this report. We note that in Western Washington the main difference between the updated USGS 2018 NSHM and the USGS 2014 NHSM was the inclusion of basin effects. Therefore, we used the USGS 2014 NSHMP deaggregation as an approximation for USGS 2018 NSHM deaggregation data and the 2014 data was used to evaluate the seismic hazard and source characteristics at the site.

We used the deaggregation data to evaluate the number of the time histories needed for each seismic source to include in the base input time history suite. The summary of the seismic hazard contributions for the base SEE and FEE design are tabulated in Exhibits A-1 and A-2, respectively.

Exhibit A-1: SEE Hazard Contribution (in Percent) for Seismic Sources

Period (seconds)	Crustal	SFZ Only	CSZ Interface	CSZ Intraslab
0	46	24	12	42
0.1	43	21	7	50
0.2	44	22	10	46
0.3	48	25	13	39
0.5	54	28	18	28
0.75	53	28	25	22
1	52	28	31	17
2	43	25	48	9
3	39	24	56	5
4	36	22	60	4
5	35	21	62	3

Notes:
CSZ = Cascadia subduction zone; SFZ = Seattle fault zone

Exhibit A-2: FEE Hazard Contribution (in Percent) for Seismic Sources

Period (seconds)	Crustal	SFZ Only	CSZ Interface	CSZ Intraslab
0	39	10	11	50
0.1	38	10	7	55
0.2	38	10	9	53
0.3	40	10	12	48
0.5	44	11	16	40
0.75	44	11	21	35
1	44	11	24	32
2	43	11	32	25
3	42	11	36	22
4	42	11	38	20
5	43	11	39	18

Note that the hazard contribution from each of the seismic sources vary with the period. AASHTO defines the period range of interest for the structural response as 50% to 200% of the fundamental period of the structure. Based on the hazard contributions provided in Exhibits A-1 and A-2 and considering the proximity of the Seattle fault zone (SFZ) to the project site, the seven time histories in the suite for each of the SEE and FEE design were distributed among the seismic sources as shown in Exhibit A-3.

Exhibit A-3: Number of Time Histories Selected for Seismic Sources

Design Level	Crustal	CSZ Interface	CSZ Intraslab
SEE	4	2	1
FEE	3	2	2

A.2.3.2 Selection of Time Histories

Candidate reference time histories (i.e., seed motions) were selected to be consistent with the magnitude, distance, site conditions, acceleration response spectrum shape, and tectonic regime of the seismic sources that were identified by the deaggregation data.

To select the crustal time histories, we reviewed the Pacific Earthquake Engineering Research (PEER) center Next Generation Ground Motion Attenuation Models for the Western United States (NGA-West2) (Bozorgnia and others, 2014) dataset for candidate earthquake time histories. The NGA-West2 dataset includes significant number of

previously processed (filtered and/or baseline corrected) records from crustal events. In general, the NGA-West2 motions require no additional processing.

Unlike crustal events, there are relatively few recorded large-magnitude subduction interface or medium-magnitude subduction intraslab earthquakes. There are no recorded CSZ interface earthquakes in modern history, and our time history selection was limited to areas with similar active subduction zones (e.g., South America, and Japan). For CSZ interface and intraslab events, we reviewed the South American, the Center for Engineering Strong Motion Data, the National Strong Motion Project, and K-NET (Kyoshin network) and KiK-net (Kiban Kyoshin network) datasets to select appropriate time histories. Some of the time histories in these datasets are either uncorrected (raw data) or often corrected such that it might affect the time histories in period range of interest. Therefore, we reviewed each of the selected time histories and processed (filtered and/or baseline corrected) each of the selected subduction zone seed time histories as needed.

In our selection process, the overall shape of the response spectrum and its relative closeness to the target spectrum in the period range of interest was given a priority. The time histories selected to represent the hazard from the CSZ interface were all limited to only two large-magnitude events, Moment magnitude (M_W) 9.0 Tohoku (2011) and M_W 8.8 Maule (2010), due to the lack of available records for similar, large magnitude subduction zone type of the events.

The selected time histories for SEE and FEE design are tabulated in Exhibits A-4 and A-5, respectively.

Exhibit A-4: Selected SEE Time Histories for Site Response Analysis

Mechanism	Crustal Reverse Oblique	Crustal Reverse Oblique	Crustal Reverse	Crustal Reverse	Subduction Interface	Subduction Interface	Subduction Intraslab
Earthquake	Loma Prieta	Loma Prieta	Cape Mendocino	Iwate, Japan	Maule, Chile	Tohoku, Japan	Nisqually, Washington
Earthquake Date	1989	1989	1992	2008	2010	2011	2001
Station Name	Anderson Dam (DS)	Saratoga - Aloha Ave	Loleta Fire Station	MYGH02	Cien Agronomicas, UC, La Plantina	SAWARA	Halverson Res.
MW	6.93	6.93	7.01	6.9	8.8	9	6.8
VS30 (m/sec)	489	381	516	399	-	-	-
Repi (km)	-	-	-	-	362	322	19
Rhyp (km)	-	-	-	-	363	322	57
Rjb (km)	20	8	23	5	106	21	17
Rrup (km)	20	9	26	11	117	52	49
File Name	RSN739_LOMAP_AND250	RSN802_LOMAP_STG090	RSN3750_CAPEMEND_LFS360	RSN5678_IWATE_MYGH02NS	Maule-ANTU360	Tohoku-CHB004NS	Nisqually-7008-270

Notes:

Hz = hertz; m/sec = meters per second; R_{epi} = distance to epicenter; R_{hyp} = hypocentral distance; R_{jb} – Joyner and Boore distance; R_{rup} = source-to-site rupture distance

A.2.3.3 Scaling of Time Histories

The selected time histories in Exhibits A-4 and A-5 were scaled to provide a reasonable agreement with the target SEE or FEE response acceleration spectra, respectively. We used an optimization procedure that minimized the difference between the time history acceleration response spectrum and target acceleration spectrum in equally spaced (in log scale) period intervals to calculate the scale factor for each time history.

Figures A-2 and A-3 show the response spectra of the selected time histories scaled to the target SEE and FEE spectra, respectively. As observed from these figures, the geometric mean of the seven scaled response spectra of the selected time histories approximately follows the corresponding target spectrum for both SEE and FEE design events.

Exhibit A-5: Selected FEE Time Histories for Site Response Analysis

Mechanism	Crustal Reverse Oblique	Crustal Reverse	Crustal Reverse	Subduction Interface	Subduction Interface	Subduction Intraslab	Subduction Intraslab
Earthquake	Loma Prieta	Cape Mendocino	Iwate, Japan	Maule, Chile	Tohoku, Japan	Nisqually, Washington	Olympia, Washington
Earthquake Date	1989	1992	2008	2010	2011	2001	1949
Station Name	Anderson Dam (DS)	Loleta Fire Station	MYGH02	Municip. San Jose de Maipo	SAWARA	Halverson Res.	Olympia Hwy Test Lab
MW	6.93	7.01	6.9	8.8	9	6.8	6.9
VS30 (m/sec)	489	516	399	-	-	-	-
Repi (km)	-	-	-	371	322	19	19
Rhyp (km)	-	-	-	373	322	57	60
Rjb (km)	20	23	5	125	21	17	1
Rrup (km)	20	26	11	135	52	49	48
File Name	RSN739_LOMAP_AND250	RSN3750_CAPEMEND_LFS360	RSN5678_IWATE_MYGH02NS	Maule-SJCH90	Tohoku-CHB004NS	Nisqually-7008-270	WWA-103l56ol-356

A.2.3.4 Spectral Matching of Time Histories

Scaling a time history does not change the shape of the response spectrum and is not always adequate to match the target spectrum. We spectrally matched each scaled time history to its corresponding SEE or FEE target spectrum. We used a spectral matching procedure to modify the scaled time histories to have their response spectrum close to the target spectrum. The spectral matching was performed for the period range of 0.01 to 10 seconds using the RSPM09 code (Al-Atik and Abrahamson, 2010) by adding appropriate wavelets to the scaled reference or seed time history. We evaluated the spectrally matched time histories and filtered and/or baseline corrected to remove displacement distortion imposed during the matching process.

The time history plots are provided in Figures A-4 through A-10 for SEE reference time histories, Figures A-11 through A-17 for SEE spectrally matched time histories, Figures A-18 through A-24 for FEE reference time histories, and Figures A-25 through A-31 for FEE spectrally matched time histories. Each plot includes acceleration, velocity, and displacement time histories. The response spectrum of the time histories is also compared to the target spectrum. A Husid plot, which is the Arias Intensity (the normalized sum of

squared ground acceleration values over time) variation, is also plotted in these figures. The name of the ground motion for each time history in the title of the figure includes recording station and orientation of the motion. The direction of the horizontal component is identified by azimuth in degrees (e.g., 270° or 360°), east-west (i.e., EW), or north-south (i.e., NS) directions.

A.3 SITE RESPONSE ANALYSIS

To evaluate the variability in the subsurface conditions at the site, we performed 1D site response analysis on two separate soil columns to estimate the seismic performance of the site and to develop a design ground motion response spectrum for the FEE and SEE design levels. The two soil columns were evaluated based on the conditions encountered in borings performed near Piers 15 and 16 and Piers 17 and 18, respectively. The Piers 15 and 16 soil column was evaluated based on existing Borings WSB-4 and WS-4 through WS-8. The Piers 17 and 18 soil column was evaluated based on existing Borings WSB-2 and WS-9 through 13. These explorations included shear wave velocity testing and based on the available subsurface and laboratory test data were considered representative of the subsurface conditions at the site. Our site response analysis was performed using nonlinear analyses implemented in the finite difference program FLAC (Itasca, 2018). This section describes the soil profiles, constitutive models, input parameters, our FLAC implementation procedures, and a summary of the results of our analyses. A detailed discussion of the development of our recommended design ground motion response spectrum is provided in Section A.4.

A.3.1 Soil Constitutive Model Parameters

We evaluated the soil constitutive models and input parameters for our site-specific site response analysis based on the soil conditions described in the existing subsurface explorations, the existing laboratory test data, and our experience. Soils with similar dynamic characteristics (i.e. shear strength, shear stiffness, and modulus reduction and damping behavior) were grouped together into dynamic soil units. In general, the dynamic soil units correspond to the units described in the main report and consisted of the following:

- Fill – clean to slightly silty sand
- Alluvium – clean to silty sand with scattered layers of sandy to clayey silt,
- Fluvial-Marine Deposits – silty sand to sandy silt with scattered areas of silty clay, and

- Glacial Deposits – consisting of glacially overridden sediments of variable composition including Glacial Lacustrine and Glacial Till.

Due to the presence of liquefiable soils, we performed effective stress site response analysis that accounts for the generation of excess pore pressure and cyclic softening in liquefiable soils. For predominantly sandy soils that we considered susceptible to liquefaction, we used the user-defined constitutive model PM4SAND (Boulanger and Ziotopoulou, 2015). For soils that were not considered susceptible to liquefaction and pore pressure-induced softening, we used the Mohr-Coulomb material model provided by FLAC.

The PM4SAND and Mohr-Coulomb constitutive models implemented in our FLAC analyses all require an estimate of the initial shear modulus, G_{max}. The following section describes our procedure to estimate G_{max} for each of the soil profiles considered in our analysis. Subsequent sections detail the input parameters unique to the PM4SAND and Mohr-Coulomb constitutive models.

A.3.1.1 Initial Shear Modulus Profiles

We evaluated the initial shear modulus for each soil profile using an estimate of the shear wave velocity. The shear wave velocity was evaluated based on the measured shear wave velocity in Borings WSB-4 and WSB-2 for the Pier 15 and 16 and Pier 17 and 18 profiles, respectively. Plots of the shear wave velocity profiles assumed in our analysis are provided in Figures A-32 and A-33 and tabulated in Tables A-1 and A-2 for the analyses performed for the Pier 15 and 16 and Pier 17 and 18 profiles, respectively. As shown in these figures, the assumed shear wave velocity profiles provide a good match to the measured data. The figures and tables mentioned previously also include the assumed soil stratigraphy for the site response analyses performed for each profile.

To be consistent with the assumed shear wave velocity used to develop our base input motions, the base of our site response models corresponded to the depth where material with a shear wave velocity of 1,850 fps was encountered.

A.3.1.2 PM4SAND Input Parameters

Soils that were considered susceptible to liquefaction and/or pore pressure-induced cyclic degradation were evaluated using the PM4SAND constitutive model. The subsurface conditions at each of the considered locations included stratified deposits consisting of sands, silts, and clays of variable composition, consistency, and density. We performed preliminary liquefaction susceptibility analyses of each of the geologic units based on our geologic interpretation and the screening methodologies presented by Boulanger and Idriss

(2006) and Bray and Sancio (2006), which evaluate the susceptibility to liquefaction based on the soil's composition and index properties. Based on these analyses, portions of the Fill, Alluvium, and Fluvial-Marine deposits include sands and low to nonplastic silts that were considered susceptible to liquefaction and pore pressure-induced cyclic degradation.

The soils that were considered susceptible to liquefaction were predominantly sandy soils or nonplastic silts and were evaluated using the PM4SAND constitutive model. The PM4SAND model follows the basic framework of the bounding surface plasticity model presented by Manzari and Dafailias (1997) and Dafalias and Manzari (2004) with modifications to provide better approximations of the cyclic behavior of sands as observed in the field and during laboratory testing. The PM4SAND model consists of three primary input parameters as follows:

- Relative density, Dr: which controls the stress-strain response characteristics of the soil. Per Boulanger and Ziotopoulou (2015), the corrected SPT blow count $(N_1)_{60}$ can be related to relative density using the expression $Dr = [(N_1)_{60} / 46]^{1/2}$.
- Shear modulus coefficient, Go: which is used to compute the small strain shear modulus, G_{max}.
- Contraction rate parameter, hpo: which adjusts the rate at which excess pore pressure is developed and, therefore, when liquefaction is triggered for a given cyclic loading history.

Using single element tests with constant amplitude harmonic loading, we calibrated the PM4SAND input parameter hpo so the model response matched the SPT liquefaction triggering charts in Boulanger and Idriss (2014) for a given value of Dr and Go. The parameter for Go was determined so that the G_{max} value computed internally by PM4SAND matched the G_{max} value computed based on the assumed density and shear wave velocity profiles described previously. The input parameter for Dr was determined using the observed blow counts from our subsurface investigation program and the Dr-$(N_1)_{60}$ relationship provided above.

A.3.1.3 Mohr-Coulomb Material Input Parameters

Soils that were not considered susceptible to liquefaction were modeled using the Mohr-Coulomb material model provided in FLAC. The Mohr-Coulomb model treats the soil as a purely-elastic-purely-plastic material. The model behaves as a linear elastic material at shear stresses less than the shear strength of the soil; if the shear strength of the soil is reached or exceeded, the model behaves as a purely plastic material.

The input properties for the Mohr-Coulomb model, as implemented by FLAC, include mass density, cohesion, friction angle, tension limit, dilation angle, bulk modulus, and shear modulus. The mass density accounts for the mass of the soil; the cohesion, friction angle, tension limit, and dilation angle describe the shear strength of the soil; and the bulk and shear modulus describe the elastic behavior of the soil.

The elastic behavior of the Mohr-Coulomb (when the shear stress is below the strength limit) alone do not account for the strain-dependent modulus reduction and damping behavior that is observed in actual soils (e.g., Vucetic and Dobry, 1991). We used FLAC's hysteretic model in conjunction with the Mohr-Coulomb model to provide a more accurate representation of the dynamic soil response. The hysteretic model allows the user to provide a modulus reduction curve, expressed as a closed-form equation, which is used by FLAC to modify the elastic response to be consistent with the cyclic behavior of soils as observed in cyclic laboratory experiments. The damping ratio is not specified in FLAC and is obtained based on the results of single-element simulations. We calibrated the hysteretic input parameters to obtain a reasonable match to the target modulus reduction and damping ratio relationships for each soil type.

We selected the target modulus reduction and damping ratio relationships from models available in literature based on the available subsurface and laboratory test data and our experience with dynamic analyses in similar soil types. We modeled the non-liquefiable fine-grained Fluvial-Marine Deposits and Glacial Deposits using the Darendeli (2001) relationships that evaluate the modulus reduction and damping behavior based on the Plasticity Index, overconsolidation ratio, and the overburden effective stress of the soil.

The input parameters for mass density and soil strength parameters were evaluated based on the available subsurface and laboratory test data and our experience with similar soils. The bulk modulus and shear modulus were evaluated using the shear wave velocity profiles described previously and elasticity equations using a Poisson's ratio that was evaluated for each soil unit based on our experience with similar soils. Tables A-1 and A-2 show the Mohr-Coulomb input parameters used in our analyses for the Pier 15 and 16 and Pier 17 and 18 profiles, respectively. These parameters include the mass density, soil strength parameters, modulus reduction curves, and the Poisson's ratio.

A.3.2 Model Implementation

For each soil profile that was included in our site response analysis, the 1D site response analysis model consisted of a single column of elements with parameters corresponding to the soil properties described in Section A.3.1., Figures A-32 to A-33, and Tables A-1 to A-3.

The boundary conditions were set to approximate free-field conditions along the model sides and a non-reflecting (i.e., compliant) boundary at the base of the model. The input base motions were applied as an outcrop motion. The site response analysis was conducted in the following stages:

- Set initial soil stresses based on the assumed soil densities and assuming a horizontal stress coefficient of 0.5;
- Set initial pore pressures based on the location of the water elevation;
- Assign elastic properties based on the shear wave velocity profiles described in Section A.3.1.1.;
- Solve for static equilibrium assuming elastic conditions for all units;
- Assign the Mohr-Coulomb constitutive model to the appropriate soil units and solve for equilibrium;
- Assign the hysteretic constitutive model to the soil units modeled using the Mohr-Coulomb and elastic soil models. Assign PM4SAND constitutive model parameters in the liquefiable soil units;
- Solve for equilibrium to initialize the hysteretic and PM4SAND material models;
- Assign the input ground motion at the base of the model as an upward propagating shear stress history; and
- Solve for the time dependent response of the model due to loading by the input earthquake ground motions.

A.3.3 Site Response Analysis Results

We performed our site-specific site response analysis using the suite of input motions described in Section A.2 using the soil profiles, soil models and input parameters and implementation procedure presented in Section A.3.2. The input motion was applied at the base of the 1D profile and the soil response was computed. Site response analyses were performed for both ground motion design levels (FEE and SEE) considering the subsurface conditions encountered at Pier 15 and 16 and Pier 17 and 18. Profile plots summarizing the peak displacement, peak acceleration, peak shear strain, and peak pore pressure ratio as a function of depth for each set of analyses are summarized in Figures A-34 through A-37. Profile plots summarizing the analyses results for the FEE design level are provided in Figures A-34 and A-35 for the Pier 15 and 16 and Pier 17 and 18 profiles, respectively. Profile plots summarizing the analyses results for the SEE design level are provided in Figures A-36 and A-37 for the Pier 15 and 16 and Pier 17 and 18 profiles, respectively.

In addition, the results of our site response analysis were used to develop our recommended design response spectrum, as described in the following section.

A.4 RECOMMENDED DESIGN RESPONSE SPECTRA

We evaluated our recommended design response spectra based on AASHTO and the GDM. For each soil profile, we computed the surface response spectrum as well as the response spectral ratio, defined as the ratio of the surface response spectrum to the base input response spectrum. A summary of the surface response spectra and response spectral ratios are provided in Figures A-38 and A-39 for the FEE design level and in Figures A-40 and A-41 for the SEE design level.

Per AASHTO and guidance provided in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO, 2015), the surface design response spectrum was evaluated as the target design response spectrum of the base input motions (Figure A-1) multiplied by the smoothed average response spectral ratio. We evaluated the smoothed average response spectral ratio as a smoothed envelope of the geometric mean of the spectral ratios at each period from both analyses performed at each design level. A plot summarizing all the analyses performed for each design level and the smoothed design response spectral ratio is provided in Figures A-42 and A-43 for the FEE and SEE design levels, respectively.

Per AASHTO and the GDM, we evaluated the surface response spectrum from our site-response analysis based on the smoothed average response spectral ratio provided in Figures A-42 and A-43 and the target design response spectrum of the base input motions provided in Figure A-1. In addition, AASHTO and the GDM specify that the recommended design response spectrum developed from a site-specific site response analysis cannot be less than two-thirds of the specification-based spectrum evaluated for the site soil conditions without any modifications for liquefaction. Based on the measured shear wave velocity data, the seismic site conditions correspond to a Site Class D. Plots of the Site Class D specification-based spectrum, two-thirds of the Site Class D specification-based spectrum (i.e., the minimum allowable design spectrum), the target design response spectrum of the base input motions, and the surface response spectrum evaluated from our site-response analysis are provided in Figures A-44 and A-45 for the FEE and SEE design events, respectively.

For the FEE design level, Figure A-44 indicates that the surface response spectrum developed from our site-response analysis is greater than the minimum allowable spectrum

between periods of approximately 0.25 to 3 seconds and is less than the minimum allowable spectrum at all other periods. We developed our recommended design response spectrum for the FEE design level by increasing the spectrum from our site response analysis to meet the minimum allowable spectrum at periods below 0.25 seconds and above 3 seconds and smoothing the transitions to provide a reasonable spectral response in terms of acceleration, velocity, and displacement. Figure A-46 shows the acceleration, velocity, and displacement spectral response for our recommended design response spectrum, the Site Class D specification-based spectrum, and the minimum allowable spectrum.

For the SEE design level, Figure A-45 indicates that the surface response spectrum developed from our site-response analysis is less than the minimum allowable spectrum between periods of approximately 0.075 to 0.6 seconds and is greater than the minimum allowable spectrum at all other periods. We developed our recommended design response spectrum for the SEE design level by increasing the spectrum from our site response analysis to meet the minimum allowable spectrum at periods between 0.075 and 0.6 seconds and extending the plateau of the acceleration response spectrum to 1.1 seconds to provide a consistent spectral acceleration response over this period range. In addition, we performed minor smoothing at the transitions to provide a reasonable spectral response in terms of acceleration, velocity, and displacement. Figure A-47 shows the acceleration, velocity, and displacement spectral response for our recommended design response spectrum, the Site Class D specification-based spectrum, and the minimum allowable spectrum. Our recommended design response spectrum for both the FEE and SEE design events are summarized on a linear-linear plot and in tabular form in the main text.

A.5 REFERENCES

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Table A-1 - Pier 15 and 16 Site-specific Site Response Analysis Soil Input Parameters

Dynamic Soil Unit	Top Depth (feet)	Bottom Depth (feet)	V_S [1] (fps) Best	V_S [1] (fps) Min	V_S [1] (fps) Max	Poisson's Ratio	Density (slugs/ft^3)	Constitutive Model	Corrected SPT N-Value, $(N_1)_{60cs}$ (bpf)	Friction Angle (degrees)	Cohesion (psf)	Modulus Reduction and Damping Relationship
Fill (Sand)	0	13	525	405	685	0.25	3.5	PM4SAND	10	NA [2]	-	NA [3]
Alluvium (Sand)	13	24	450	345	585	0.3	3.6	PM4SAND	22	NA [2]	-	NA [3]
Alluvium (Sand)	24	30	530	410	690	0.3	3.6	PM4SAND	22	NA [2]	-	NA [3]
Alluvium (Sand)	30	39	575	440	750	0.3	3.6	PM4SAND	22	NA [2]	-	NA [3]
Fluvial-Marine Deposits (Sand)	39	45	625	480	815	0.33	3.7	PM4SAND	30	NA [2]	-	NA [3]
Fluvial-Marine Deposits (Sand)	45	51	675	520	880	0.33	3.7	PM4SAND	30	NA [2]	-	NA [3]
Fluvial-Marine Deposits (Sand)	51	57	735	565	955	0.33	3.7	PM4SAND	30	NA [2]	-	NA [3]
Fluvial-Marine Deposits (Sand)	57	63	810	625	1,055	0.33	3.7	PM4SAND	30	NA [2]	-	NA [3]
Fluvial-Marine Deposits (Sand and Silt)	63	72	875	675	1,140	0.3	3.6	PM4SAND	20	NA [2]	-	NA [3]
Fluvial-Marine Deposits (Sand and Silt)	72	78	920	710	1,195	0.3	3.6	PM4SAND	20	NA [2]	-	NA [3]
Glacial Lacustrine (Silt)	78	84	960	740	1,250	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=3.0 atm
Glacial Lacustrine (Silt)	84	90	1,060	815	1,380	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=3.0 atm
Glacial Lacustrine (Silt)	90	96	1,235	950	1,605	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=3.0 atm
Glacial Lacustrine (Silt)	96	102	1,290	990	1,680	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=3.0 atm
Glacial Lacustrine (Silt)	102	108	1,450	1,115	1,885	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=3.0 atm
Glacial Lacustrine (Silt)	108	114	1,525	1,175	1,985	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=3.5 atm
Glacial Lacustrine (Silt)	114	120	1,450	1,115	1,885	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=3.5 atm
Glacial Lacustrine (Silt)	120	126	1,325	1,020	1,725	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=3.5 atm
Glacial Lacustrine (Silt)	126	135	1,250	960	1,625	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=4.0 atm
Glacial Lacustrine (Silt)	135	141	1,460	1,115	1,885	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=4.0 atm
Glacial Till (Clay)	141	144	1,700	1,385	2,340	0.4	3.8	Mohr-Coulomb	-	19	4,000	Darendelli (2001) - PI=20, OCR=5, σ'_v=4.5 atm
Glacial Till (Clay)	144	-	1,850	1,425	2,405	0.4	3.8				Base of Model	

NOTES:
1 The shear wave velocity profiles were estimated based on the measured values at each boring location. Minimum and maximum values were evaluated by varying the estimated shear wave velocity values by approximately +/- 30 percent.
2 The strength of the soils modeled as PM4SAND is evaluated by the consitutive model as a function of generated excess pore pressure and the intial relative density of the soil evaluated based on the corrected SPT blowcount.
3 PM4SAND does not use FLAC's hysteretic model to approximate modulus reduction and damping behavior, the model input parameters were calibrated to approximate the EPRI (1993) curves at small strains.
atm = atmospheric pressure, bpf = blows per foot, fps = feet per second, ft^3 = cubic feet, OCR = overconsolidation ratio, PI = Plasticity Index, SPT = Standard Penetration Test, V_S = shear wave velocity, σ'_v = vertical effective stress.

Table A-2 - Piers 17 and 18 Site-specific Site Response Analysis Soil Input Parameters

Dynamic Soil Unit	Top Depth (feet)	Bottom Depth (feet)	V_S [1] (fps) Best	V_S [1] (fps) Min	V_S [1] (fps) Max	Poisson's Ratio	Density (slugs/ft^3)	Constitutive Model	Corrected SPT N-Value, $(N_1)_{60cs}$ (bpf)	Friction Angle (degrees)	Cohesion (psf)	Modulus Reduction and Damping Relationship
Alluvium (Sand)	0	18	475	365	620	0.3	3.5	PM4SAND	14	NA [2]	-	NA [3]
Alluvium (Sand)	18	24	520	400	675	0.3	3.5	PM4SAND	14	NA [2]	-	NA [3]
Alluvium (Sand)	24	30	590	455	770	0.3	3.5	PM4SAND	14	NA [2]	-	NA [3]
Alluvium (Sand)	30	42	690	530	900	0.33	3.7	PM4SAND	30	NA [2]	-	NA [3]
Alluvium (Sand)	42	48	610	470	795	0.33	3.7	PM4SAND	30	NA [2]	-	NA [3]
Alluvium (Sand)	48	54	655	505	850	0.33	3.7	PM4SAND	30	NA [2]	-	NA [3]
Alluvium (Sand)	54	60	690	530	900	0.33	3.7	PM4SAND	30	NA [2]	-	NA [3]
Alluvium (Sand)	60	66	700	540	910	0.3	3.6	PM4SAND	25	NA [2]	-	NA [3]
Alluvium (Sand)	66	72	750	575	975	0.3	3.6	PM4SAND	25	NA [2]	-	NA [3]
Alluvium (Sand)	72	78	805	620	1,045	0.3	3.6	PM4SAND	25	NA [2]	-	NA [3]
Alluvium (Sand)	78	102	850	655	1,105	0.3	3.6	PM4SAND	25	NA [2]	-	NA [3]
Alluvium (Sand)	102	141	860	660	1,120	0.33	3.7	PM4SAND	35	NA [2]	-	NA [3]
Alluvium (Sand)	141	144	900	690	1,170	0.33	3.7	PM4SAND	35	NA [2]	-	NA [3]
Alluvium (Sand)	144	147	1,010	775	1,315	0.33	3.7	PM4SAND	35	NA [2]	-	NA [3]
Alluvium (Sand)	147	150	1,150	885	1,495	0.33	3.7	PM4SAND	35	NA [2]	-	NA [3]
Fluvial-Marine Deposits (Silt)	150	156	1,225	940	1,595	0.4	3.5	Mohr-Coulomb	-	30	-	Darendelli (2001) - PI=4, OCR=1.1, σ'_v=4.5 atm
Glacial Lacustrine (Silt)	156	159	1,500	1,155	1,950	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=5.0 atm
Glacial Lacustrine (Silt)	159	162	1,750	1,345	2,275	0.4	3.7	Mohr-Coulomb	-	35	-	Darendelli (2001) - PI=4, OCR=5, σ'_v=5.0 atm
Glacial Lacustrine (Silt)	162	-	1,850	1,425	2,405	0.4	3.7	Base of Model				

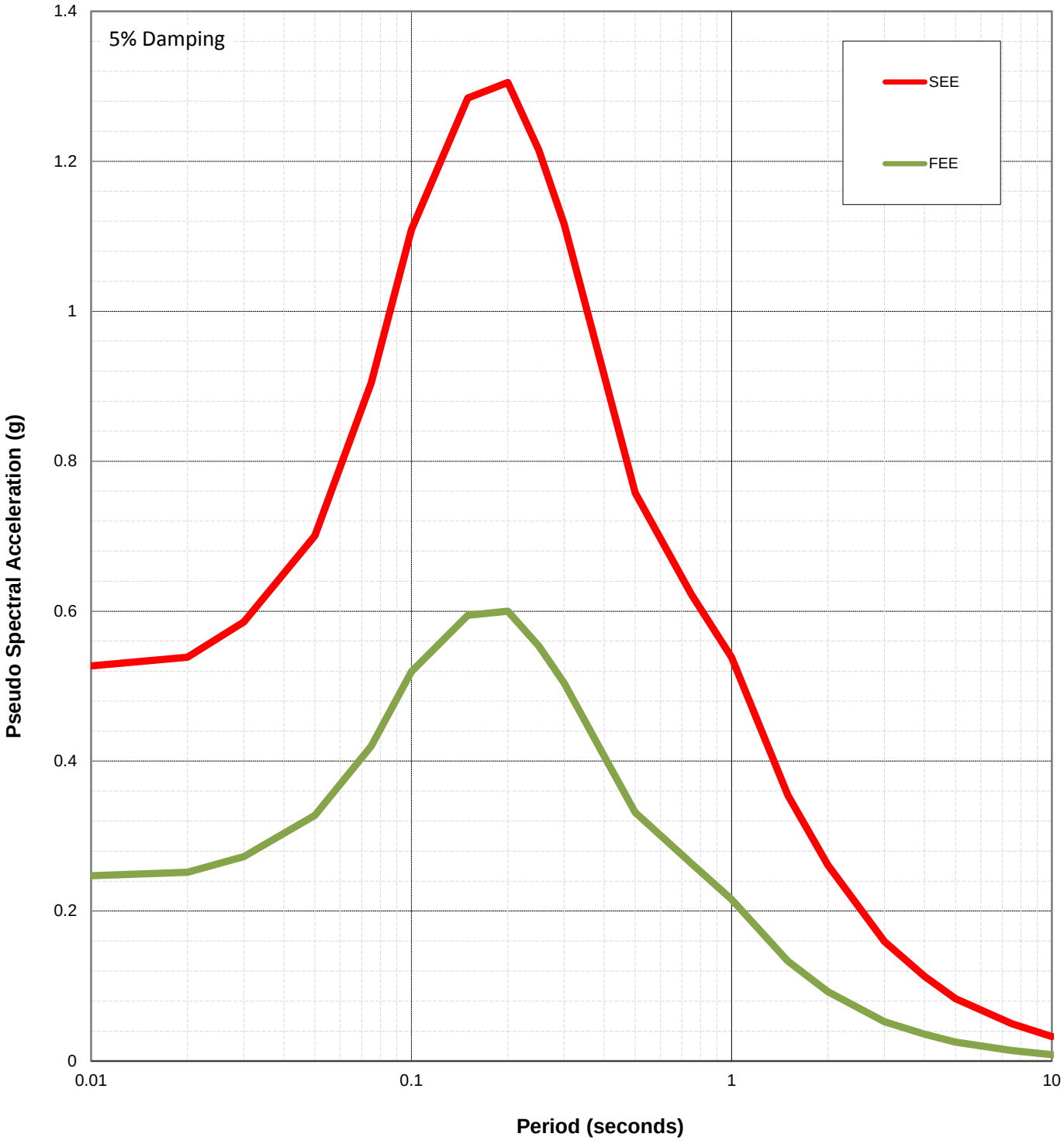
NOTES:

1 The shear wave velocity profiles were estimated based on the measured values at each boring location. Minimum and maximum values were evaluated by varying the estimated shear wave velocity values by approximately +/- 30 percent.

2 The strength of the soils modeled as PM4SAND is evaluated by the consitutive model as a function of generated excess pore pressure and the intial relative density of the soil evaluated based on the corrected SPT blowcount.

3 PM4SAND does not use FLAC's hysteretic model to approximate modulus reduction and damping behavior, the model input parameters were calibrated to approximate the EPRI (1993) curves at small strains.

atm = atmospheric pressure, bpf = blows per foot, fps = feet per second, ft^3 = cubic feet, OCR = overconsolidation ratio, PI = Plasticity Index, SPT = Standard Penetration Test, V_S = shear wave velocity, σ'_v = vertical effective stress.



NOTES

FEE = functional evaluation earthquake; SEE = safety evaluation earthquake; SF = scale factor

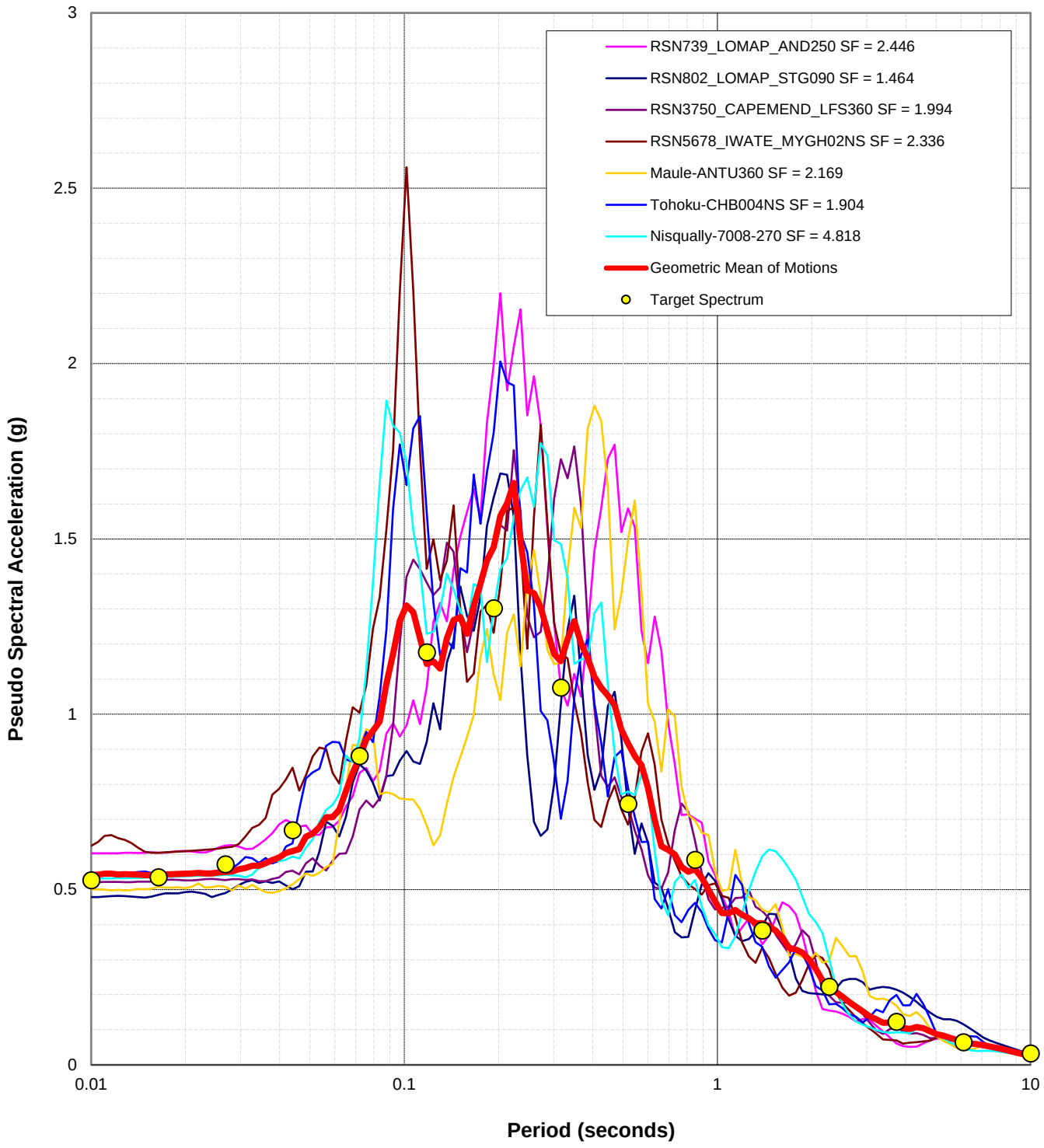
SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

BASE ACCELERATION SPECTRA
SITE CLASS C

September 2020 105492-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. A-1



NOTES

SEE = safety evaluation earthquake; SF = scale factor

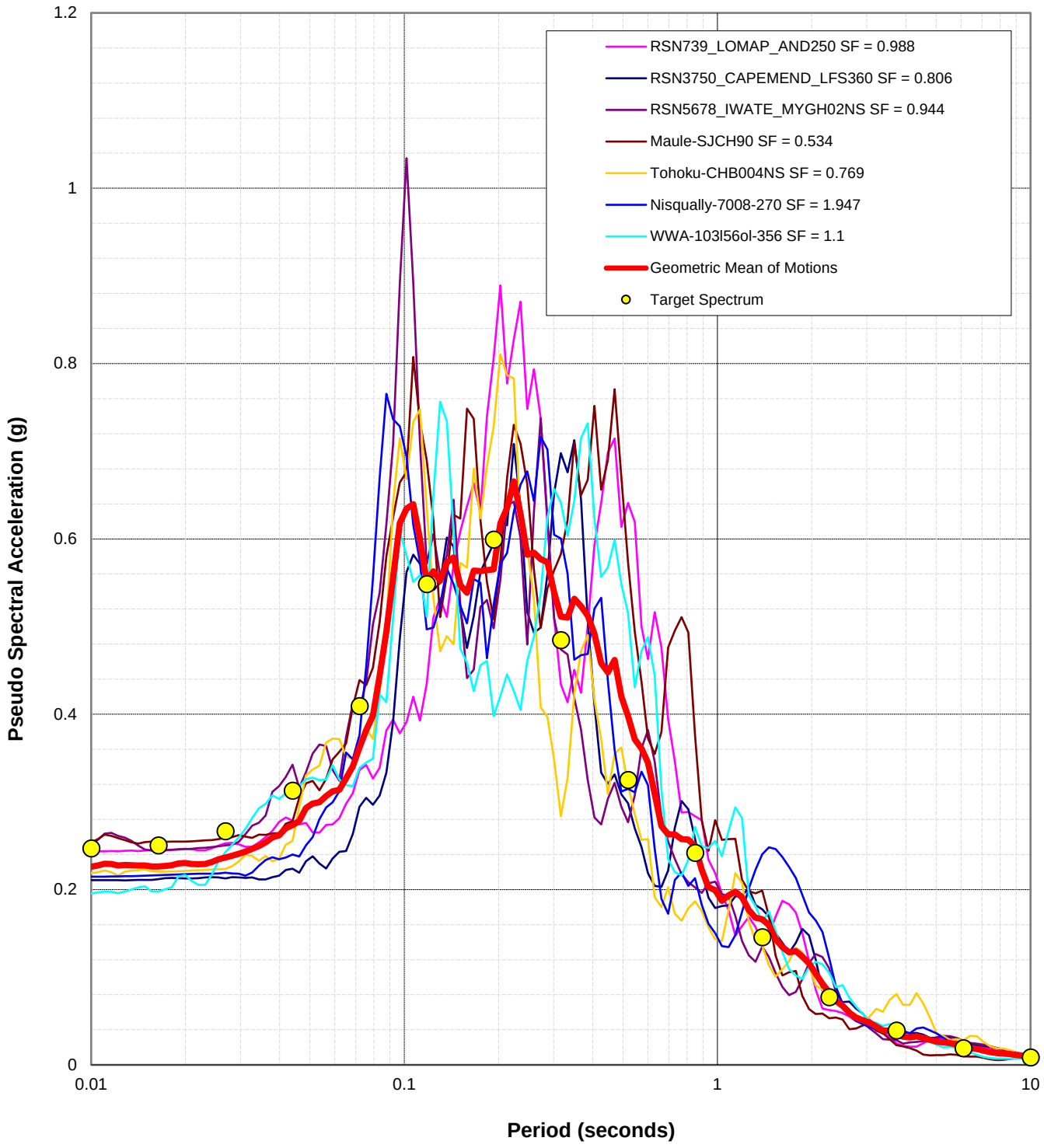
SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

SEE SCALED MOTIONS RESPONSE SPECTRA

September 2020 105492-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. A-2



NOTES

FEE = functional evaluation earthquake; SF = scale factor

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

FEE SCALED MOTIONS RESPONSE SPECTRA

September 2020 105492-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. A-3

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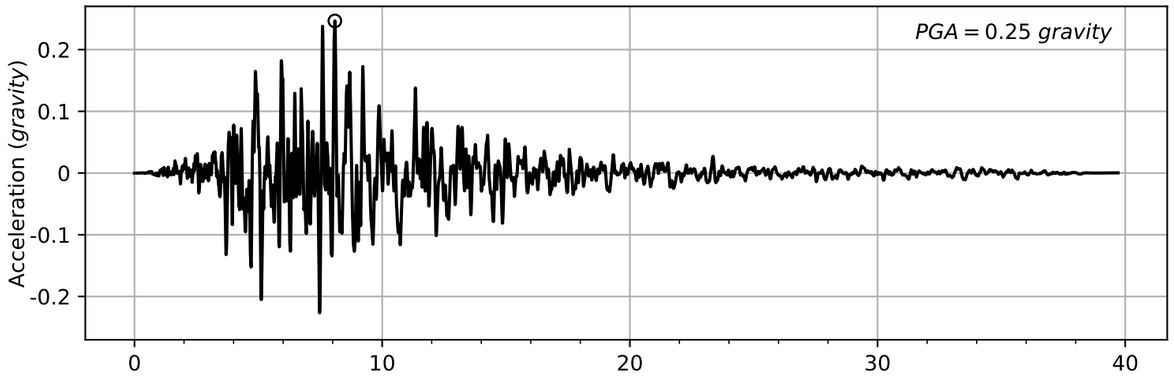

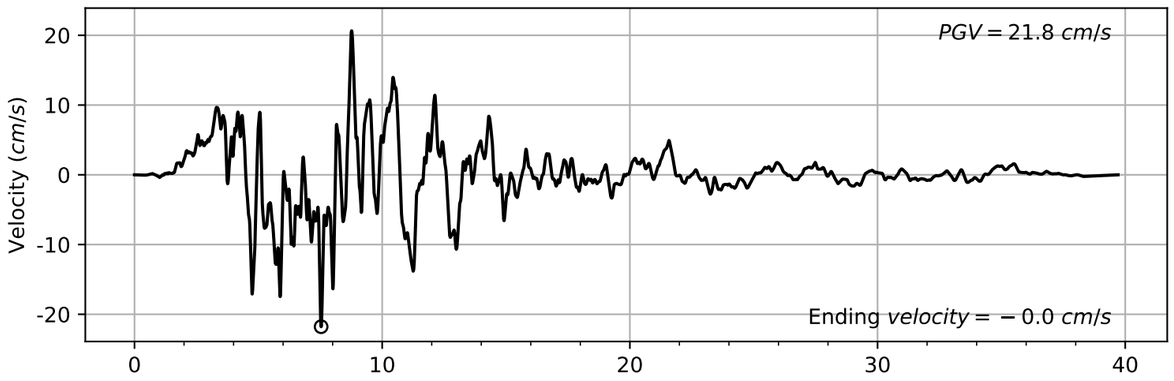

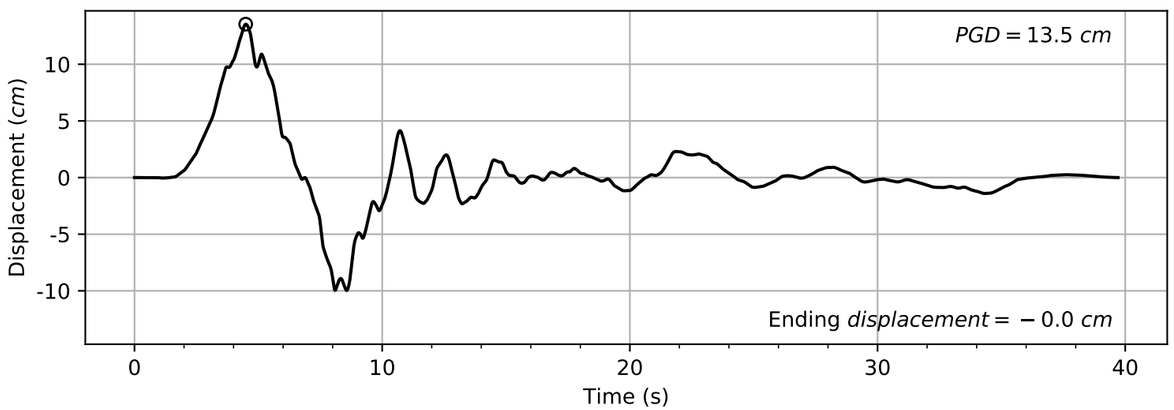

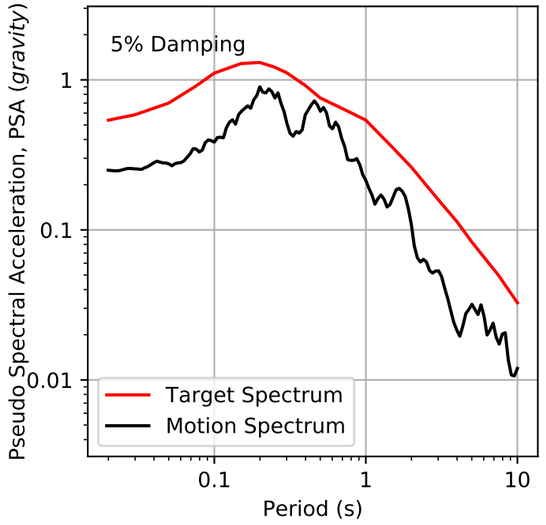

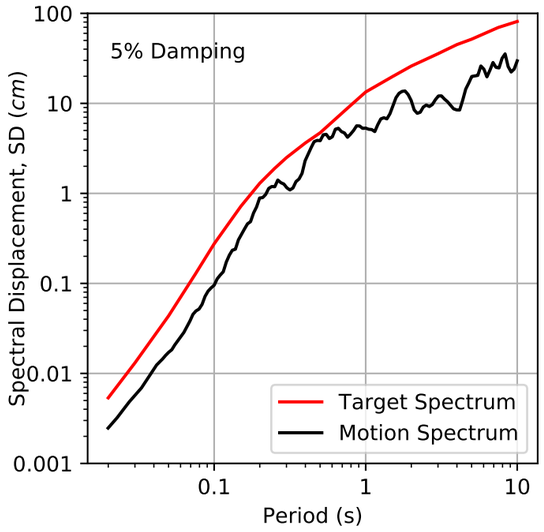

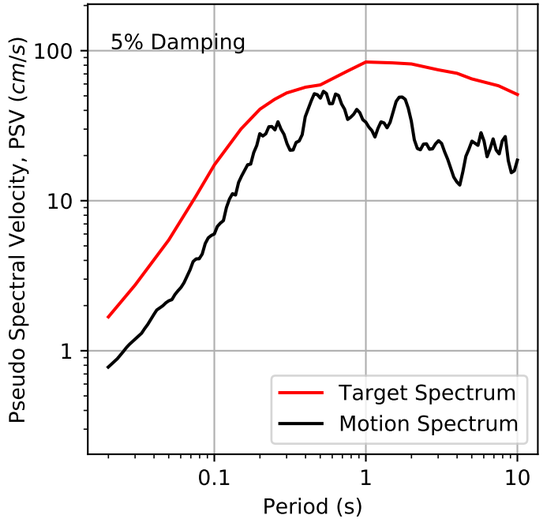

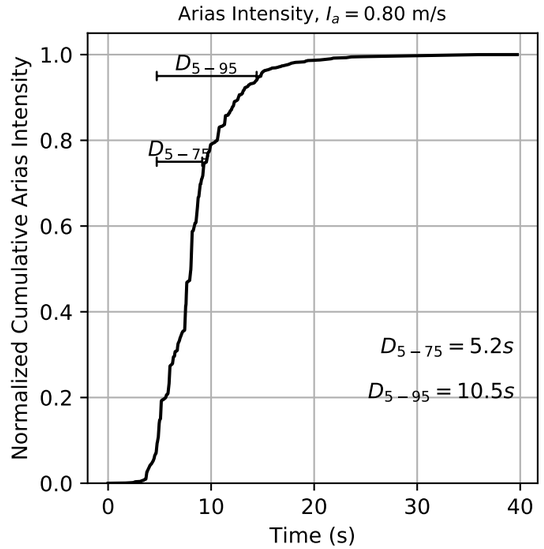

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
3. cm = centimeters
4. cm/s = centimeters per second
5. s = seconds
6. D_{5-75} = significant duration from 5 to 75 percent of normalized cumulative Arias intensity
7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

LOMA PRIETA 1989
ANDERSON DAM (DS), 250°
SEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-4

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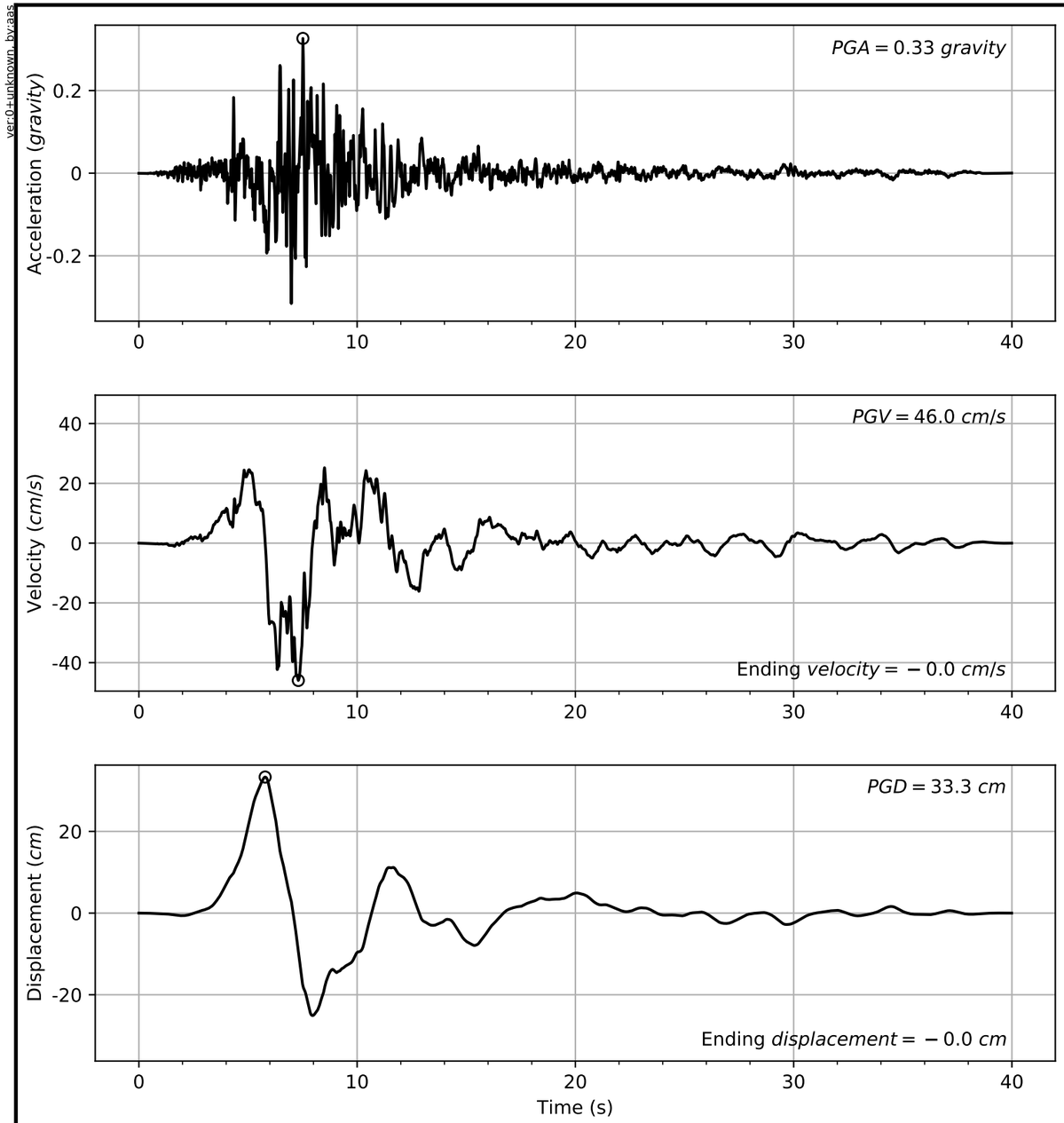

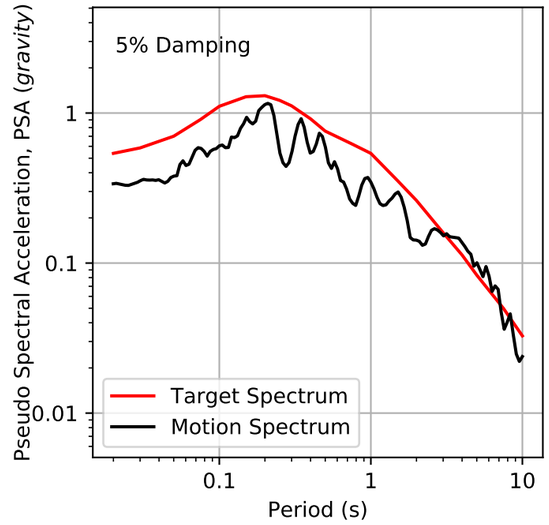

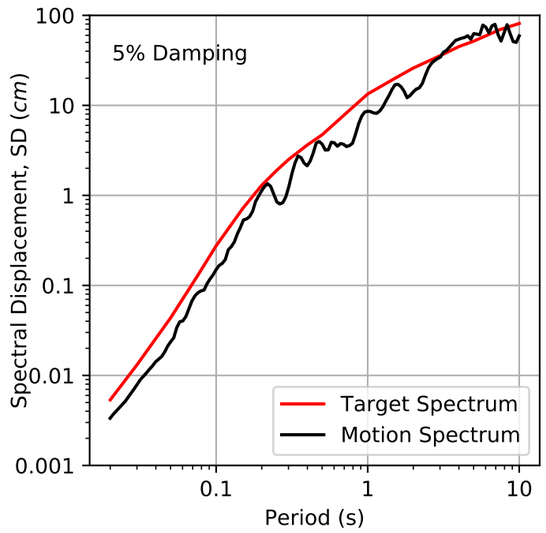

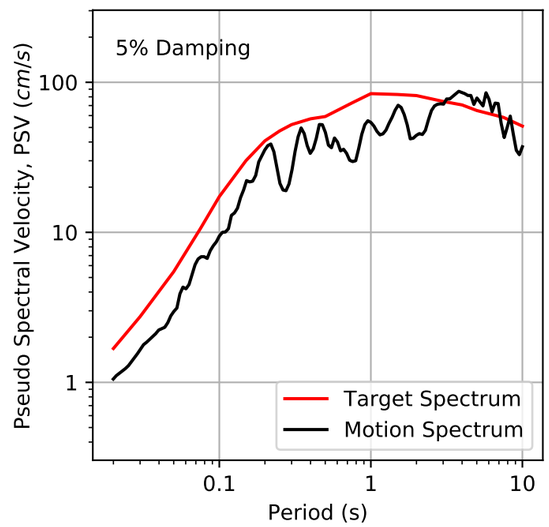

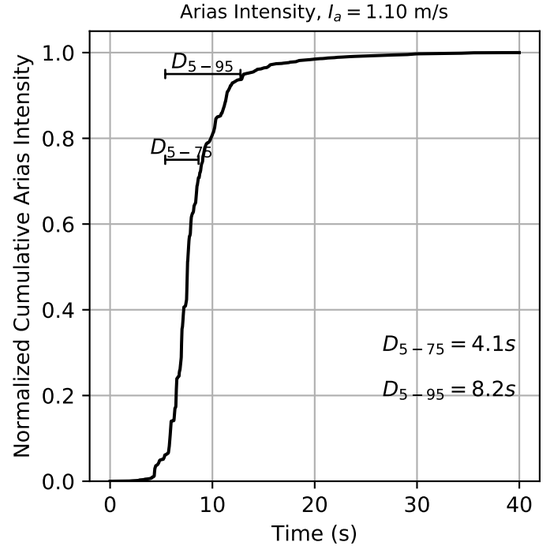

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
3. cm = centimeters
4. cm/s = centimeters per second
5. s = seconds
6. D_{5-75} = significant duration from 5 to 75 percent of normalized cumulative Arias intensity
7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

LOMA PRIETA 1989
SARATOGA - ALOHA AVE, 90°
SEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-5

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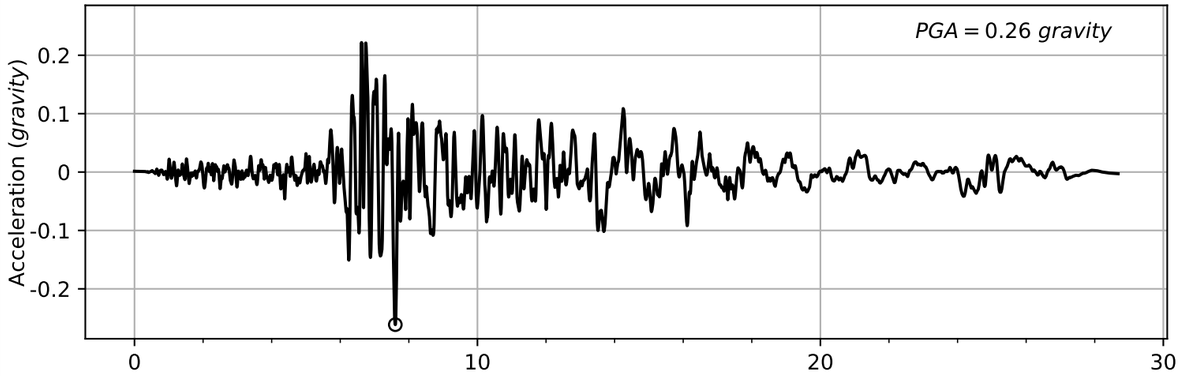

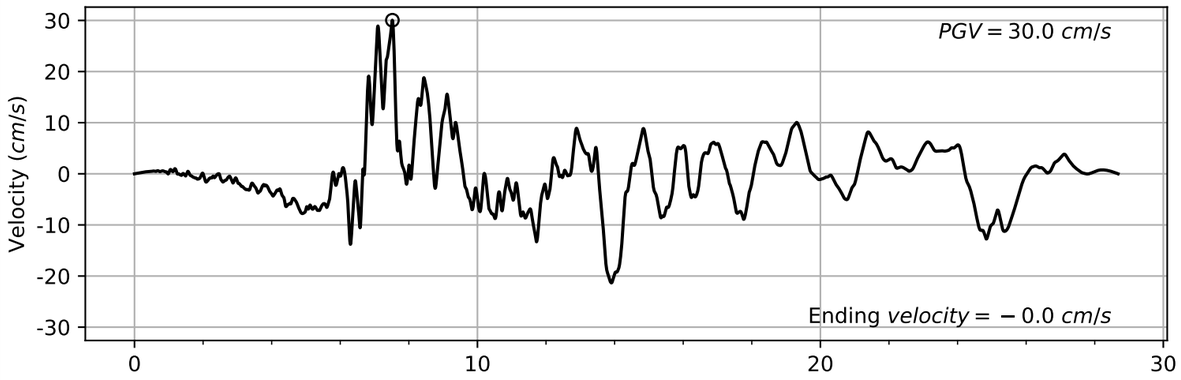

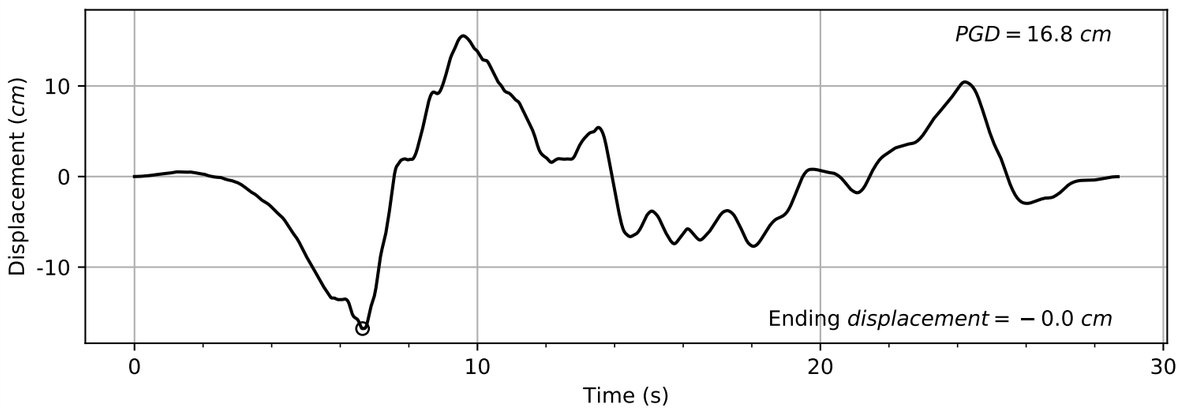

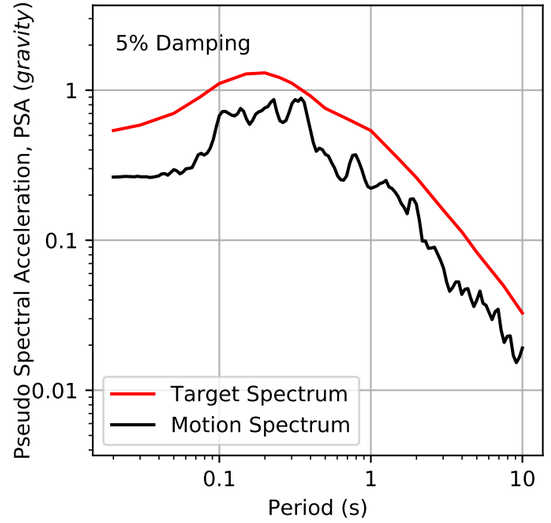

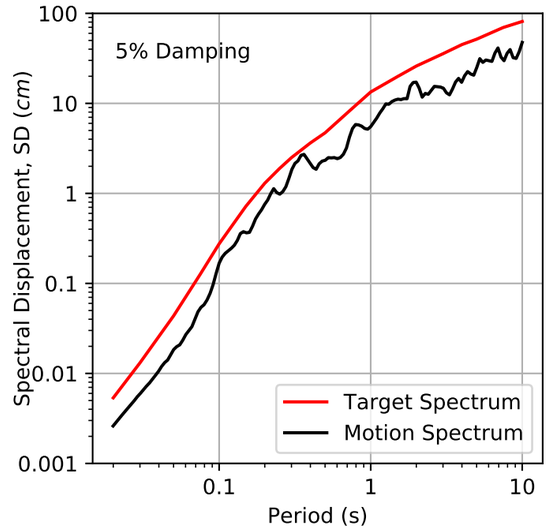

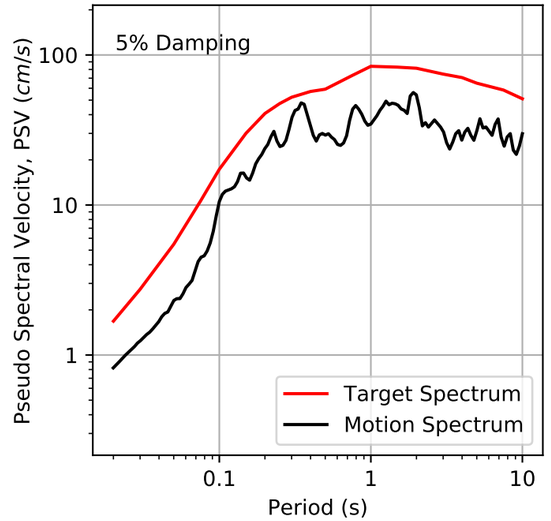

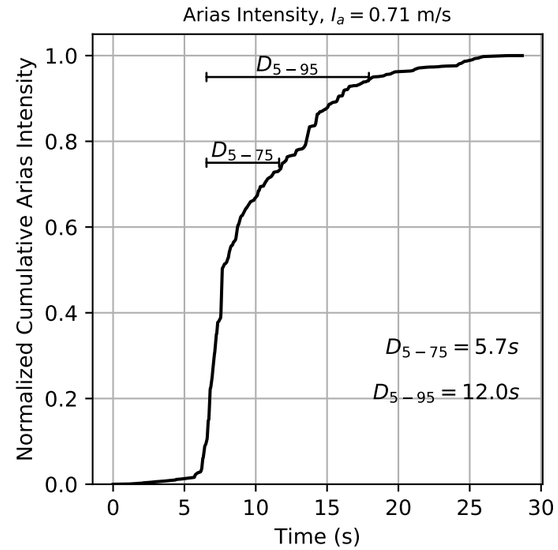

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
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4. cm/s = centimeters per second
5. s = seconds
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7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

CAPE MENDOCINO 1992
LOLETA FIRE STATION, 360°
SEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-6

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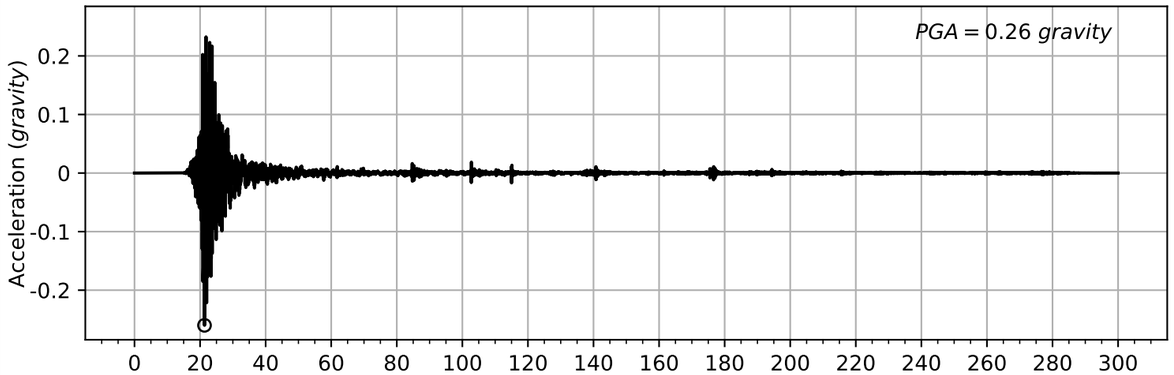

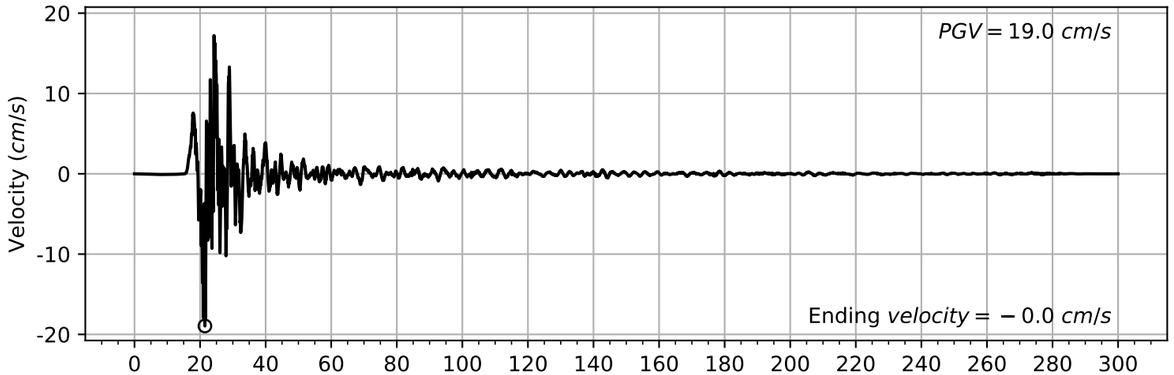

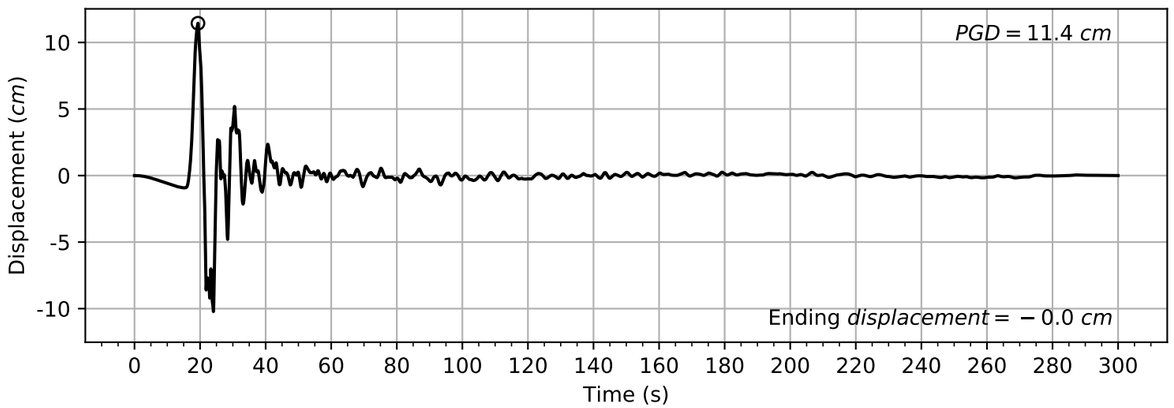

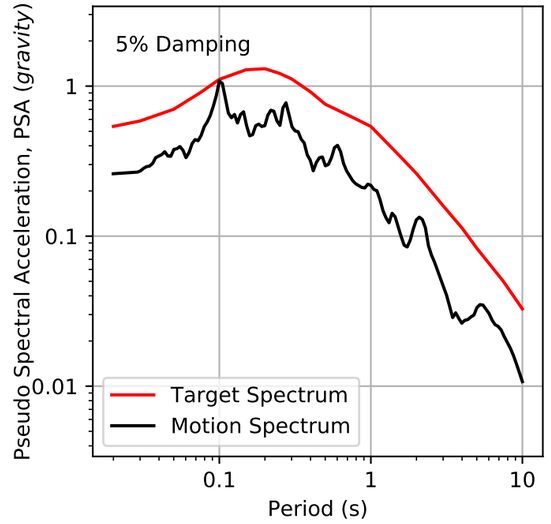

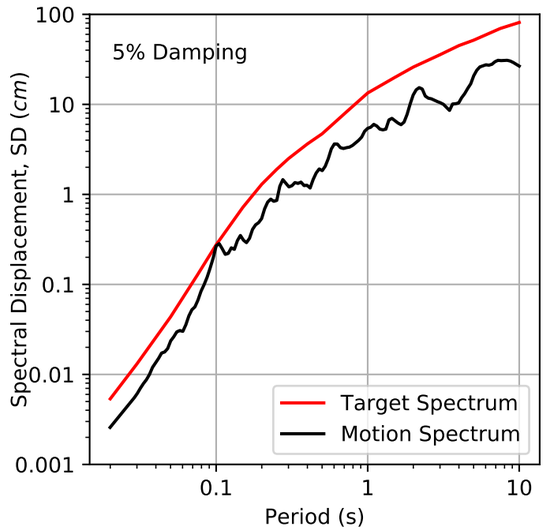

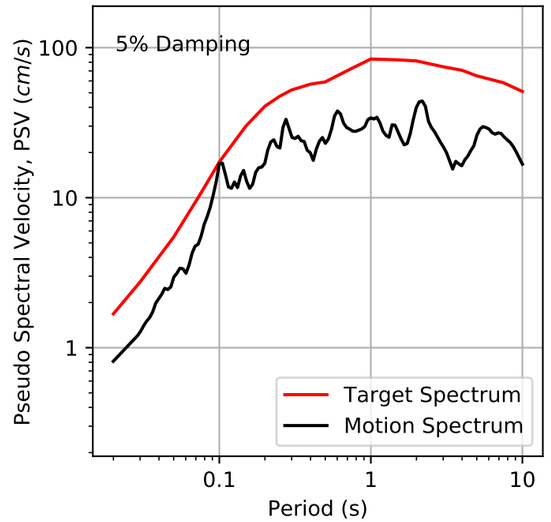

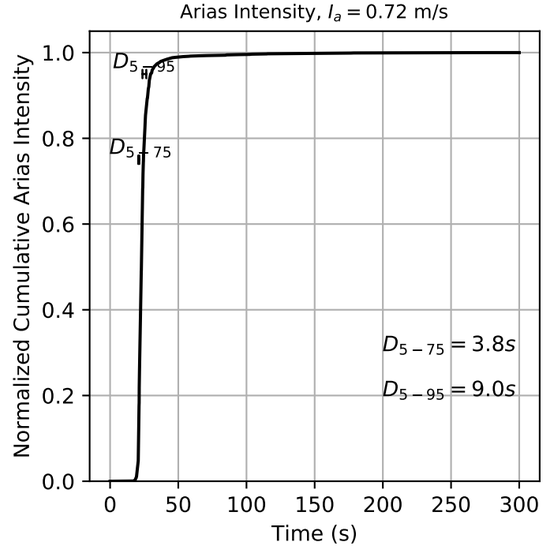

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
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5. s = seconds
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7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

IWATE, JAPAN 2008
MYGH02, NS
SEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-7

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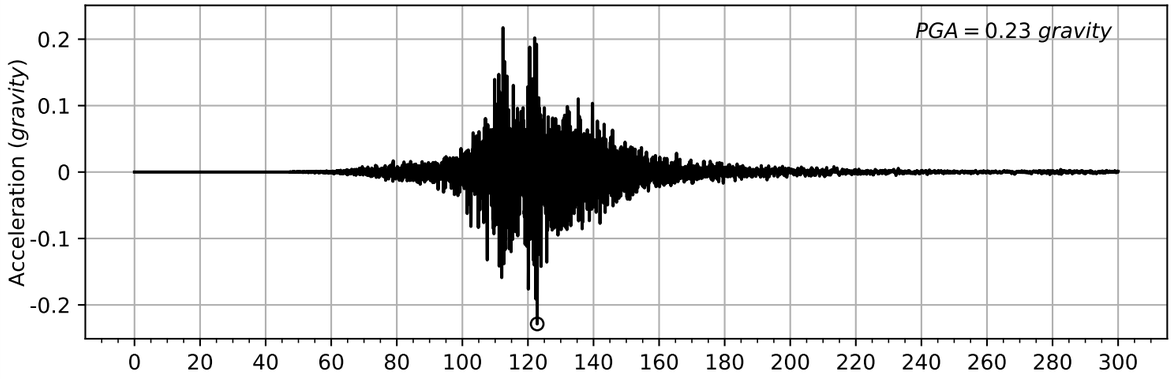

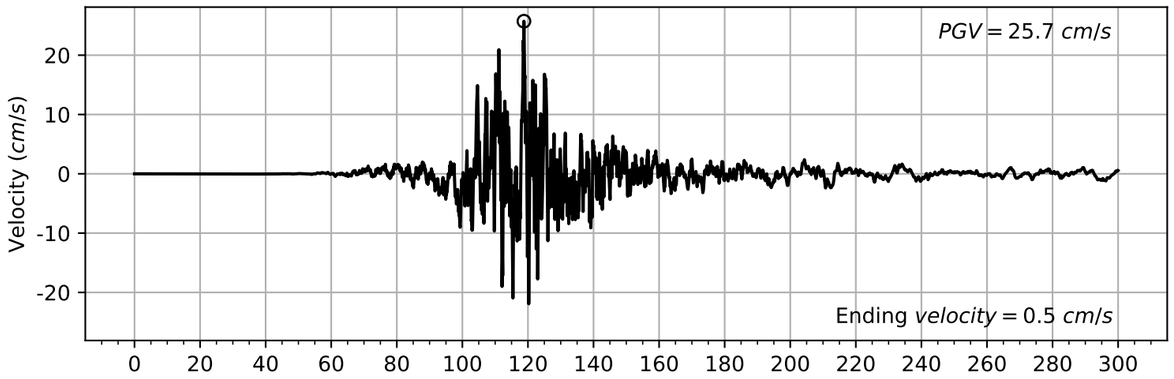

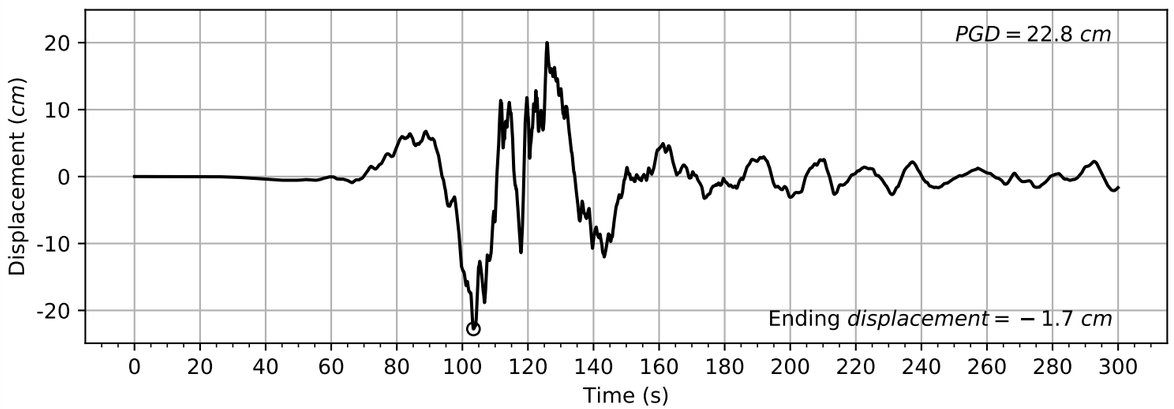

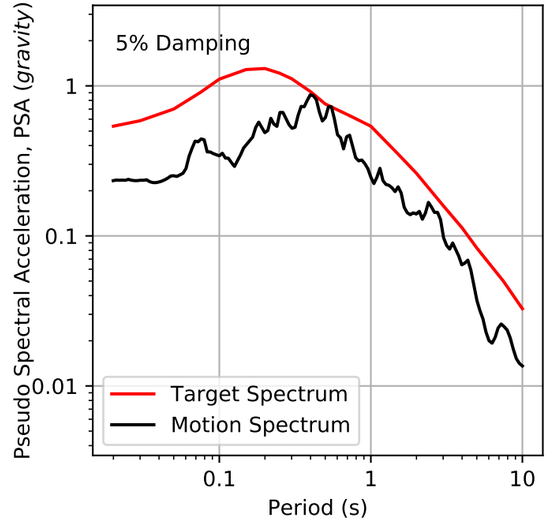

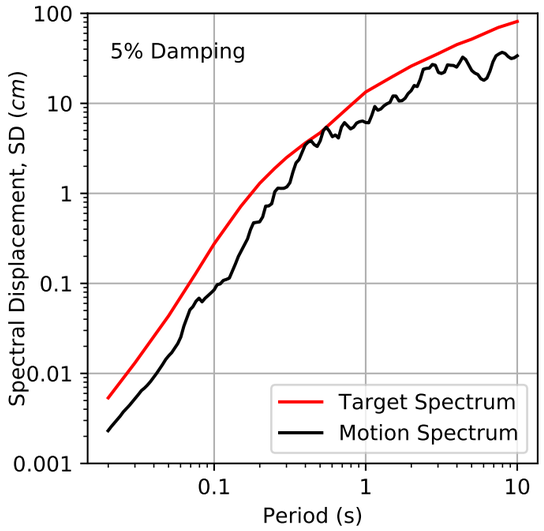

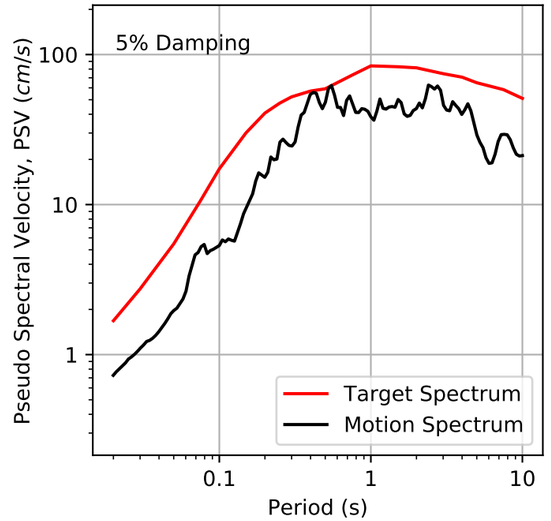

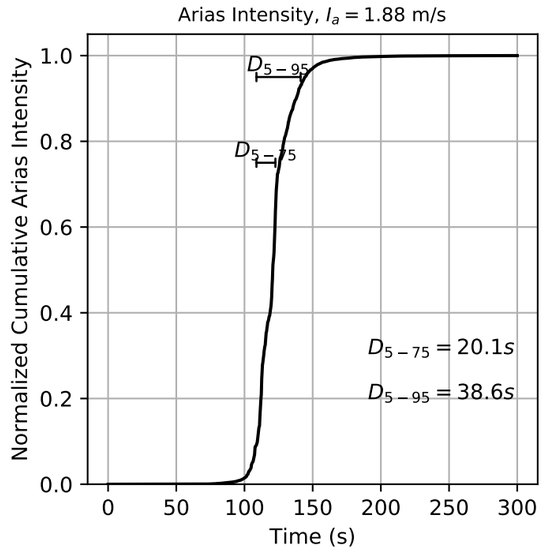

Notes:
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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

MAULE, CHILE 2010
CIEN AGRONOMICAS, 360°
SEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-8

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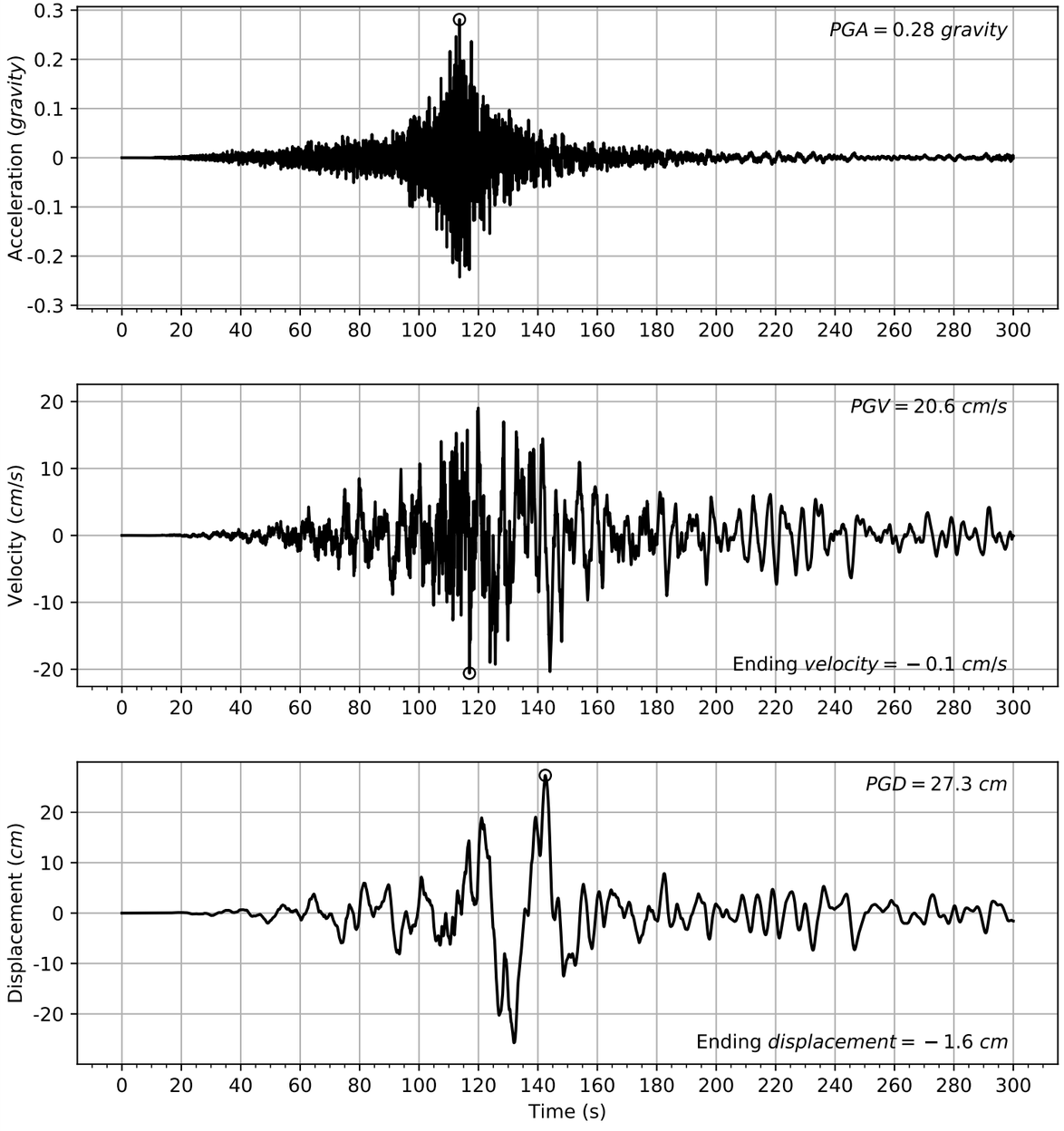

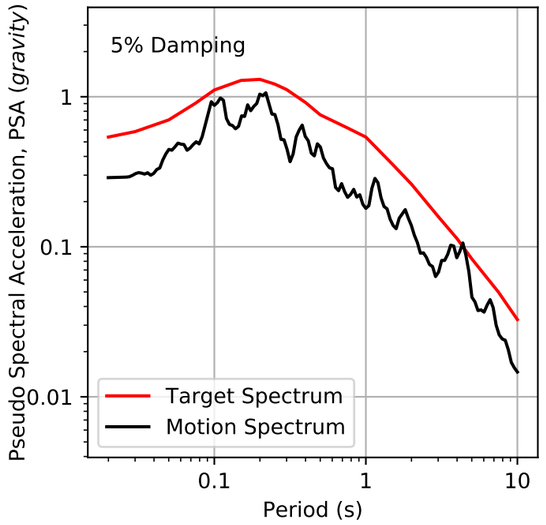

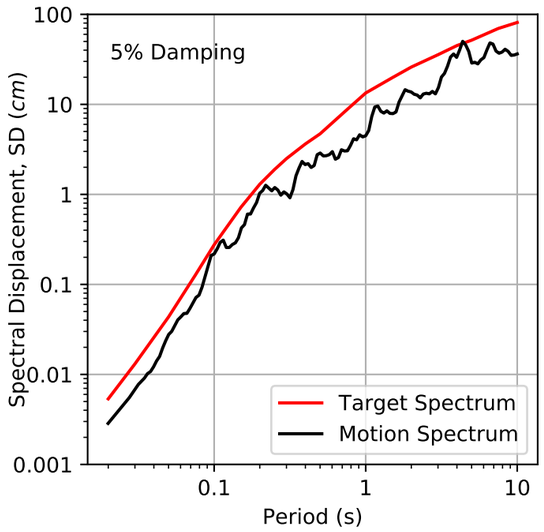

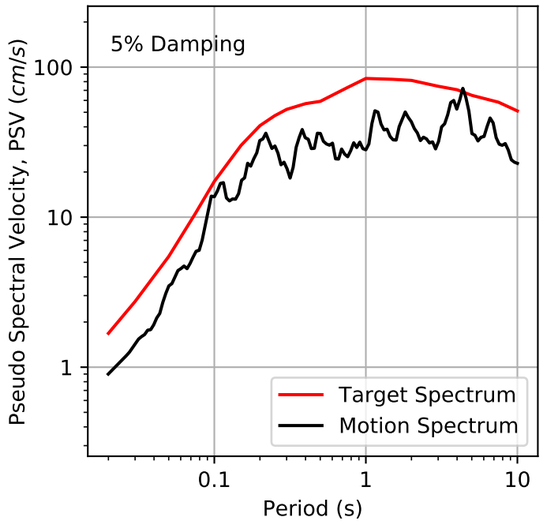

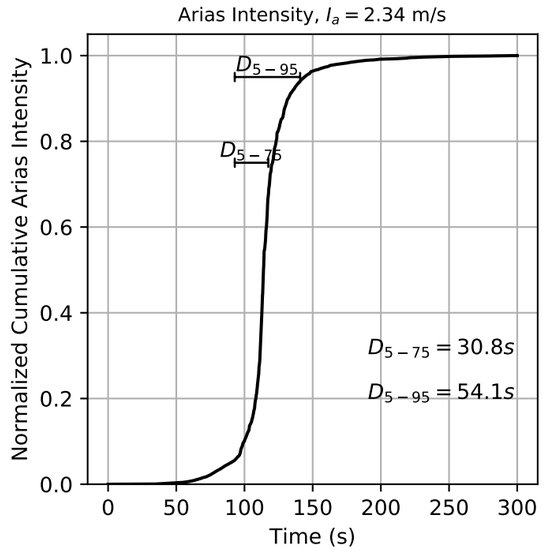

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
3. cm = centimeters
4. cm/s = centimeters per second
5. s = seconds
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7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

TOHOKU, JAPAN 2011
SAWARA, NS
SEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-9

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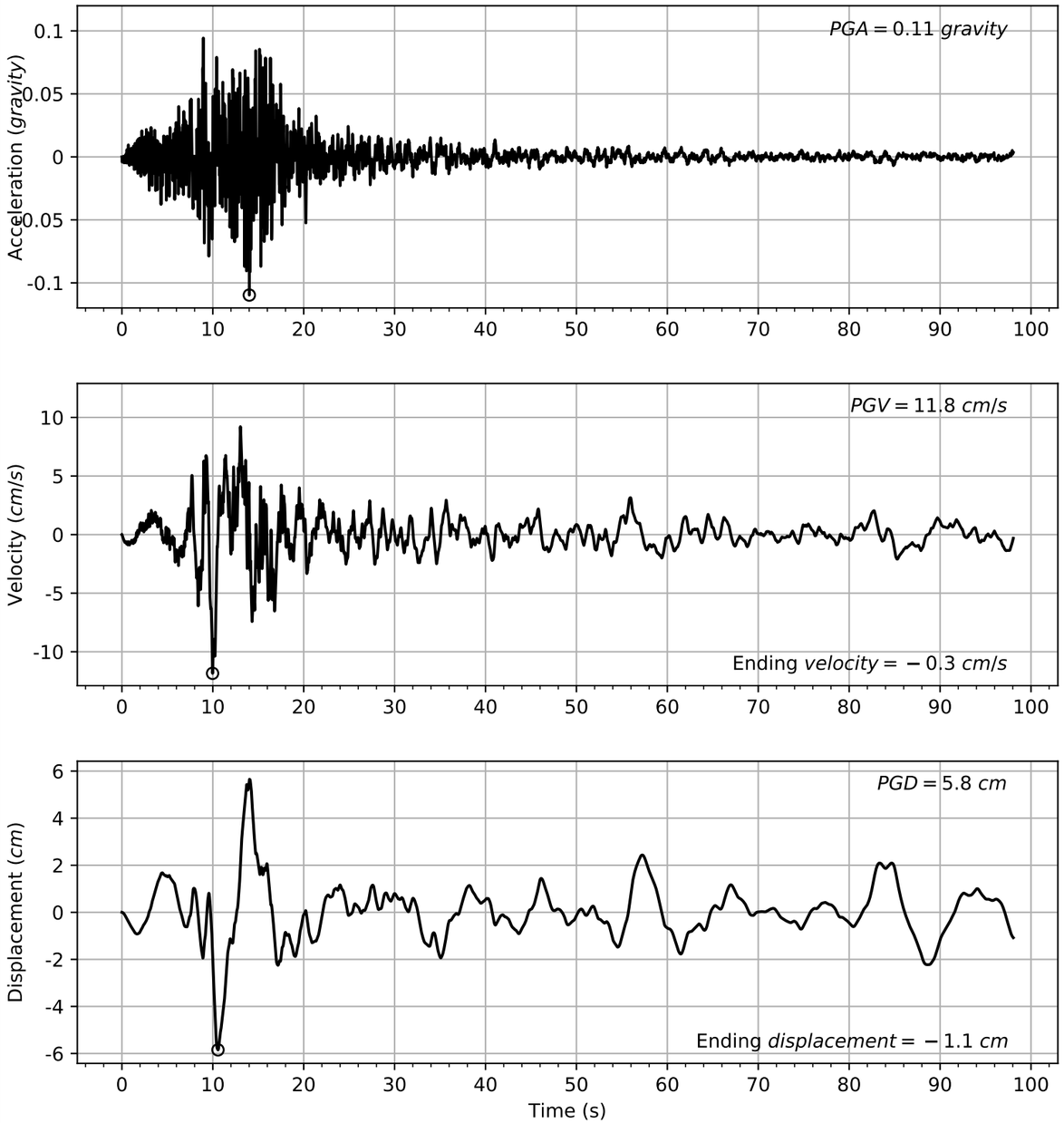

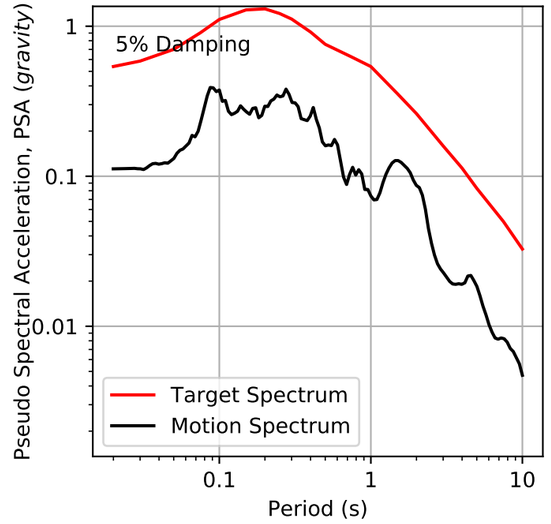

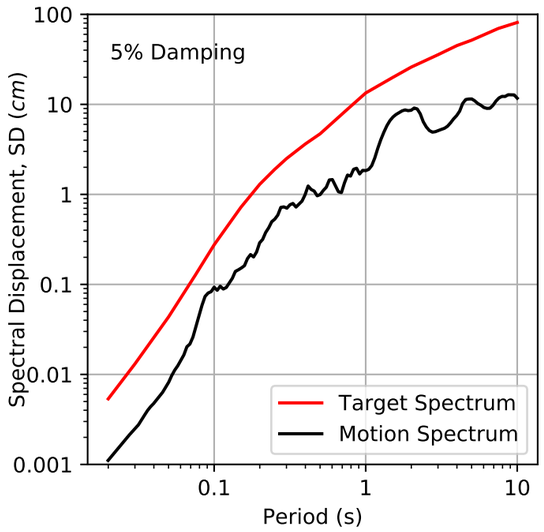

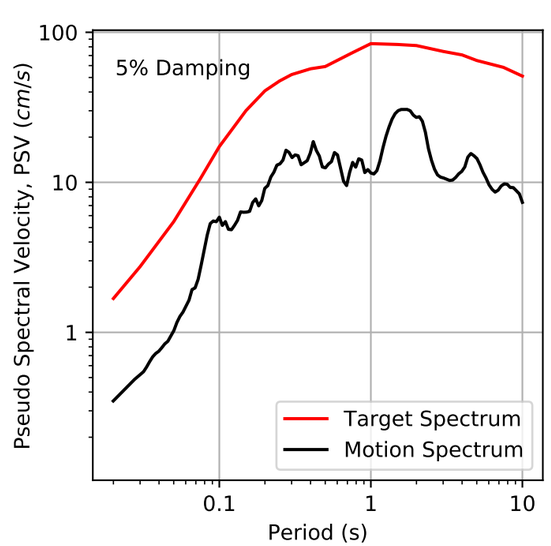

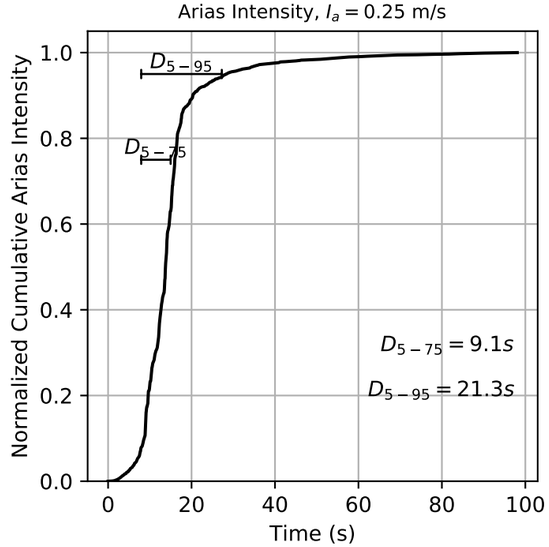

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
3. cm = centimeters
4. cm/s = centimeters per second
5. s = seconds
6. D_{5-75} = significant duration from 5 to 75 percent of normalized cumulative Arias intensity
7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

NISQUALLY, WASHINGTON 2001
HALVERSON RES., 270°
SEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-10

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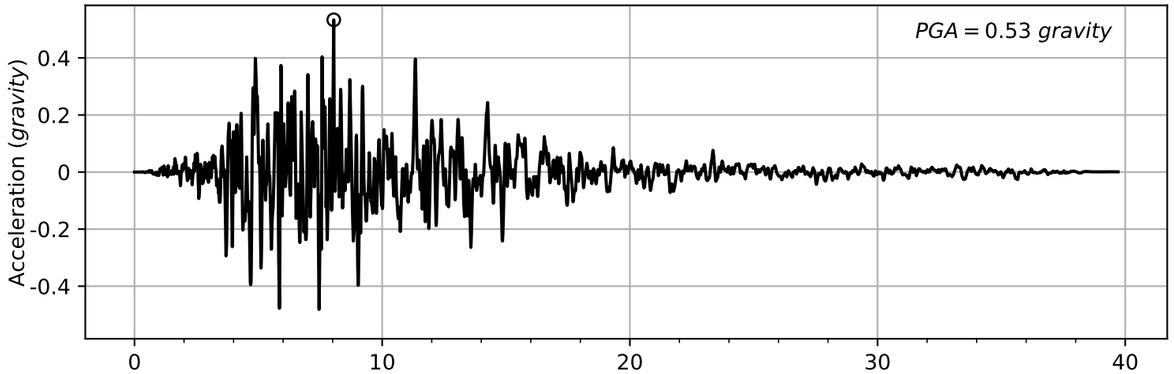

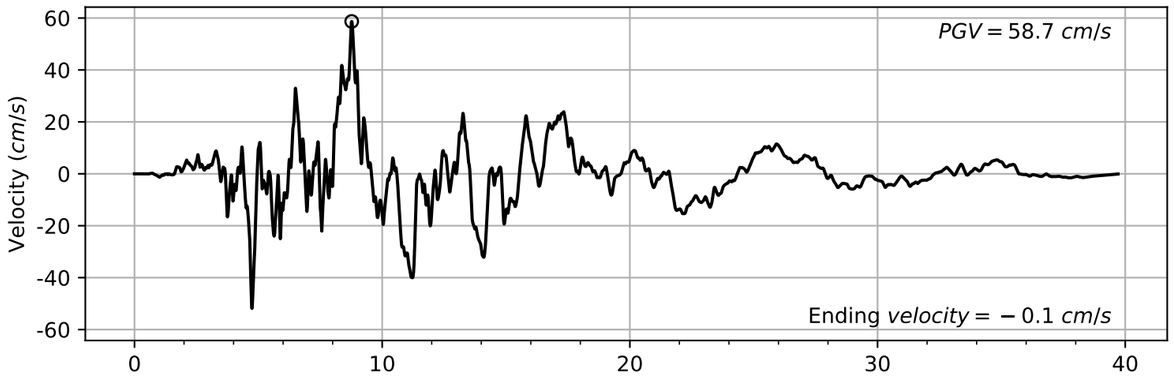

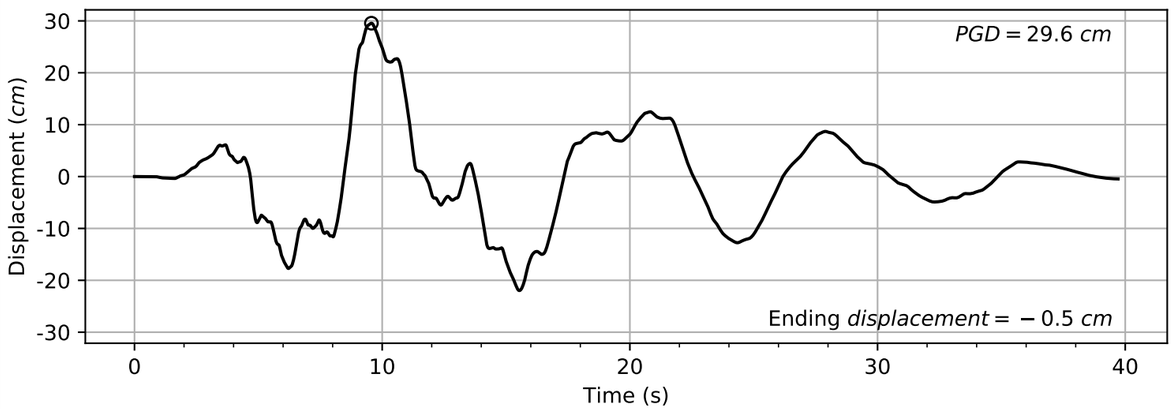

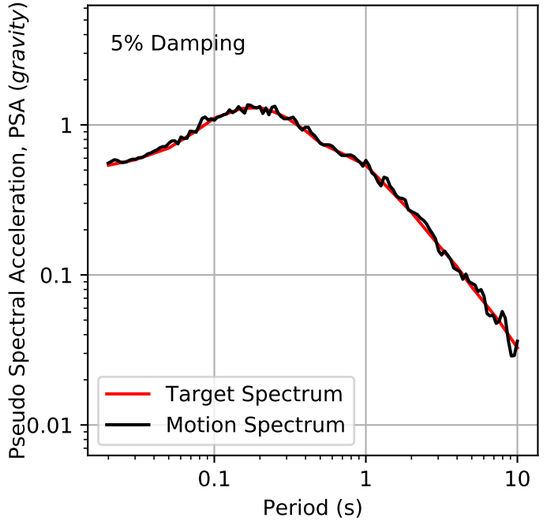

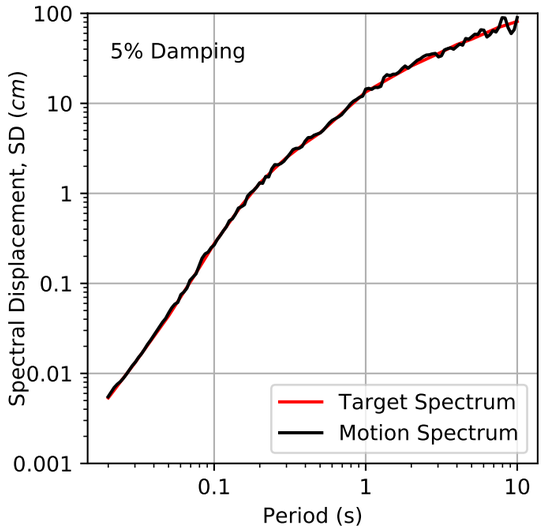

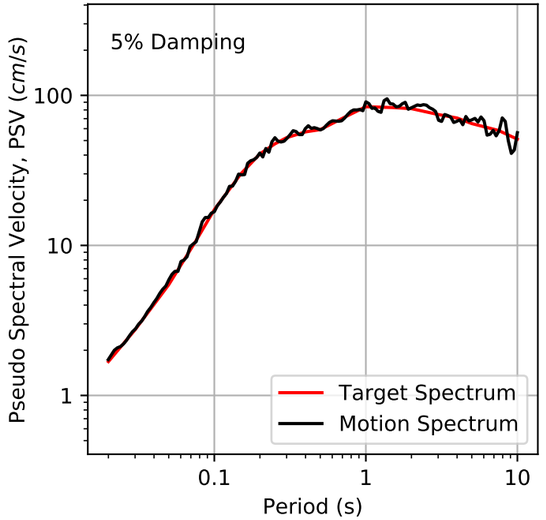

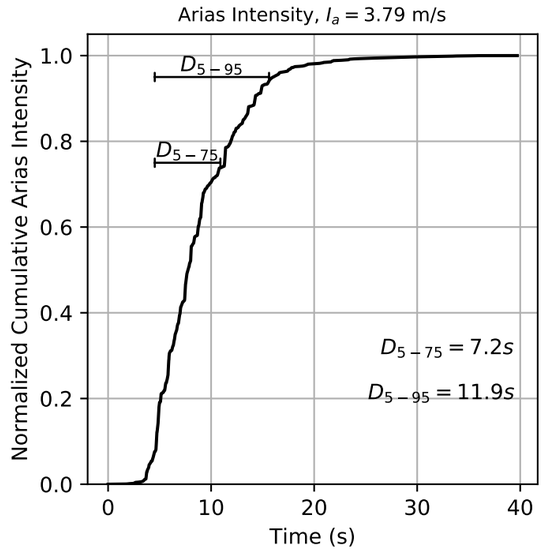

Notes:
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5. s = seconds
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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

LOMA PRIETA 1989
ANDERSON DAM (DS), 250°
SEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-11

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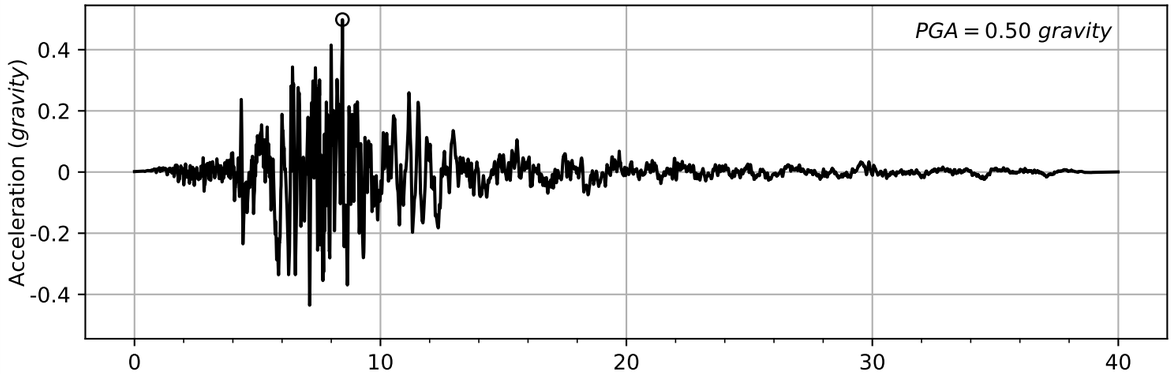

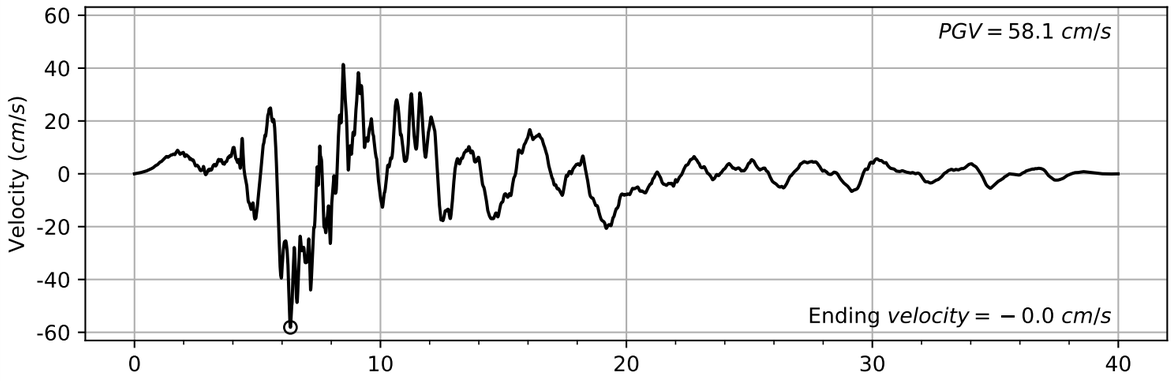

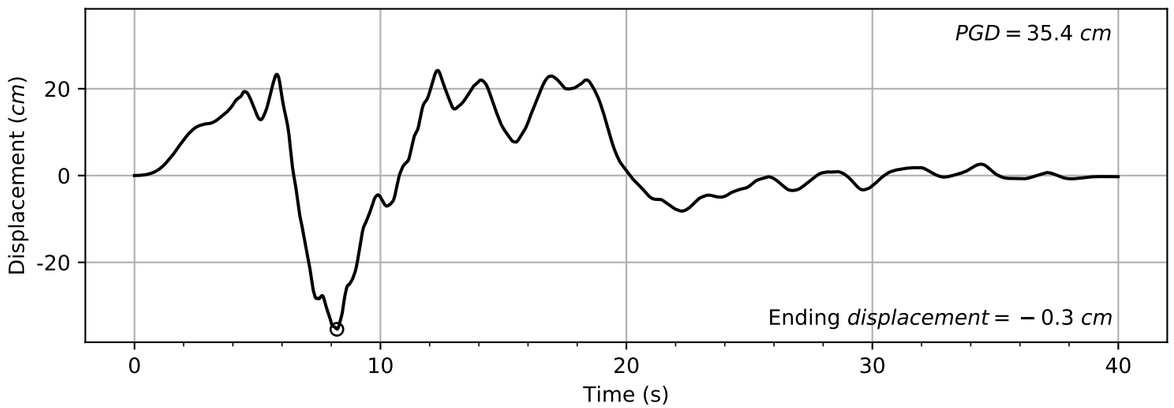

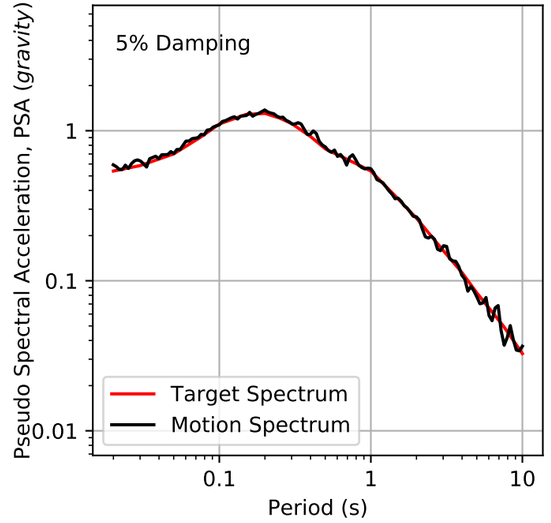

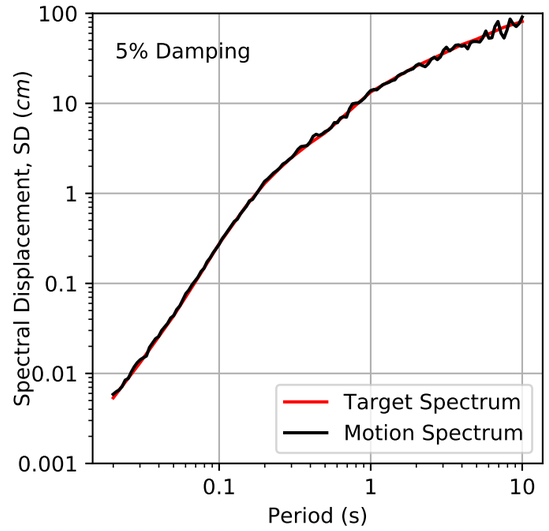

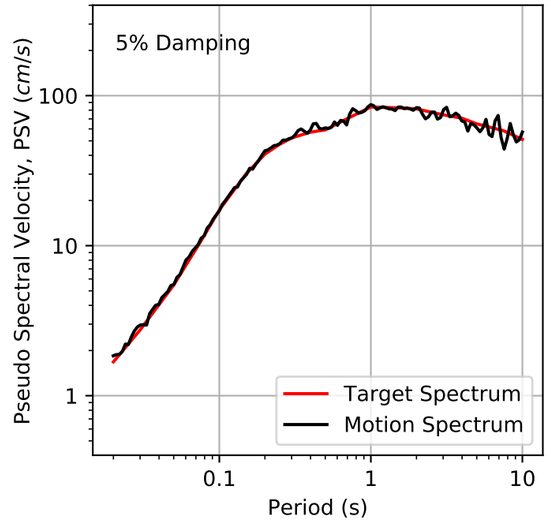

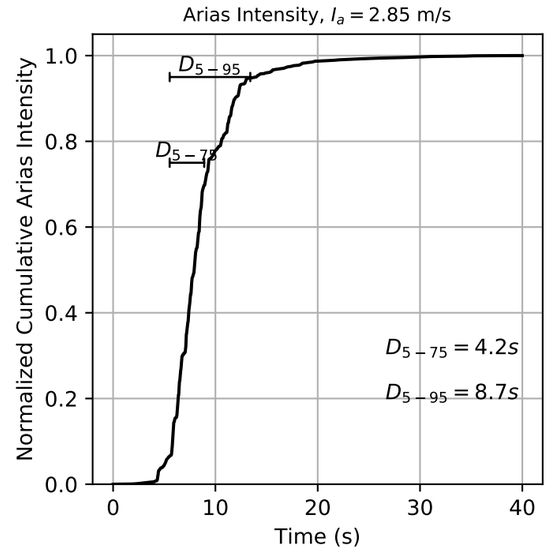

Notes:
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2. m/sec = meters per second
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7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

LOMA PRIETA 1989
SARATOGA - ALOHA AVE, 90°
SEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-12

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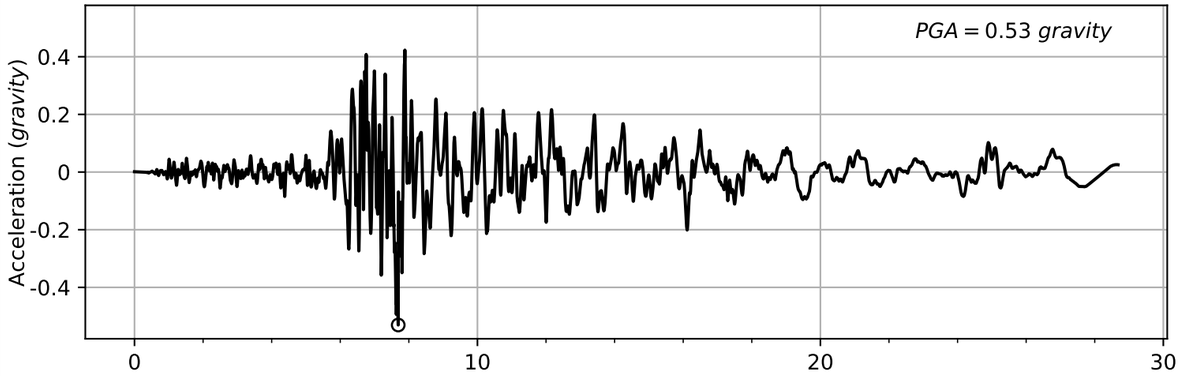

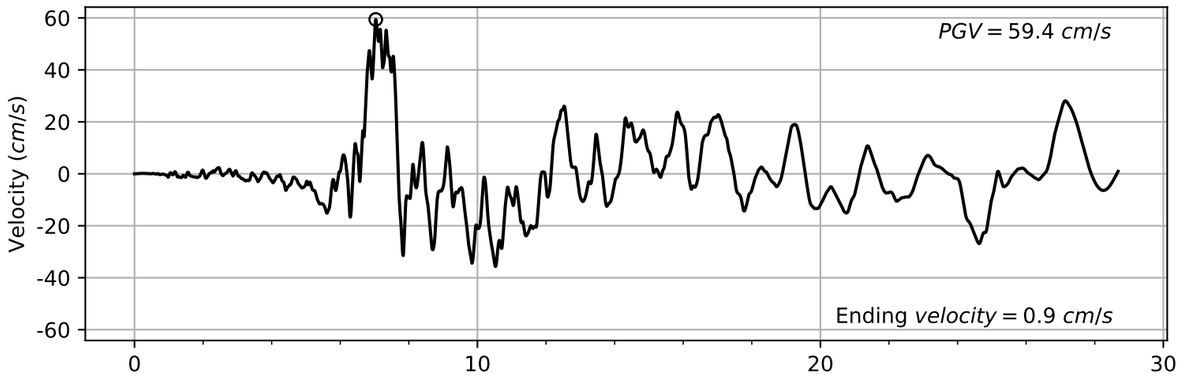

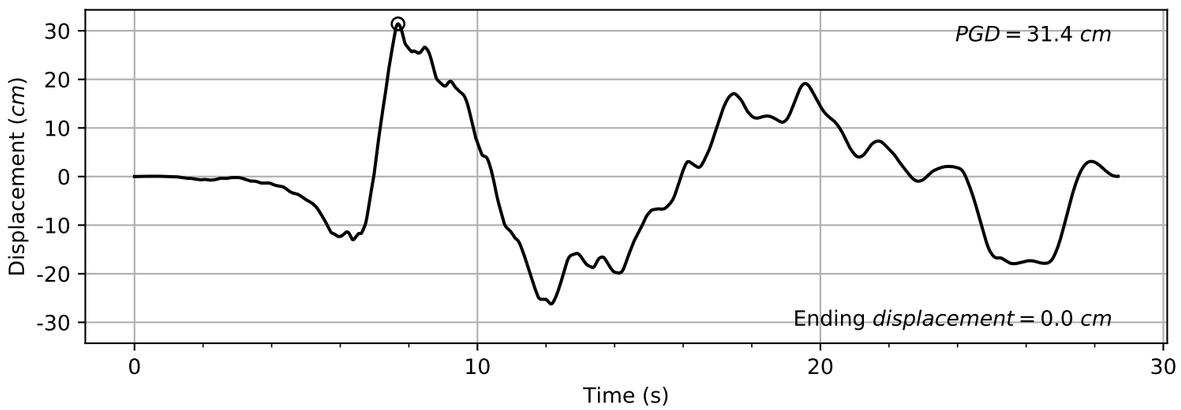

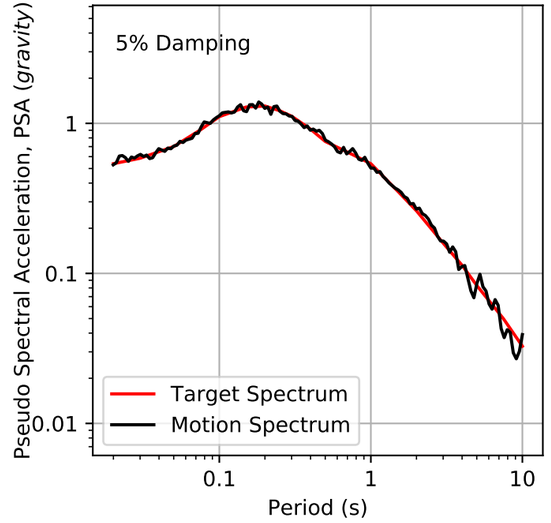

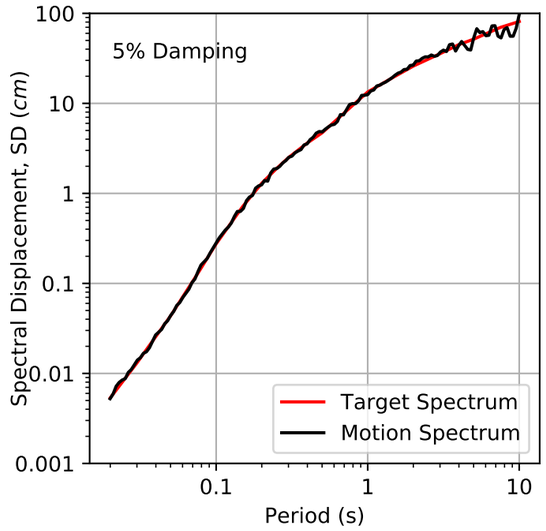

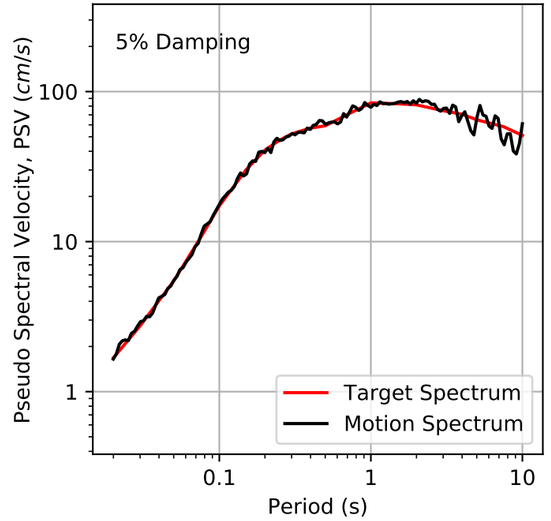

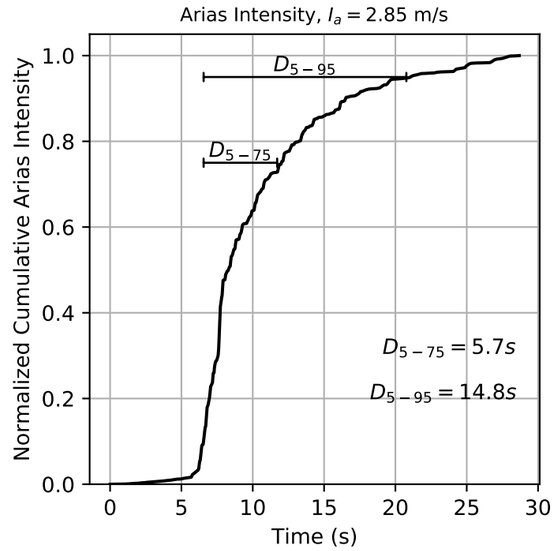

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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

CAPE MENDOCINO 1992
LOLETA FIRE STATION, 360°
SEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-13

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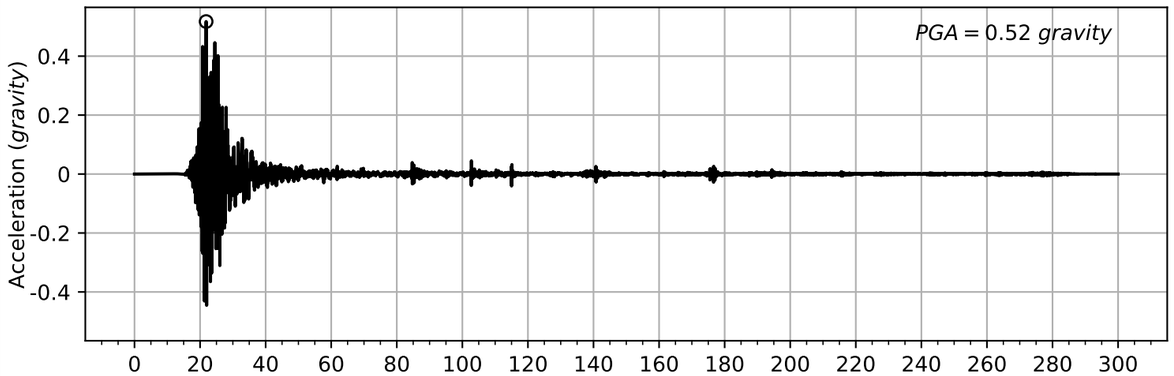

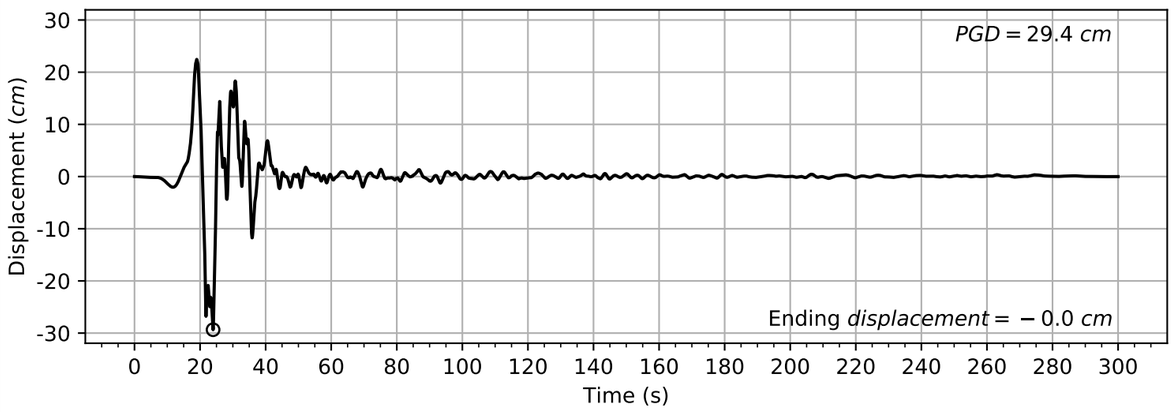

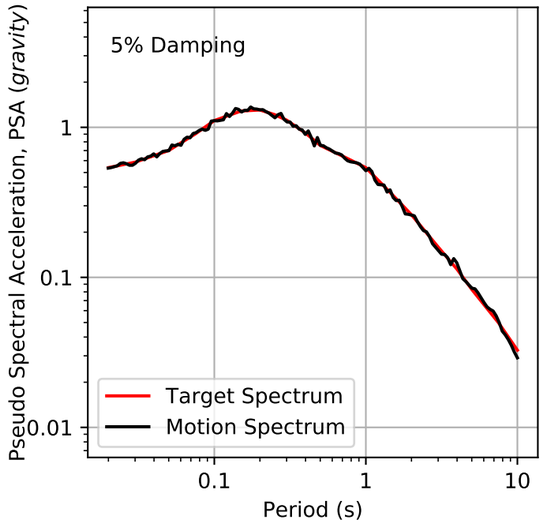

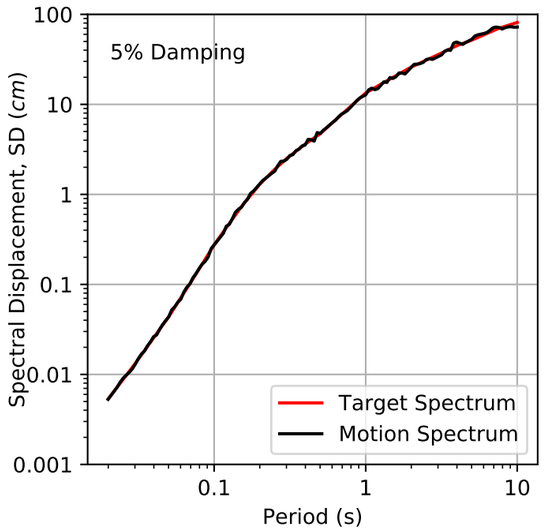

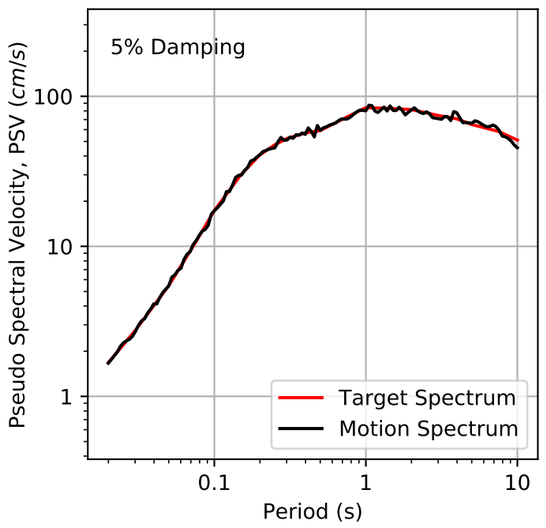

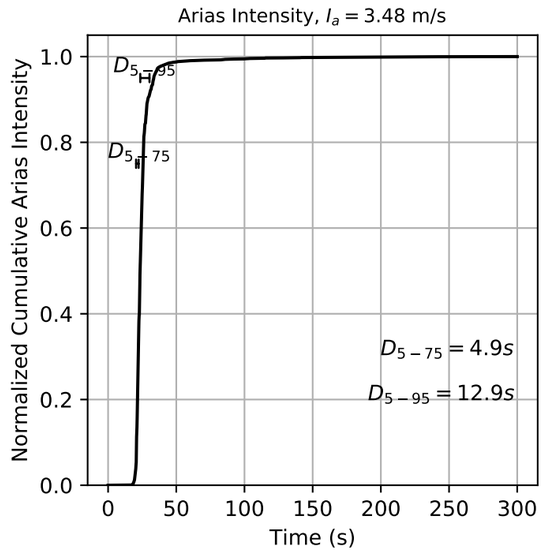

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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

IWATE, JAPAN 2008
MYGH02, NS
SEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-14

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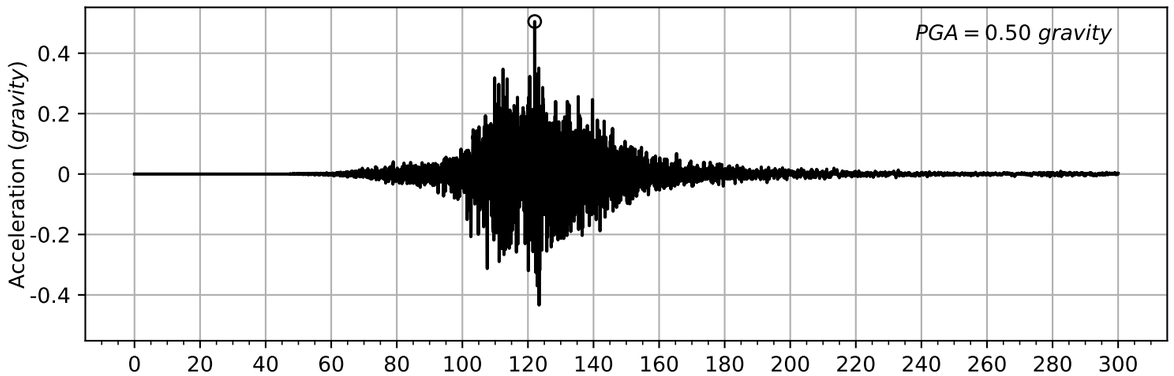

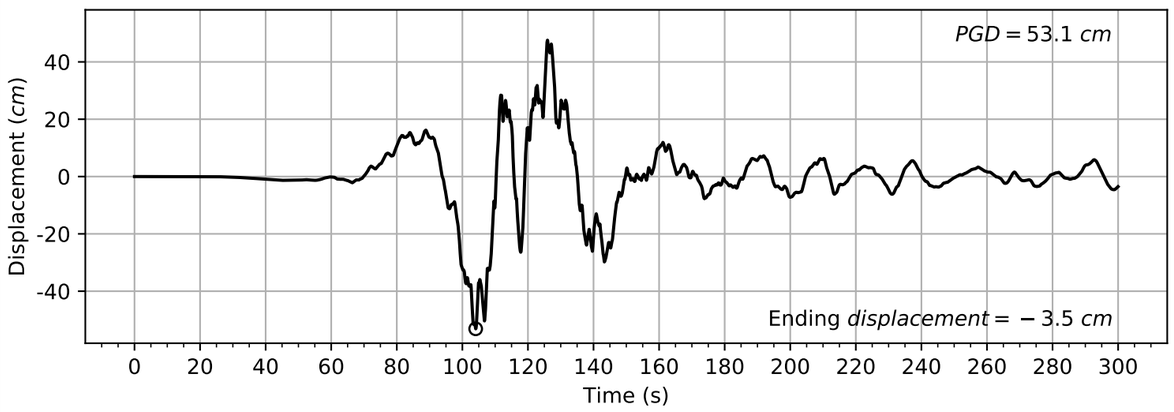

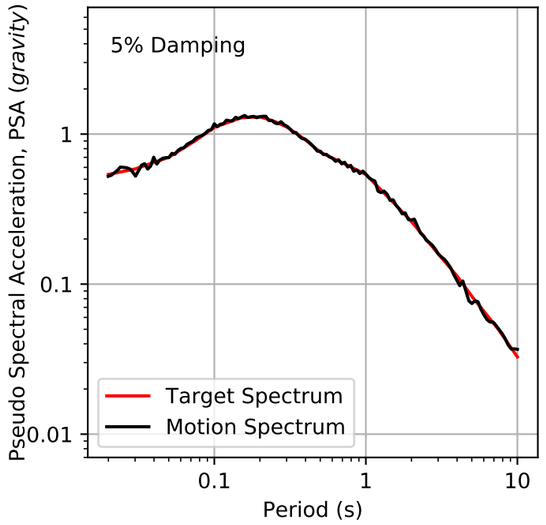

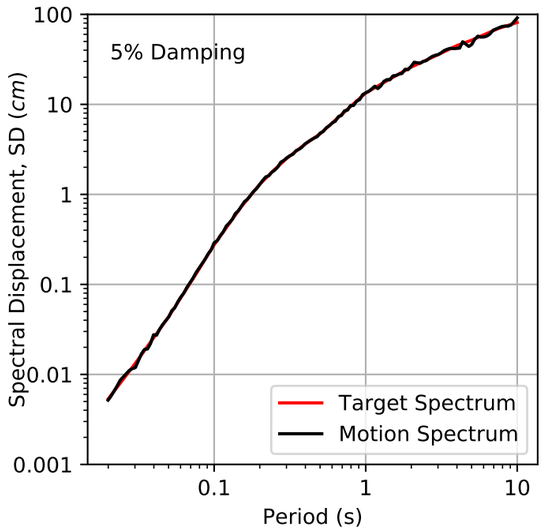

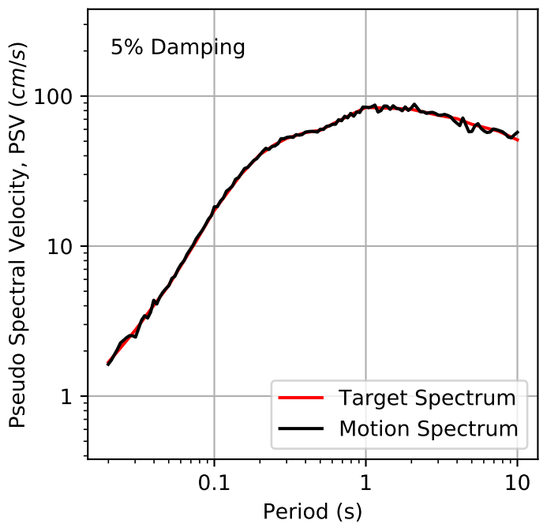

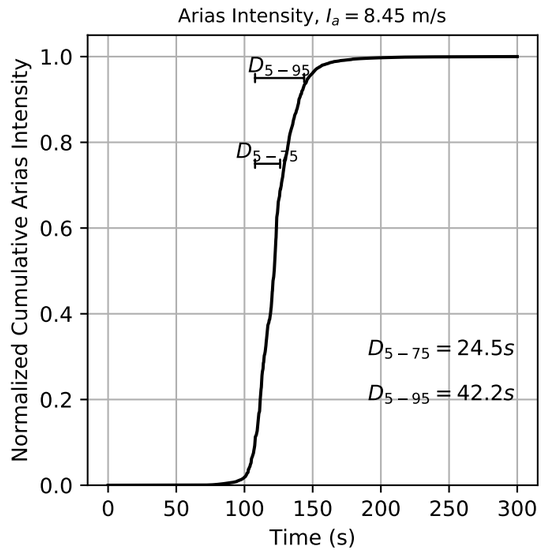

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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

MAULE, CHILE 2010
CIEN AGRONOMICAS, 360°
SEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-15

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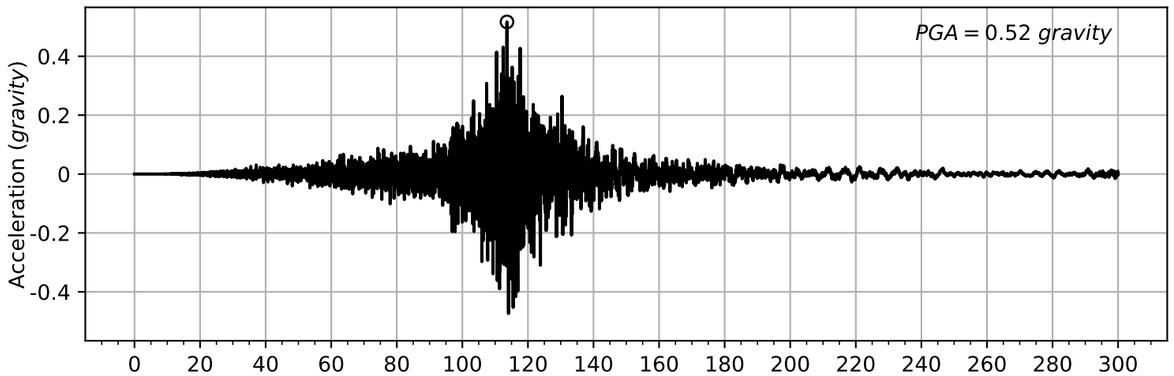

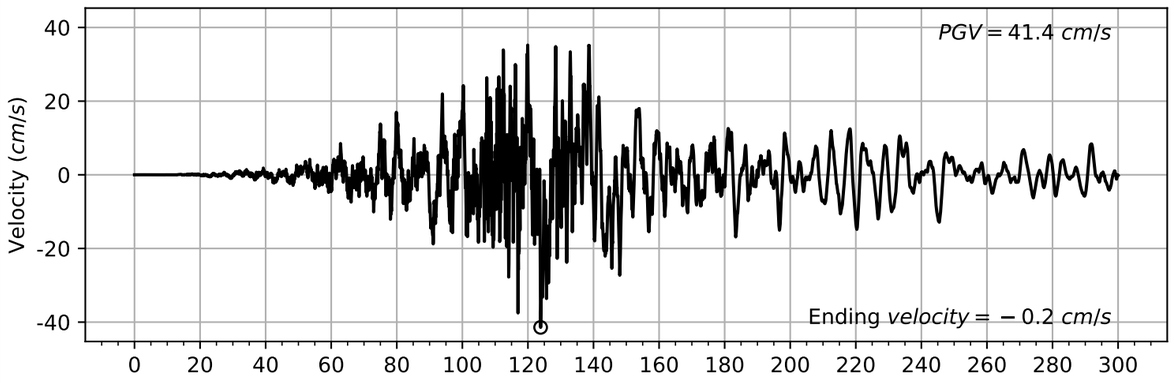

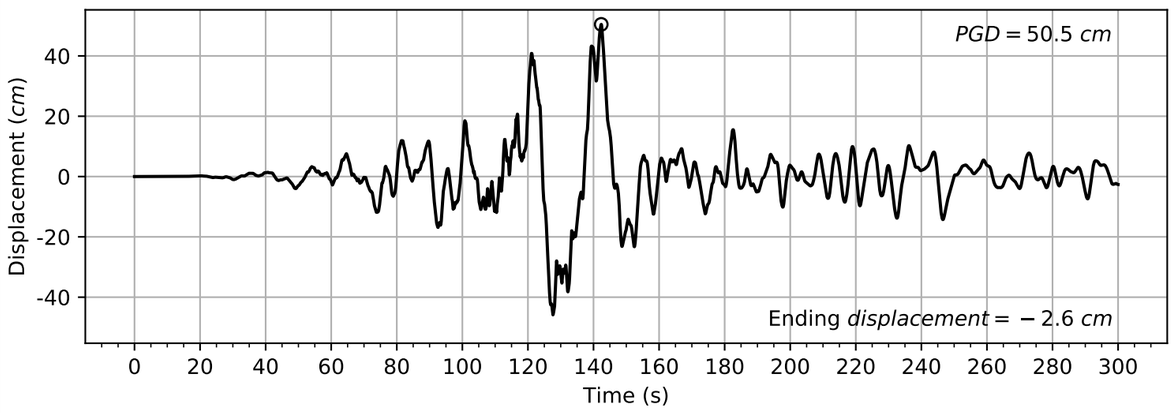

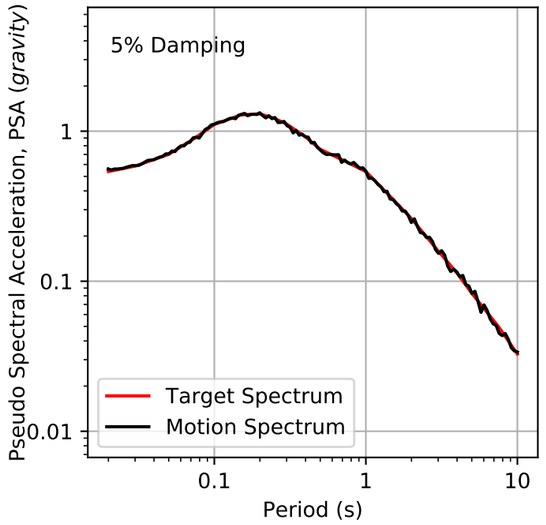

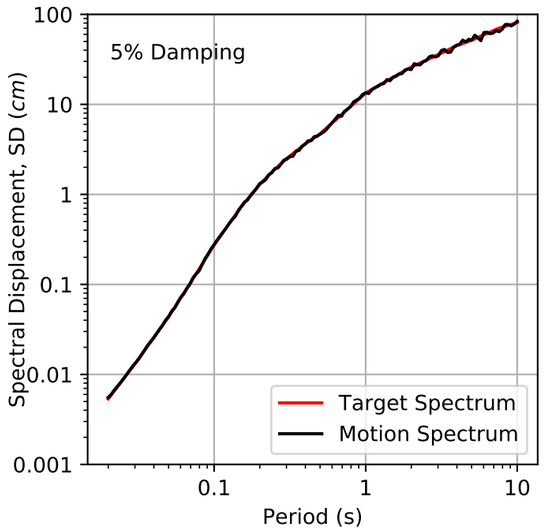

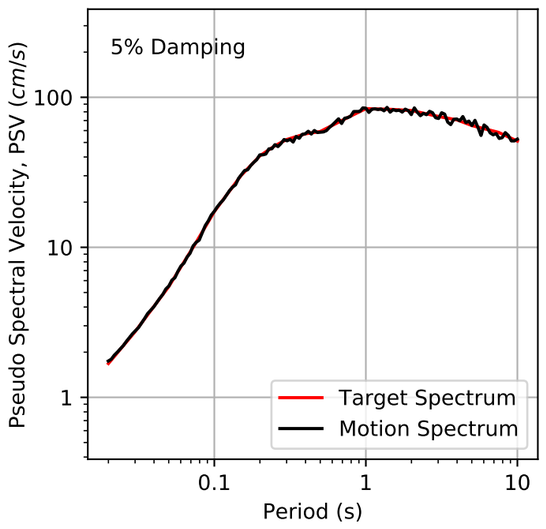

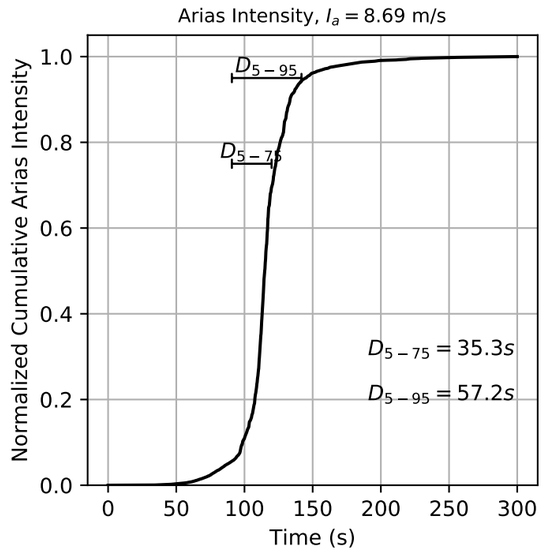

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SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

TOHOKU, JAPAN 2011
SAWARA, NS
SEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-16

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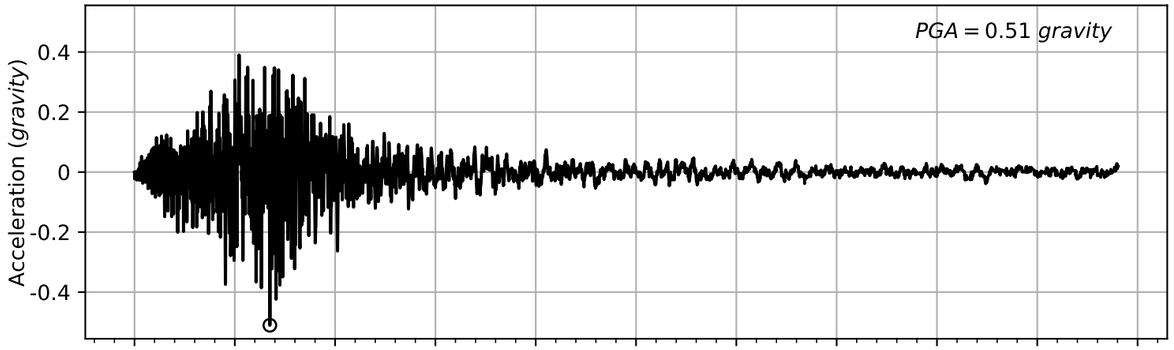

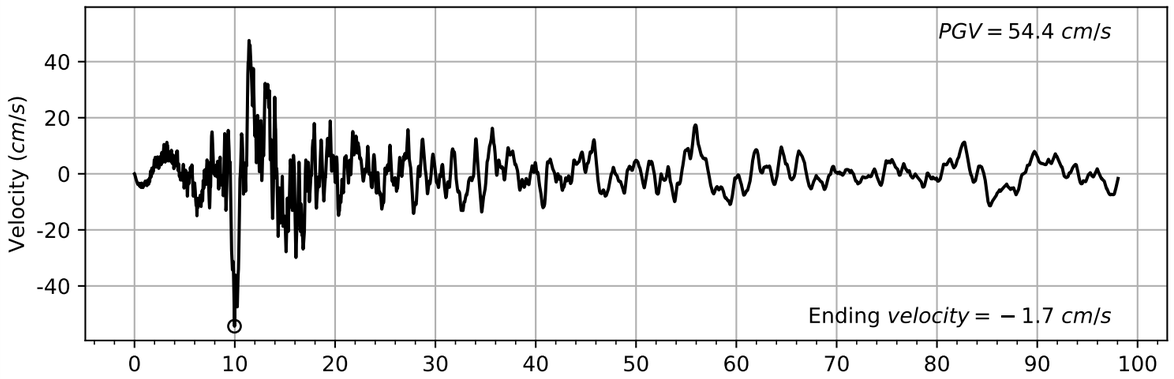

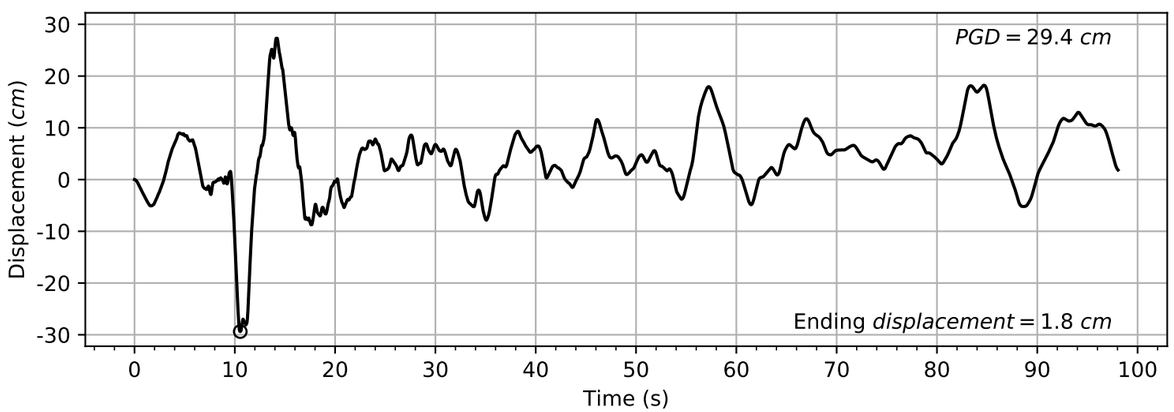

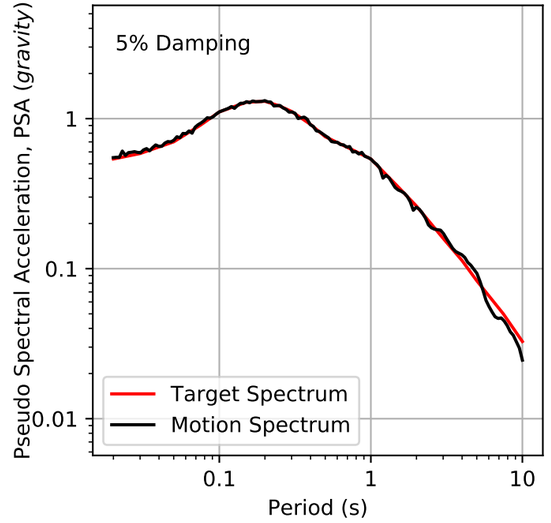

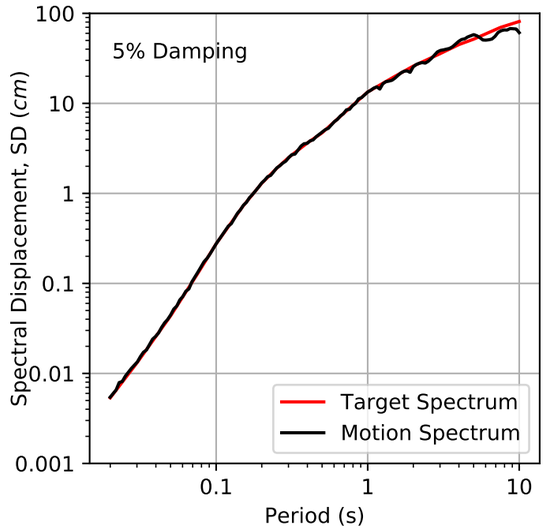

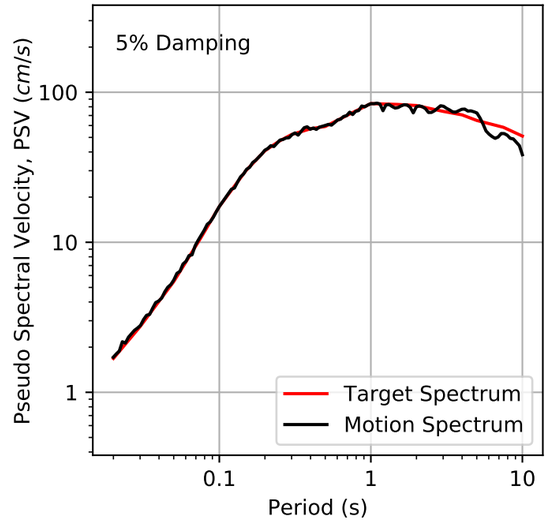

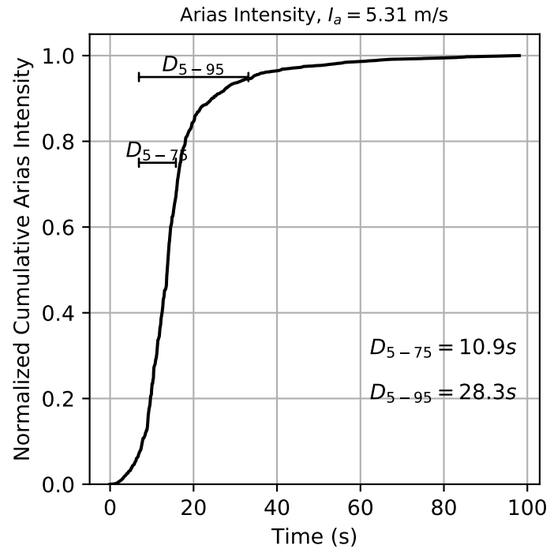

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
3. cm = centimeters
4. cm/s = centimeters per second
5. s = seconds
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7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

NISQUALLY, WASHINGTON 2001
HALVERSON RES., 270°
SEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-17

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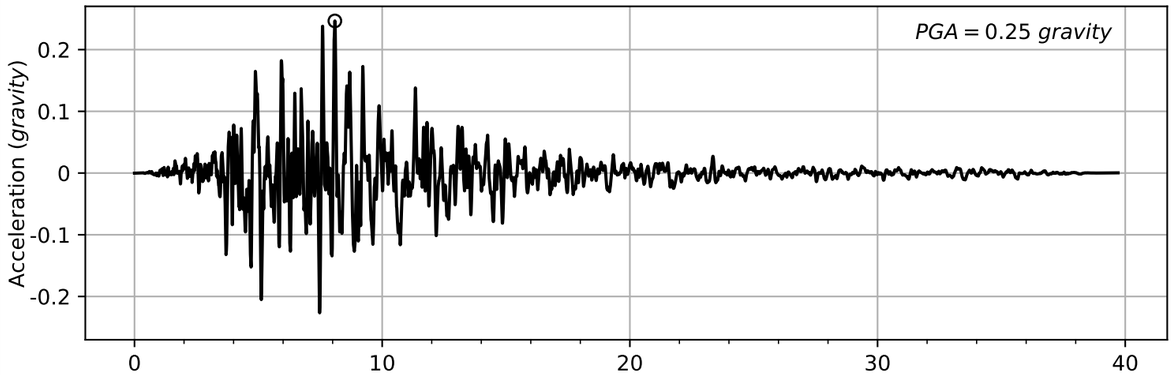

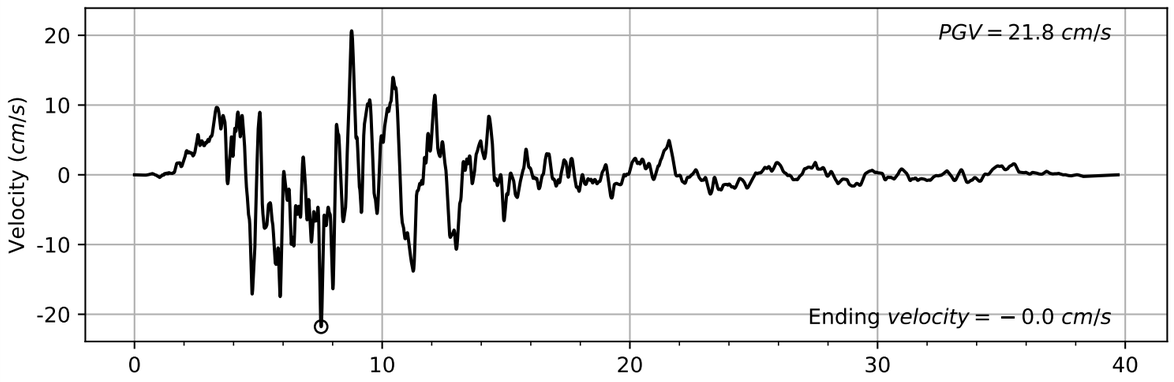

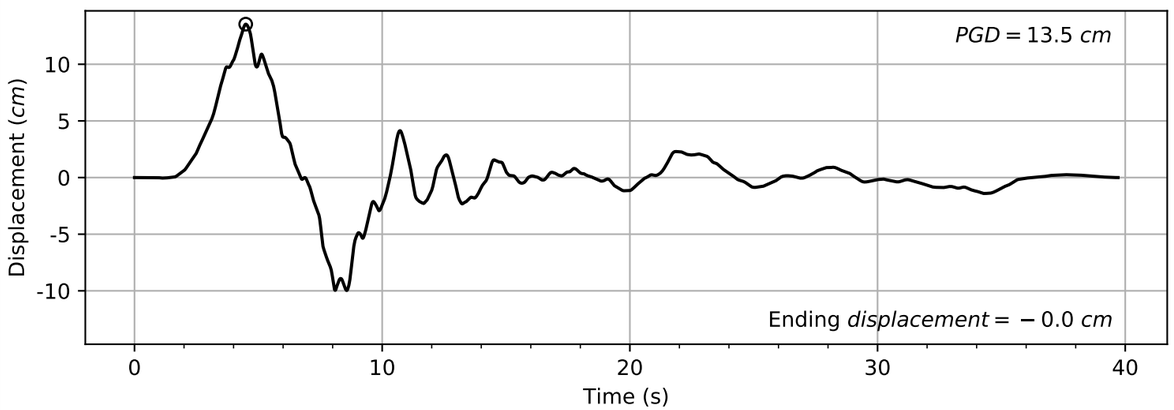

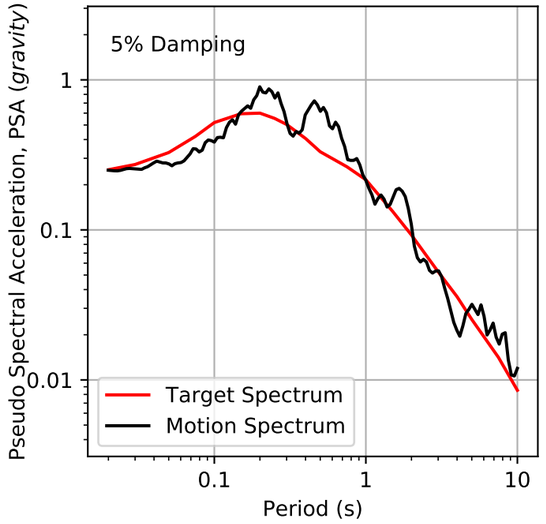

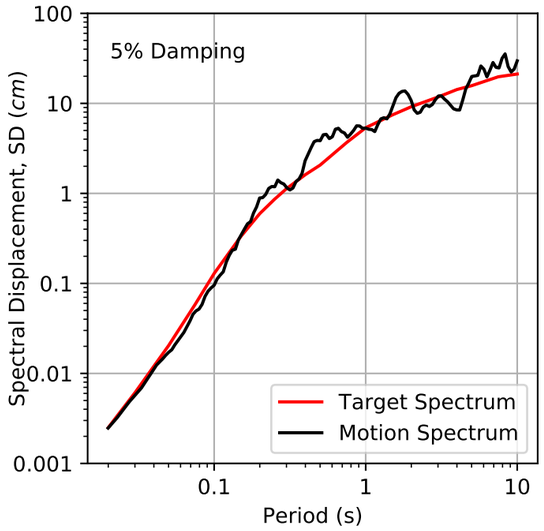

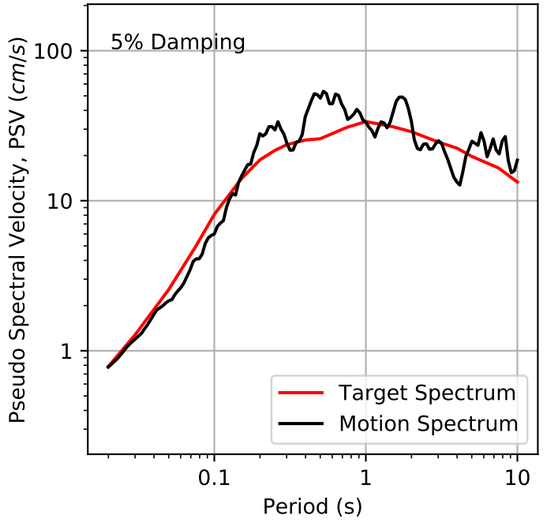

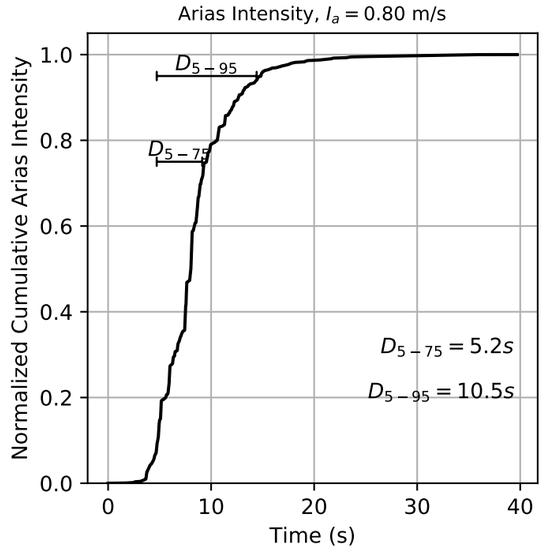

Notes:
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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

LOMA PRIETA 1989
ANDERSON DAM (DS), 250°
FEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-18

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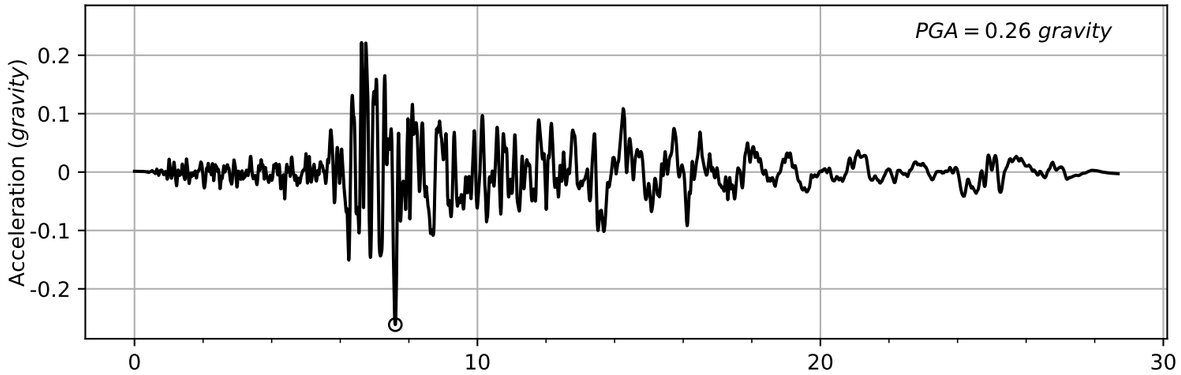

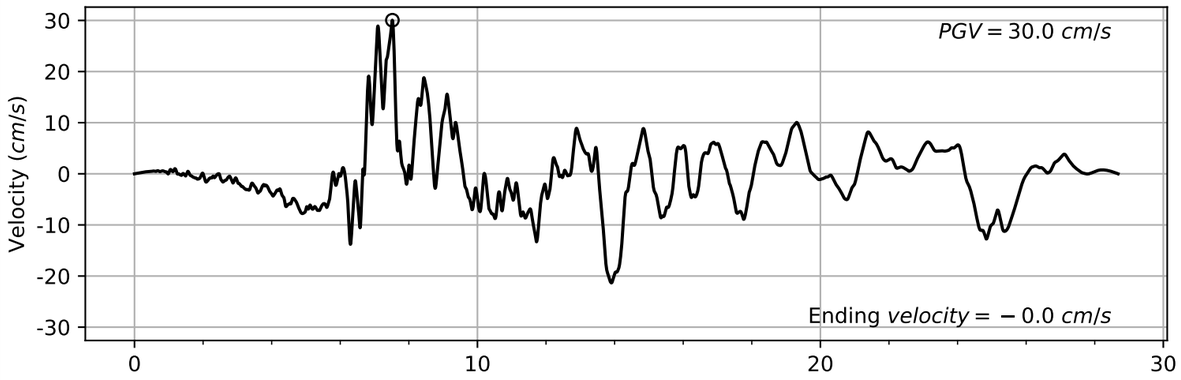

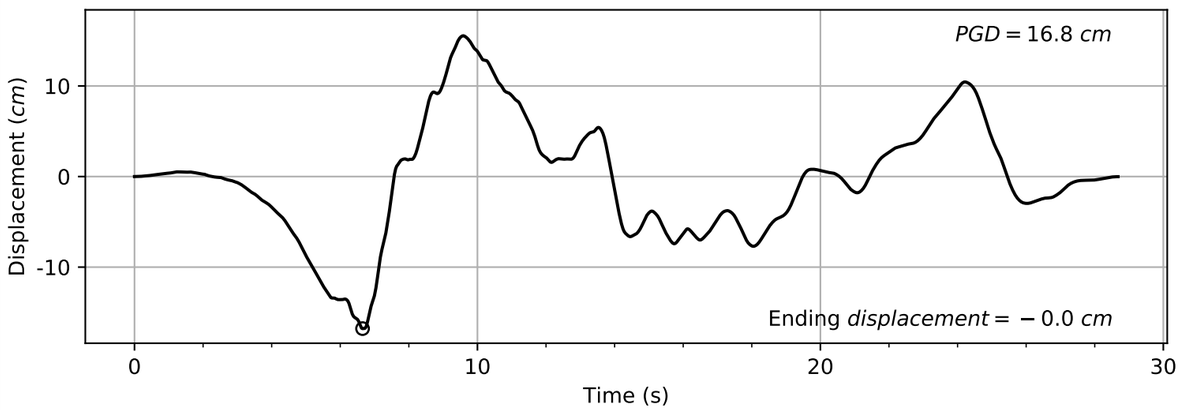

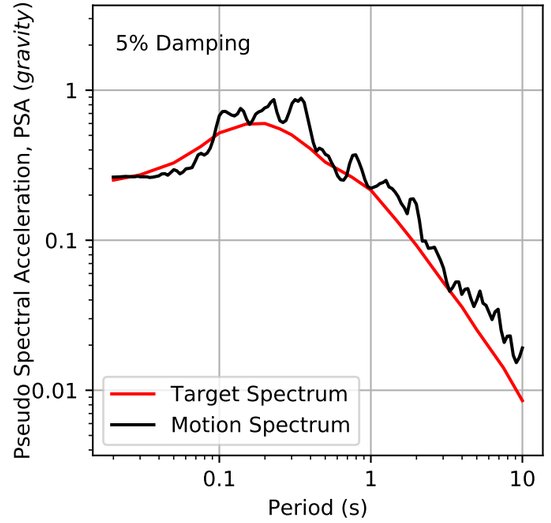

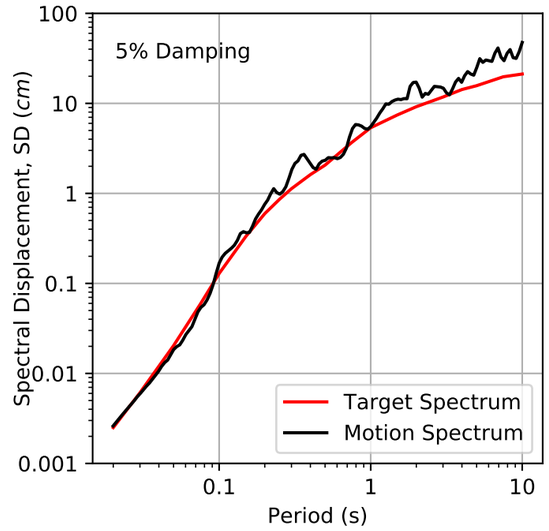

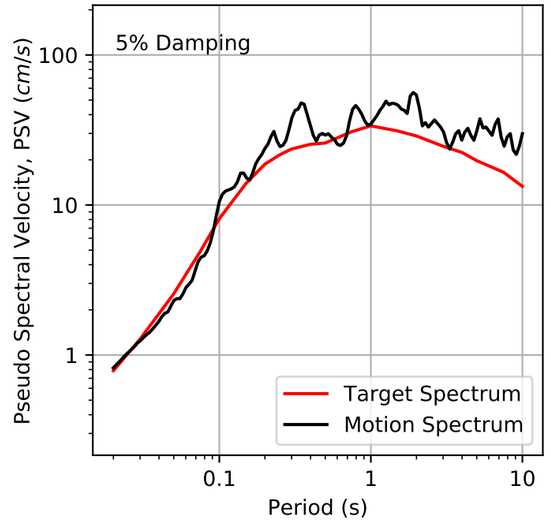

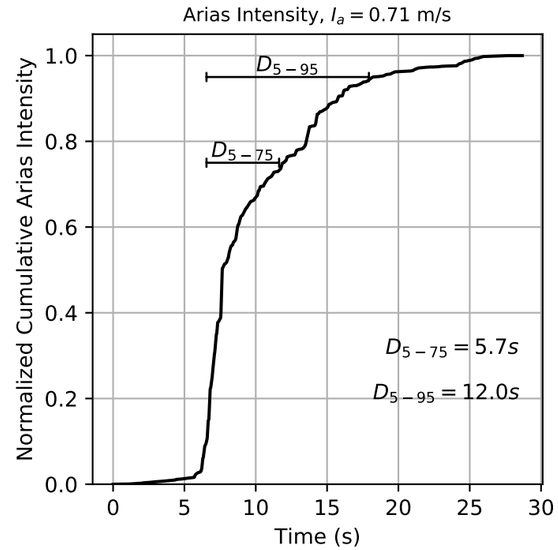

Notes:
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2. m/sec = meters per second
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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

CAPE MENDOCINO 1992
LOLETA FIRE STATION, 360°
FEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-19

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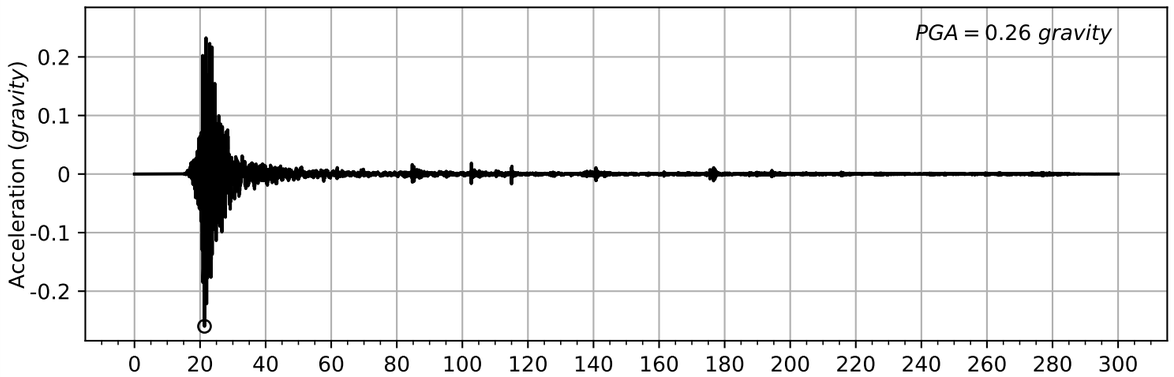

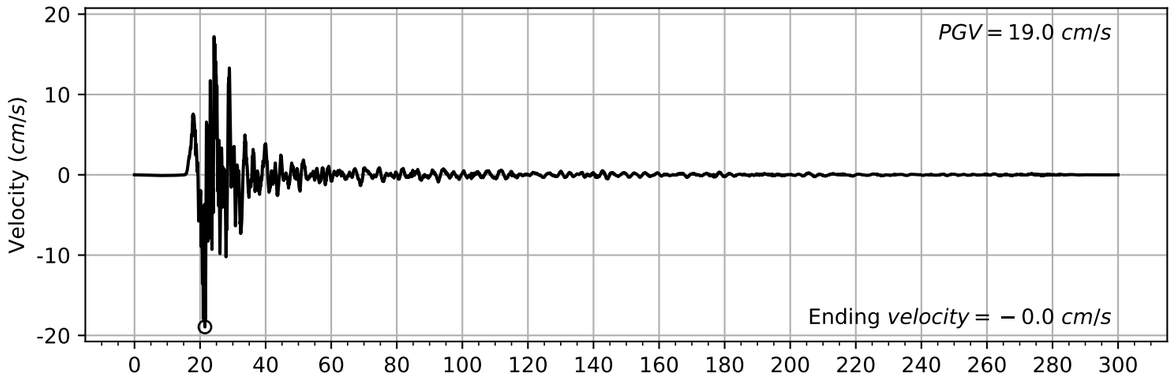

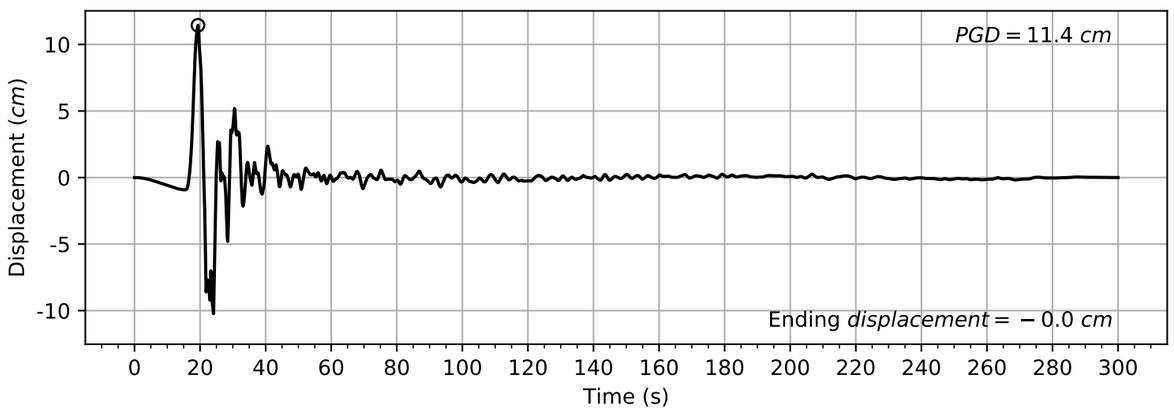

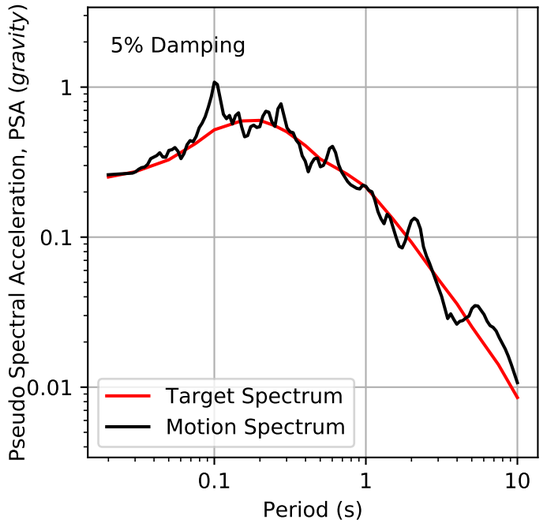

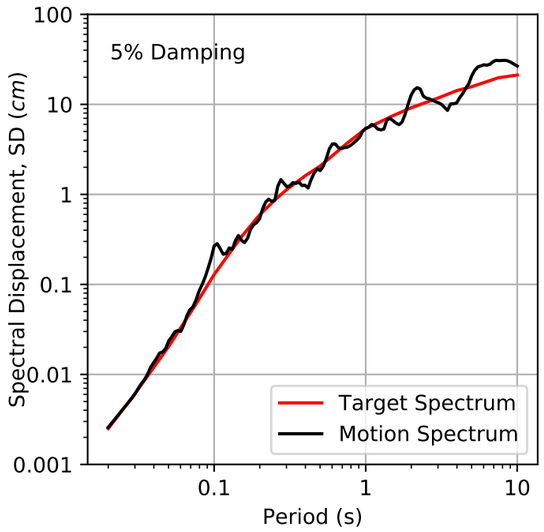

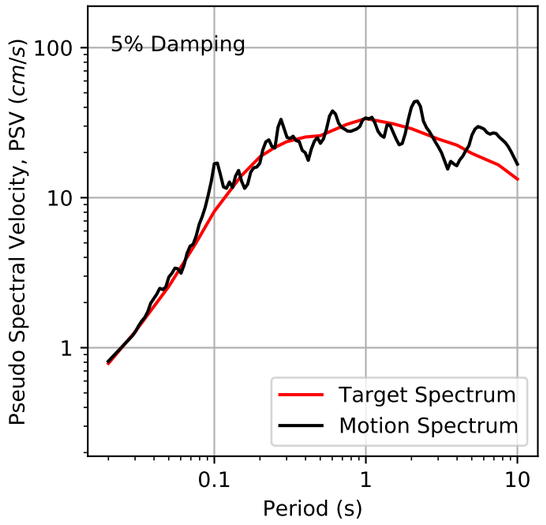

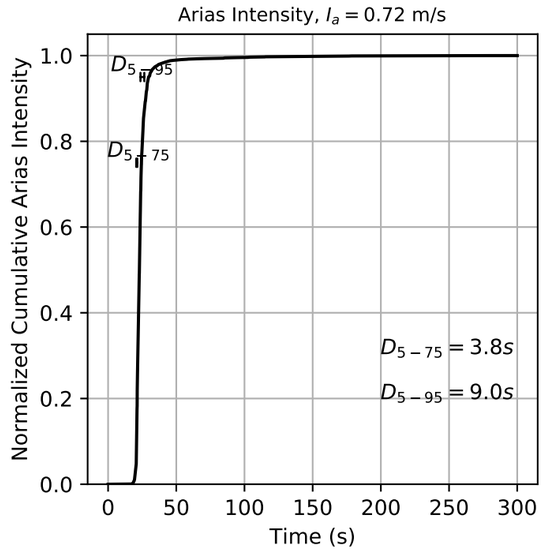

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2. m/sec = meters per second
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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

IWATE, JAPAN 2008
MYGH02, NS
FEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-20

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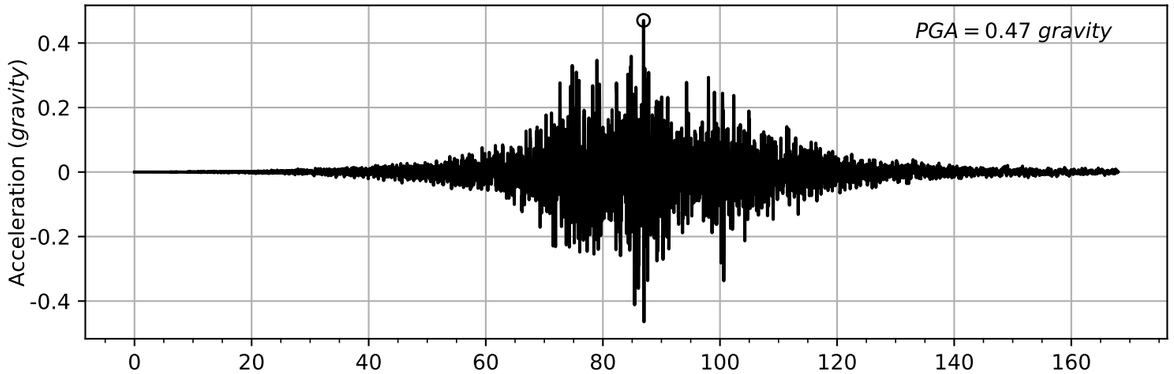

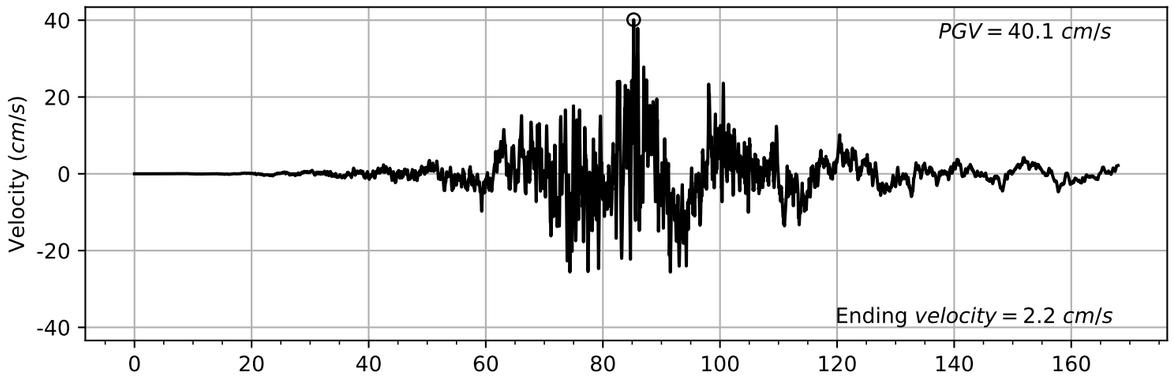

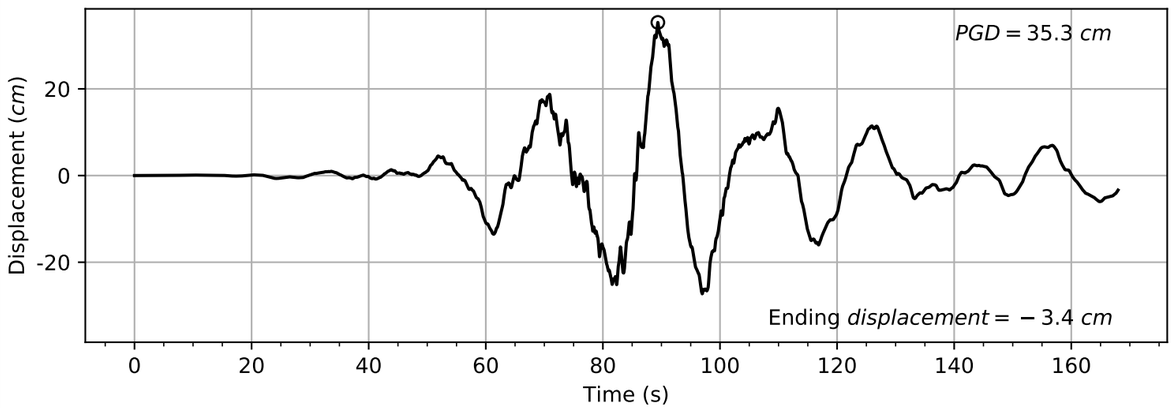

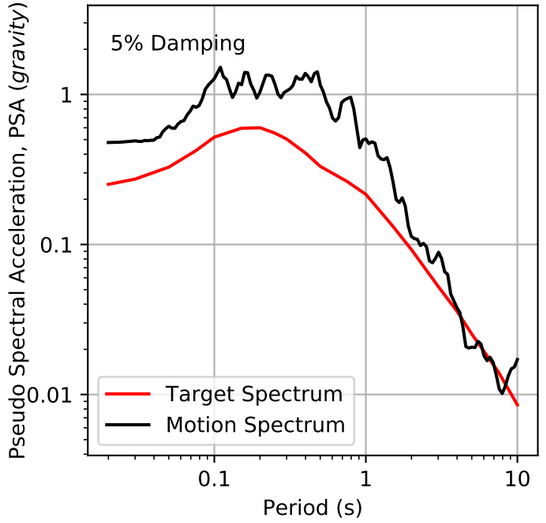

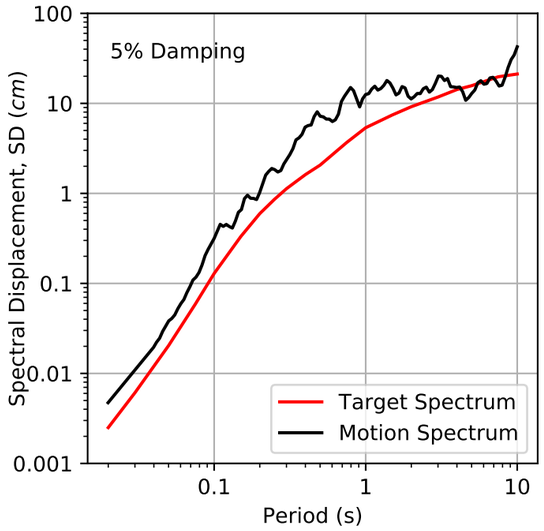

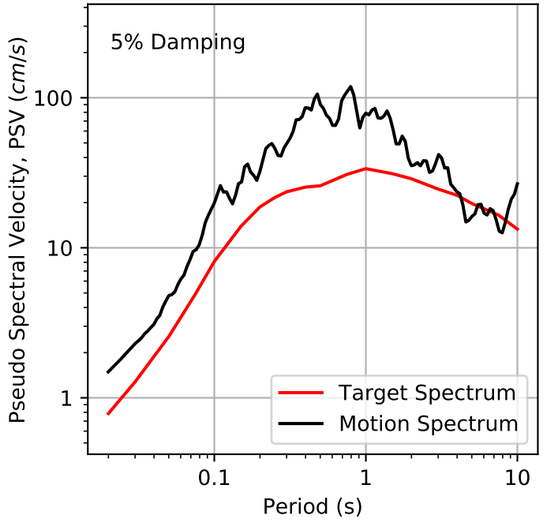

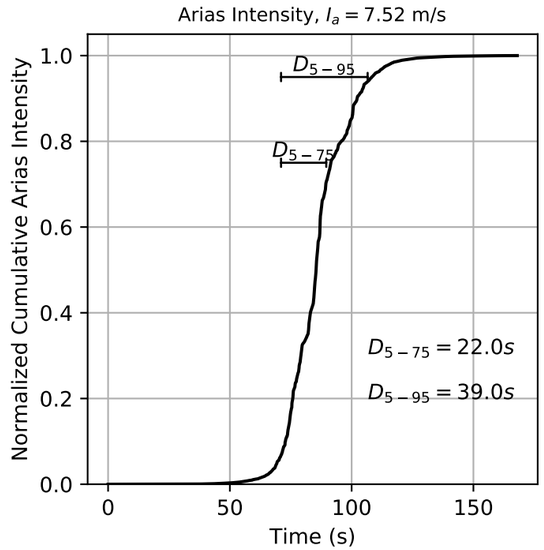

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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

MAULE, CHILE 2010
MUNI. SAN JOSE DE MAIPO, 090°
FEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-21

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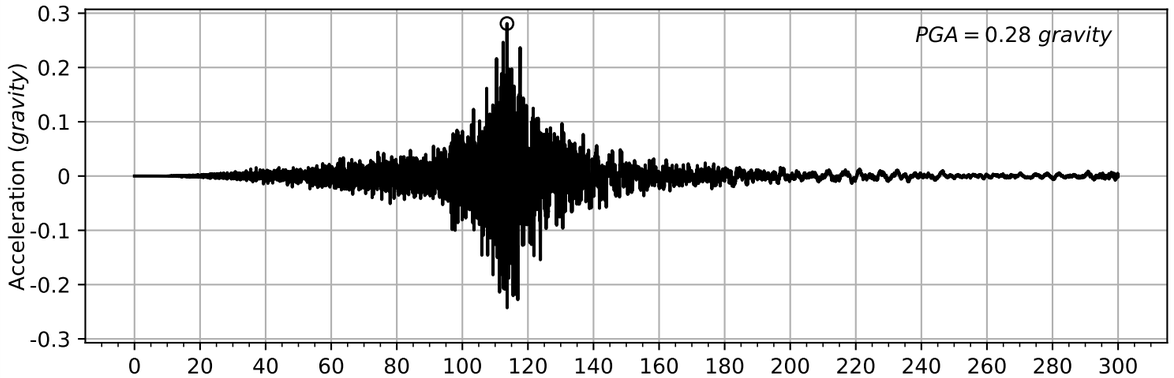

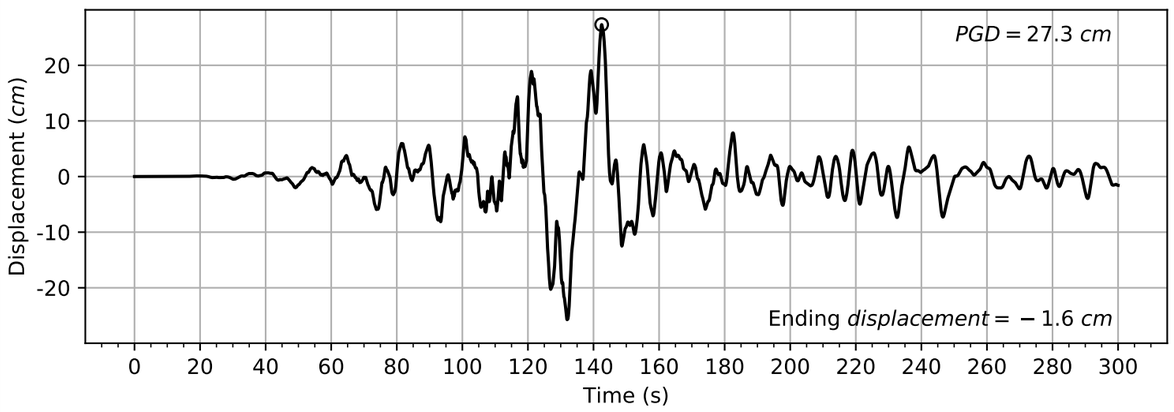

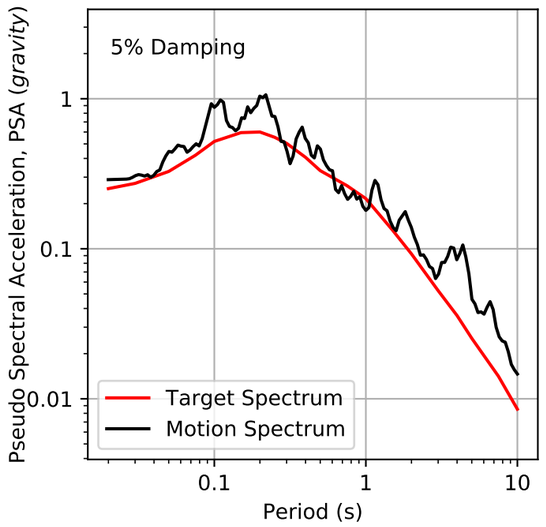

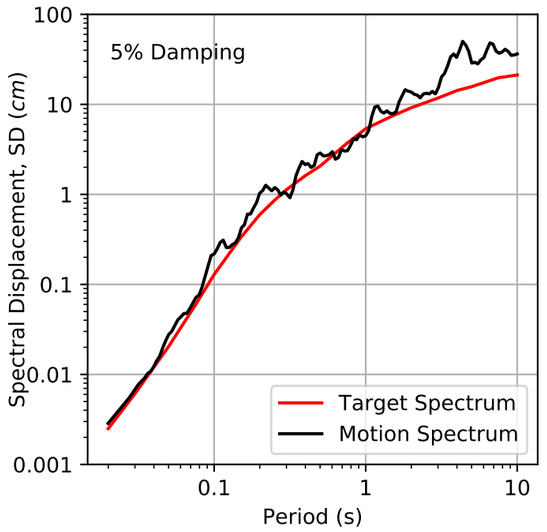

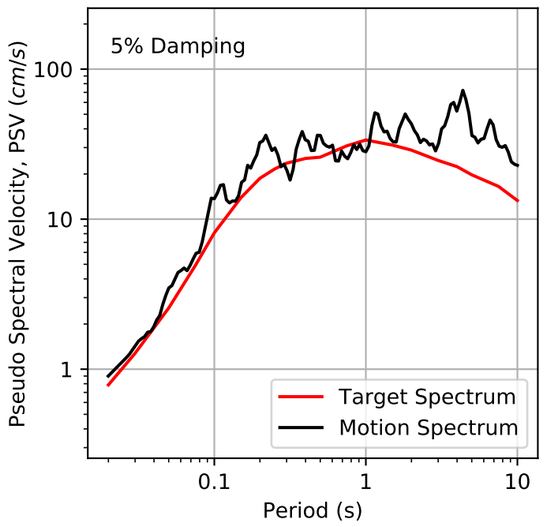

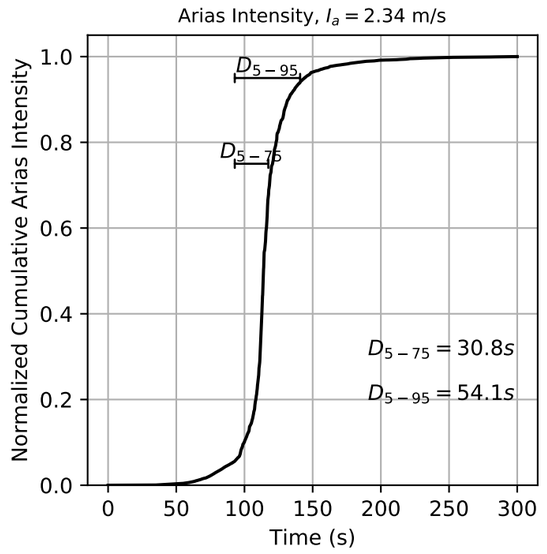

Notes:
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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

TOHOKU, JAPAN 2011
SAWARA, NS
FEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-22

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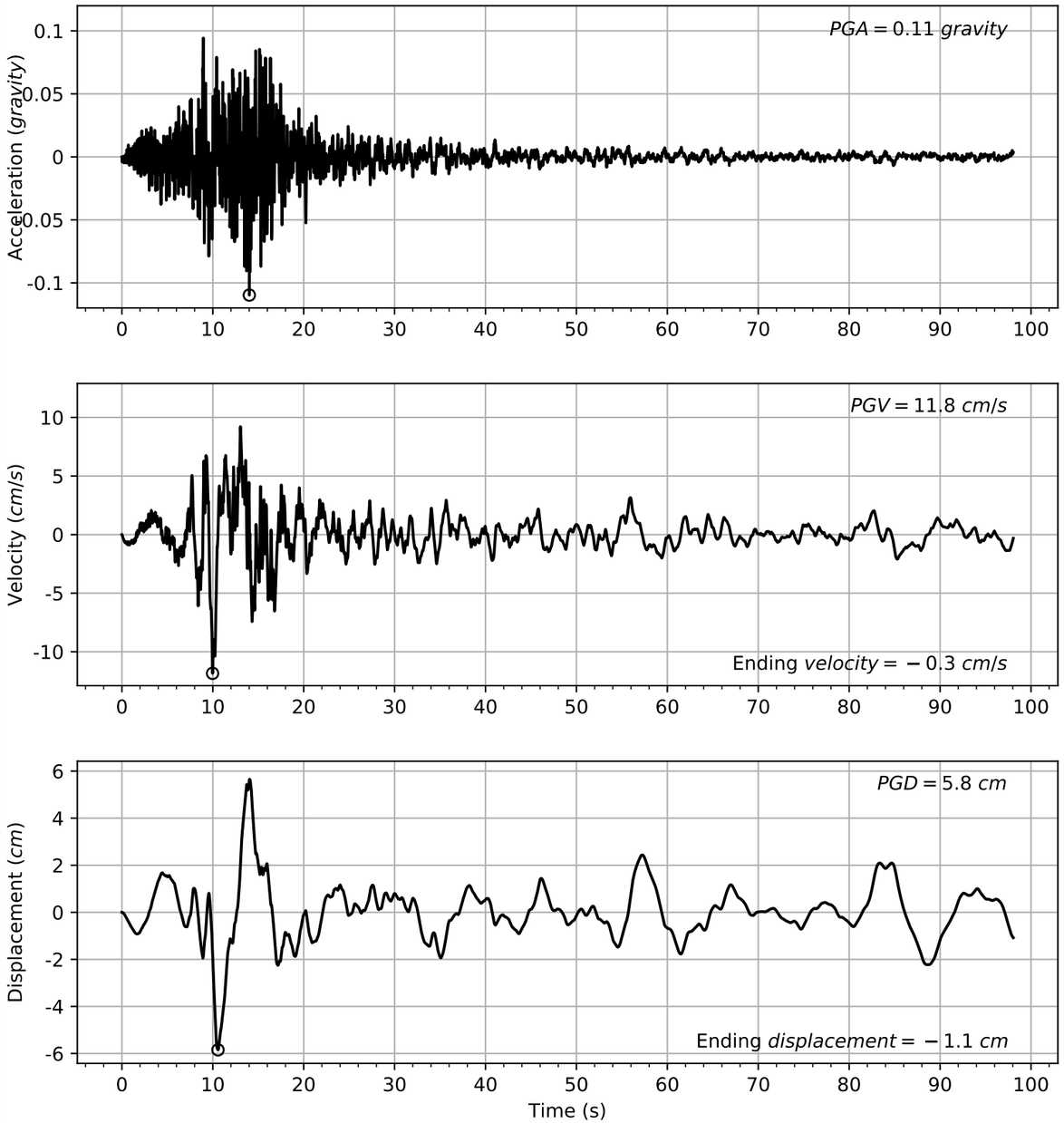

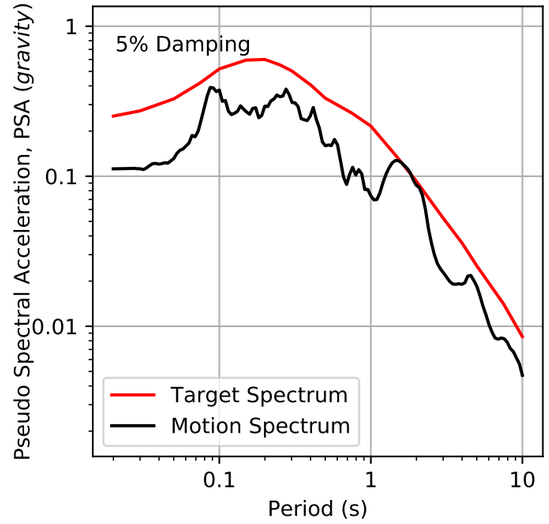

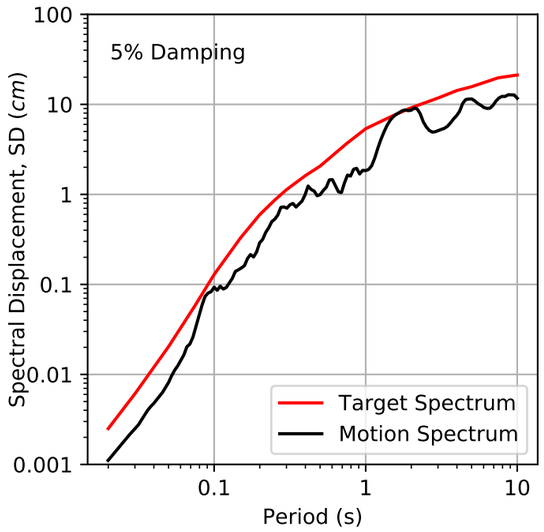

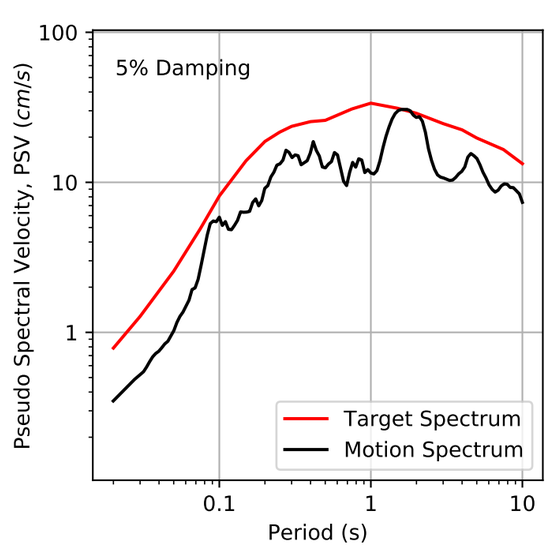

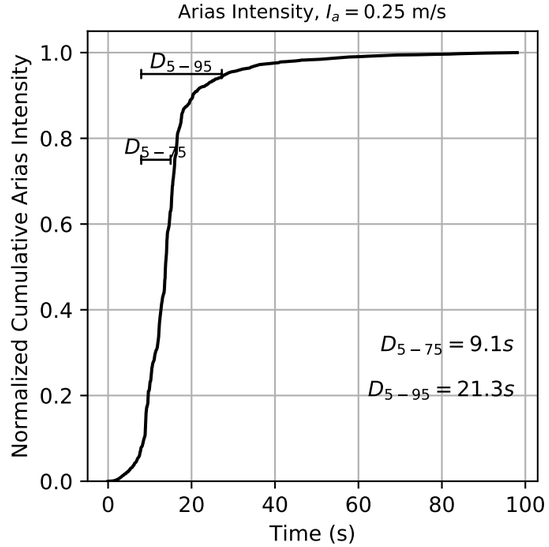

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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

NISQUALLY, WASHINGTON 2001
HALVERSON RES., 270°
FEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-23

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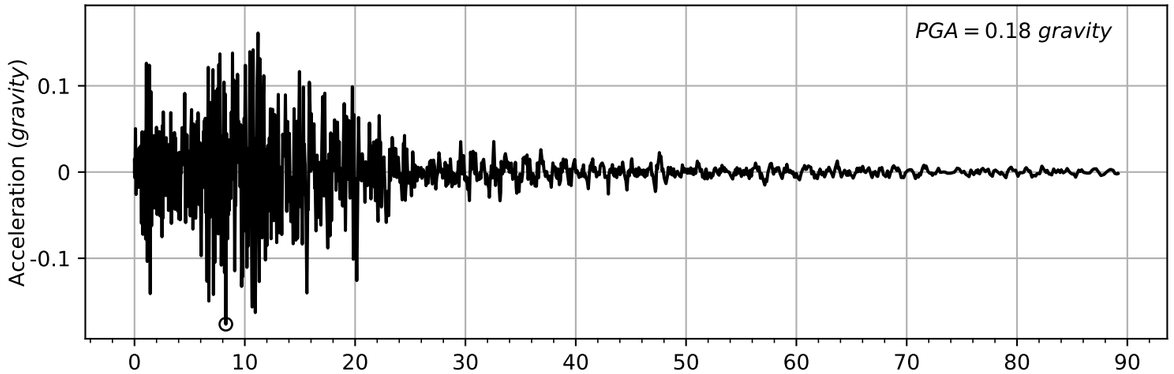

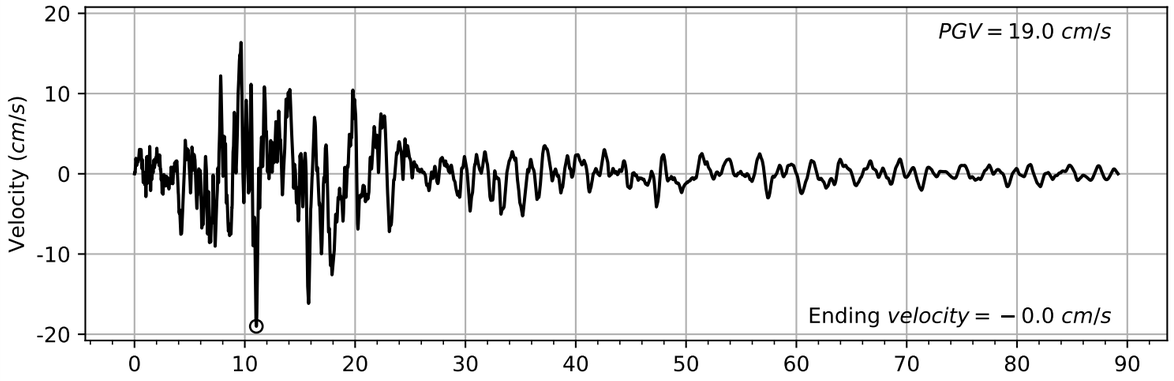

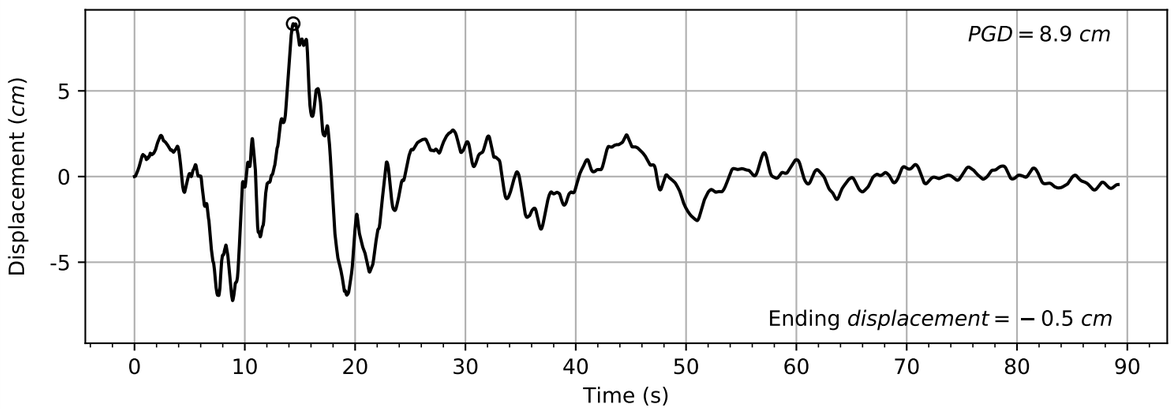

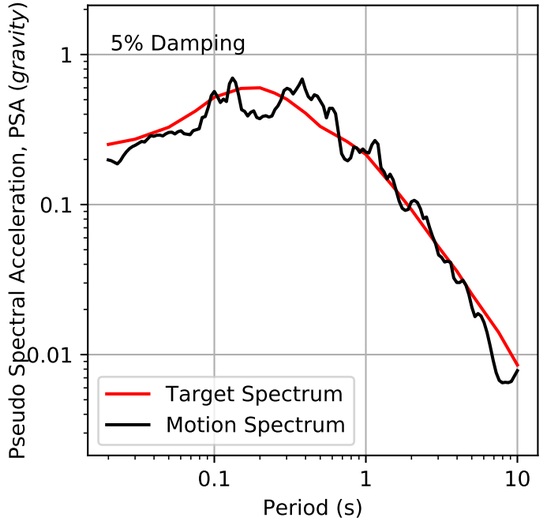

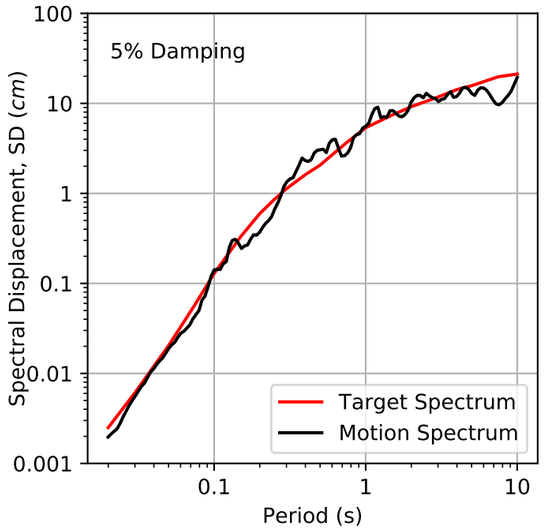

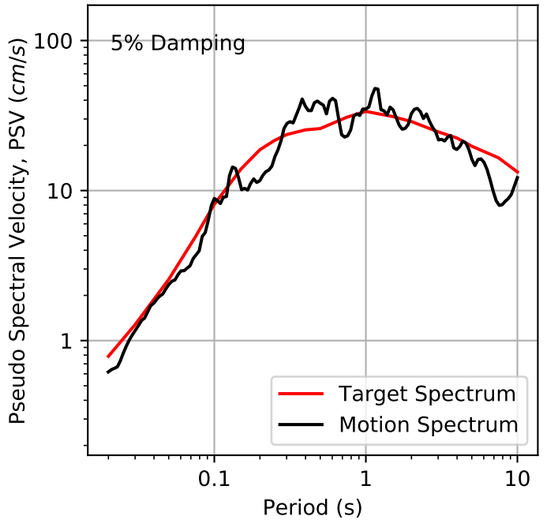

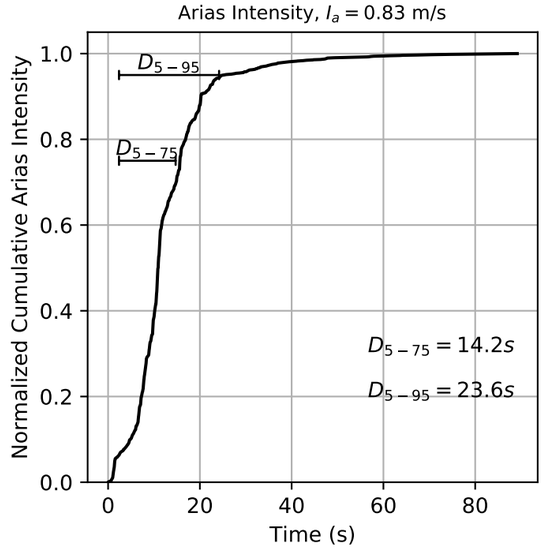

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
3. cm = centimeters
4. cm/s = centimeters per second
5. s = seconds
6. D_{5-75} = significant duration from 5 to 75 percent of normalized cumulative Arias intensity
7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

OLYMPIA, WASHINGTON 1949
OLYMPIA HWY TEST LAB, 356°
FEE REFERENCE TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-24

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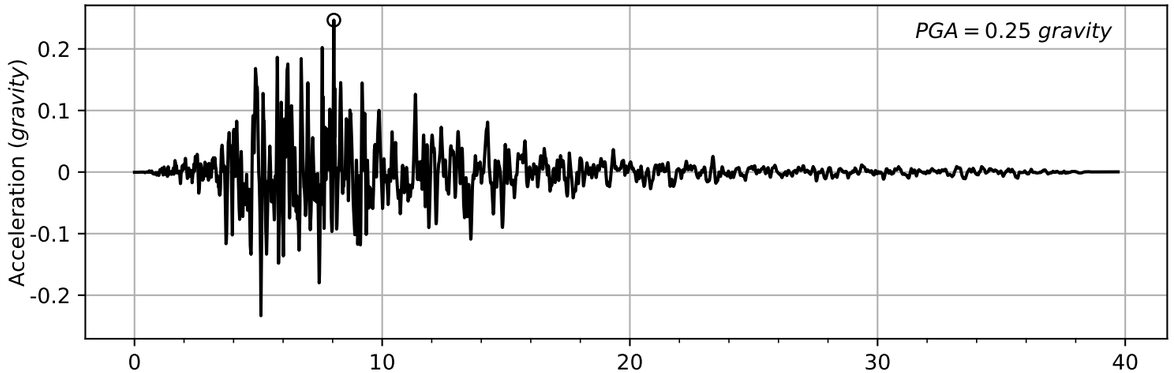

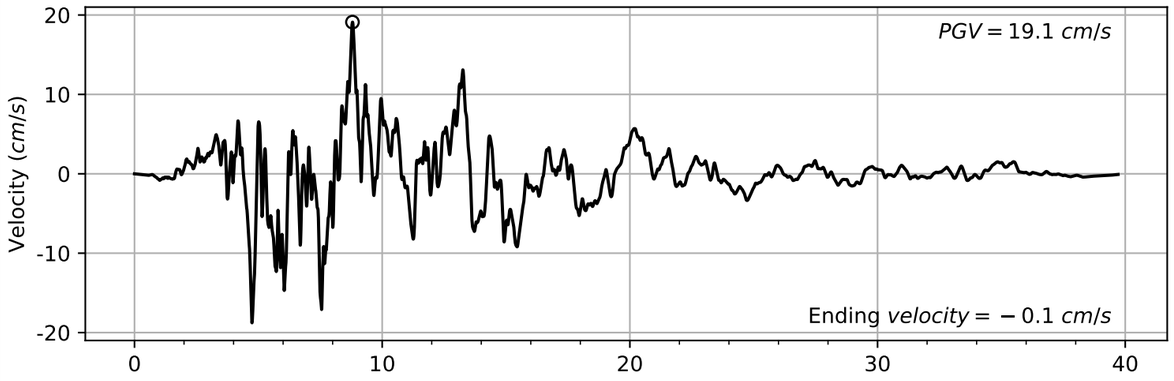

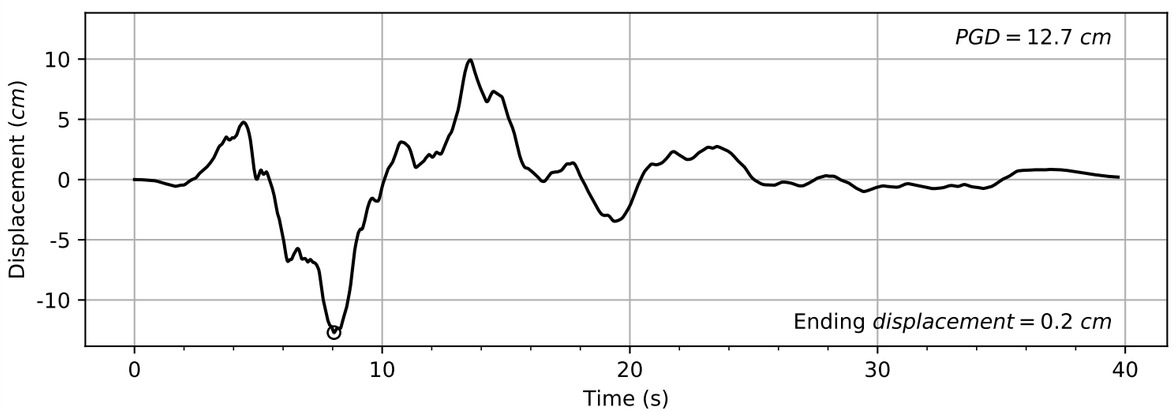

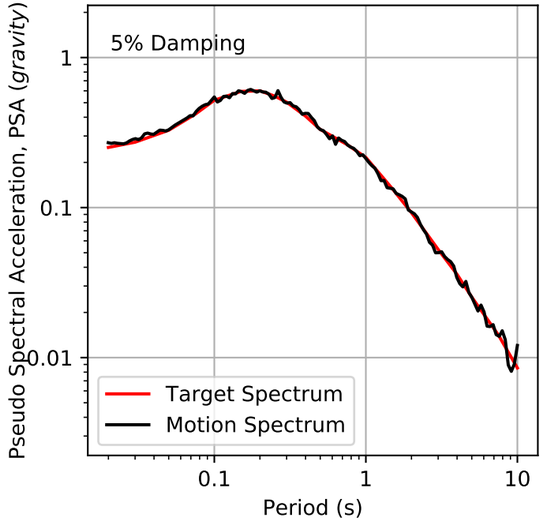

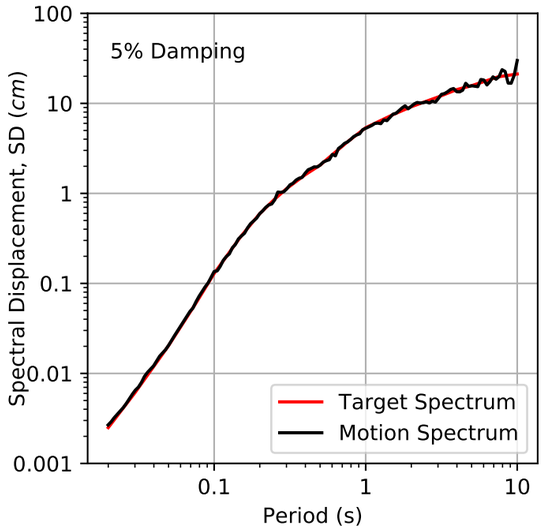

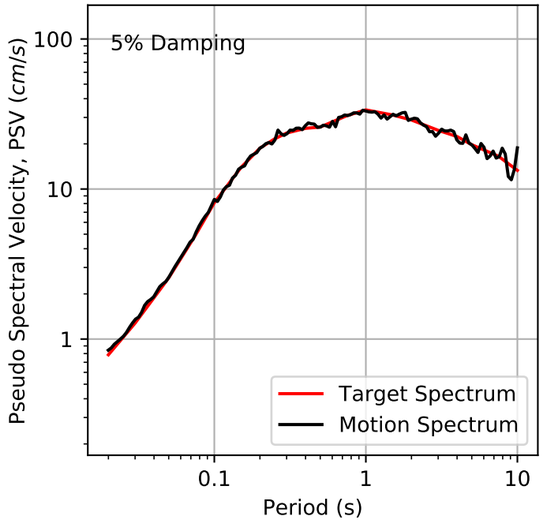

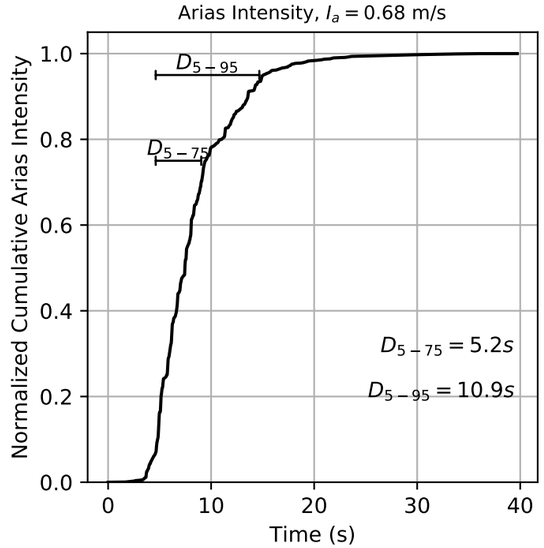

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
3. cm = centimeters
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5. s = seconds
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7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

LOMA PRIETA 1989
ANDERSON DAM (DS), 250°
FEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-25

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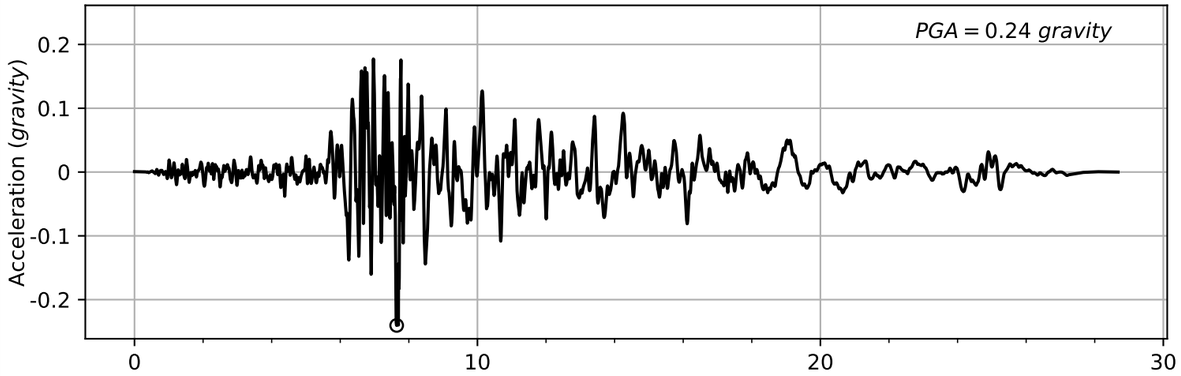

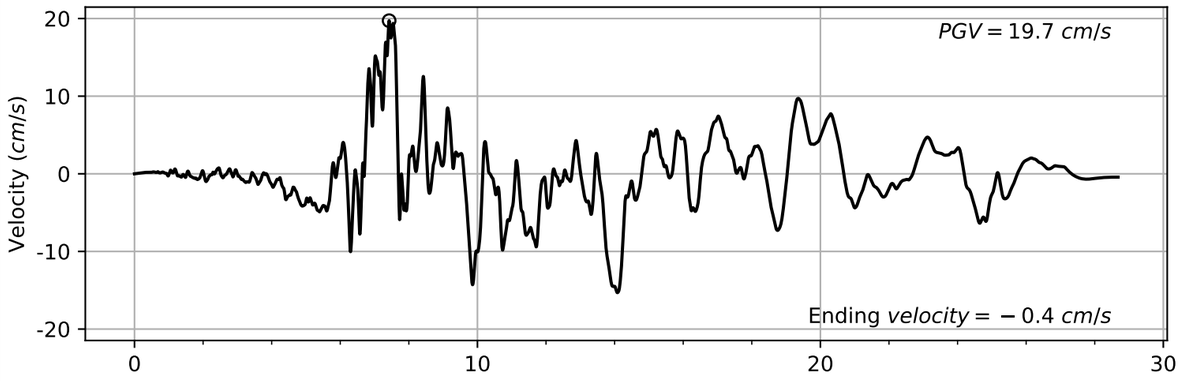

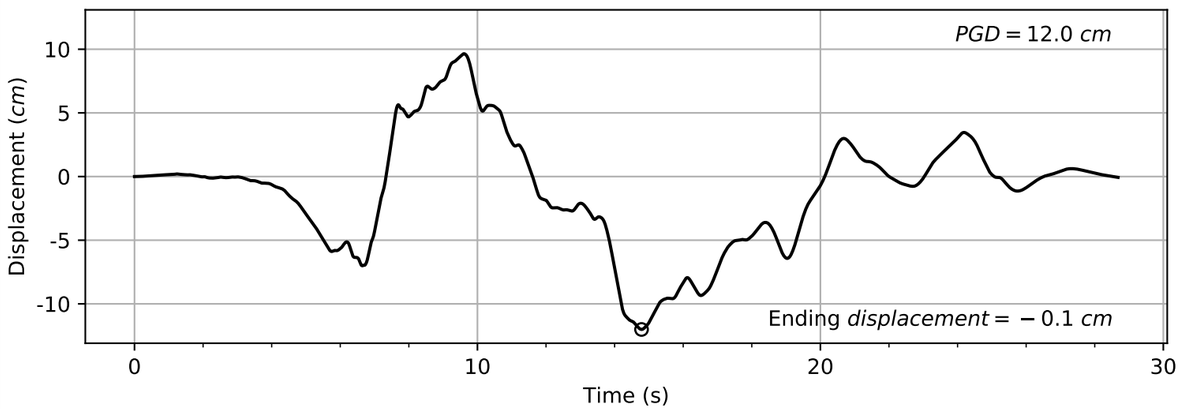

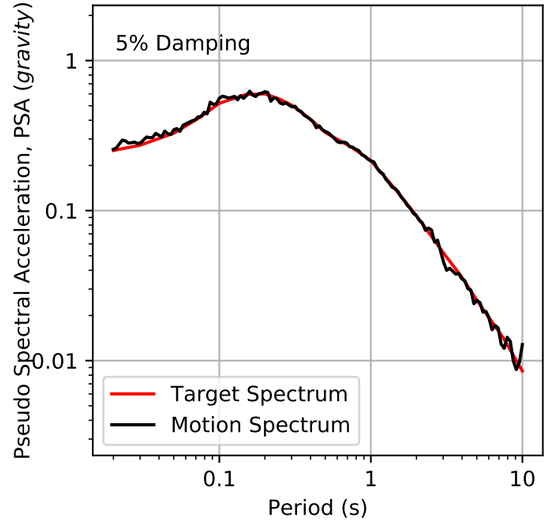

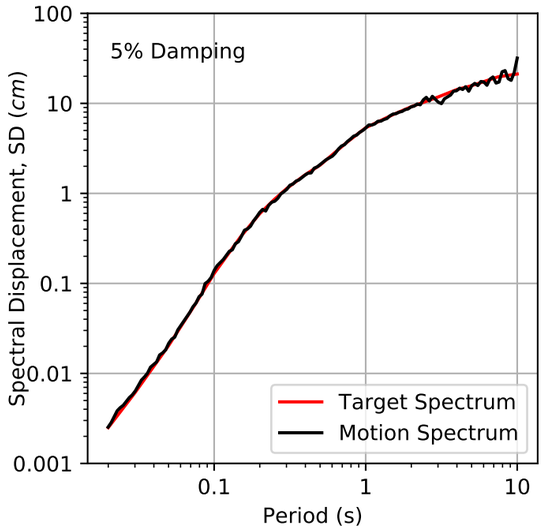

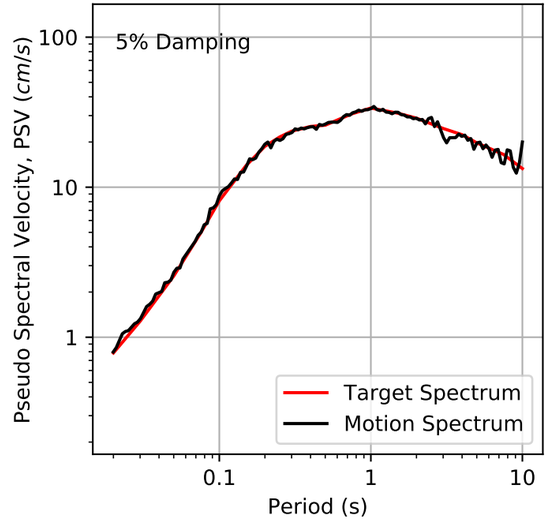

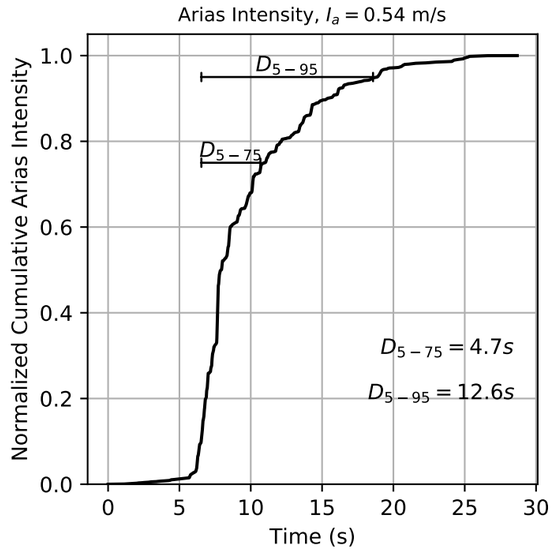

Notes:
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2. m/sec = meters per second
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7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

CAPE MENDOCINO 1992
LOLETA FIRE STATION, 360°
FEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-26

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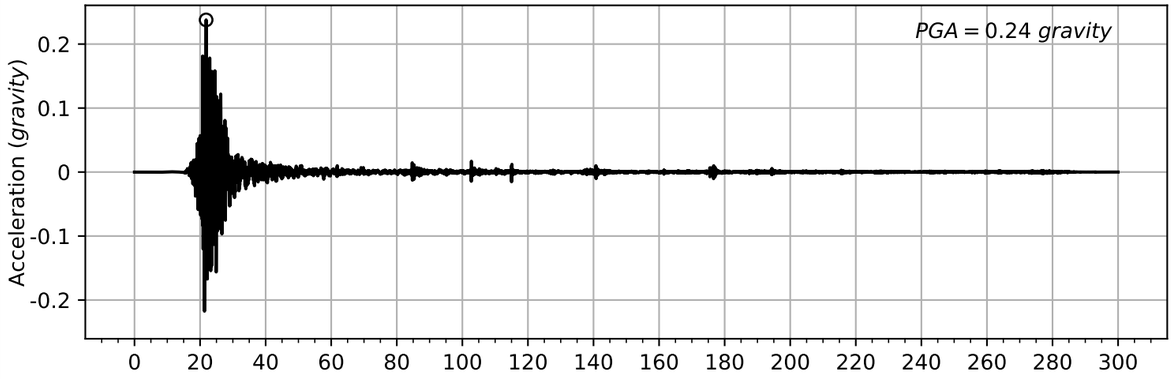

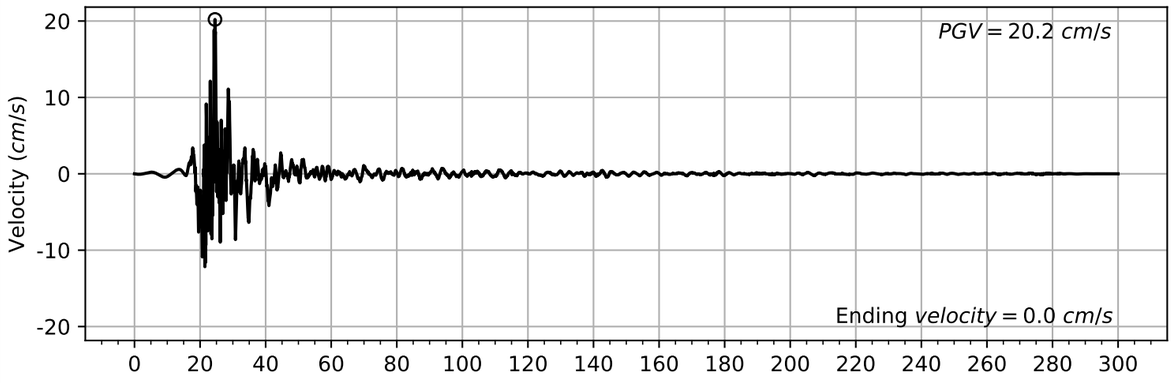

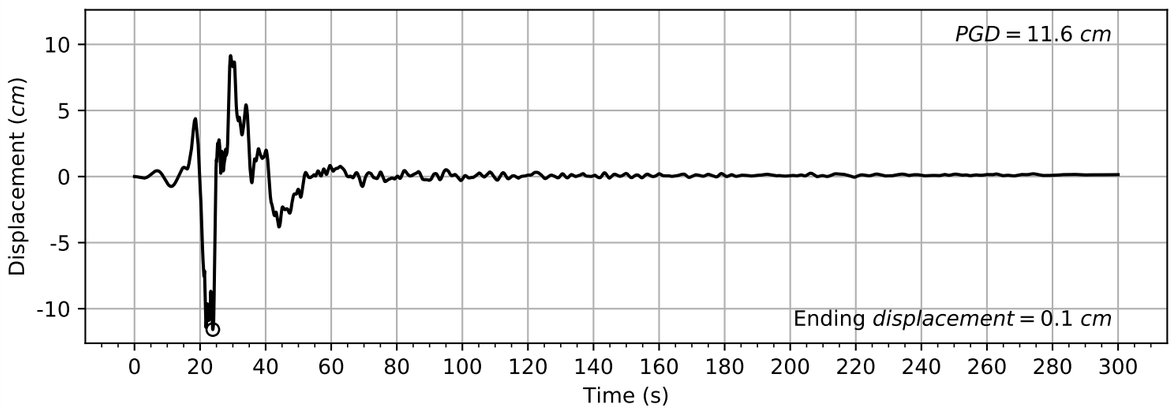

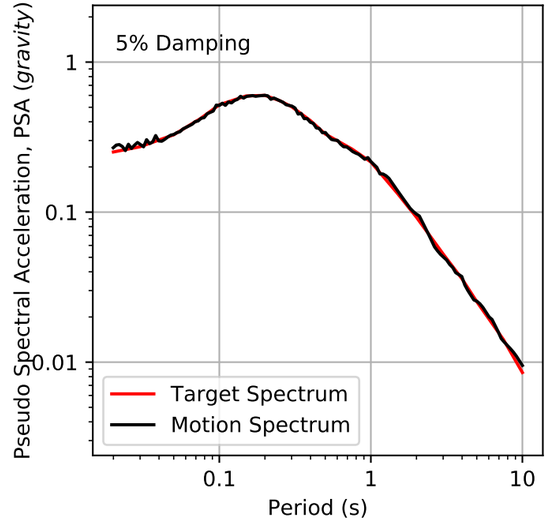

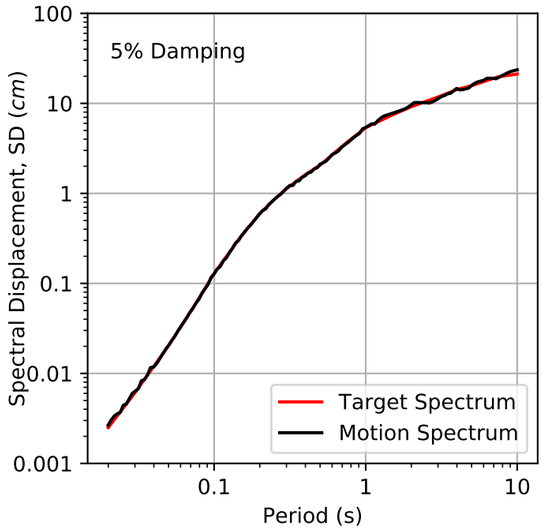

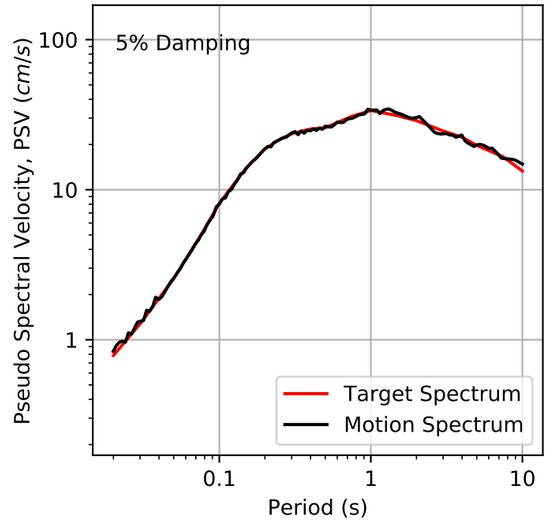

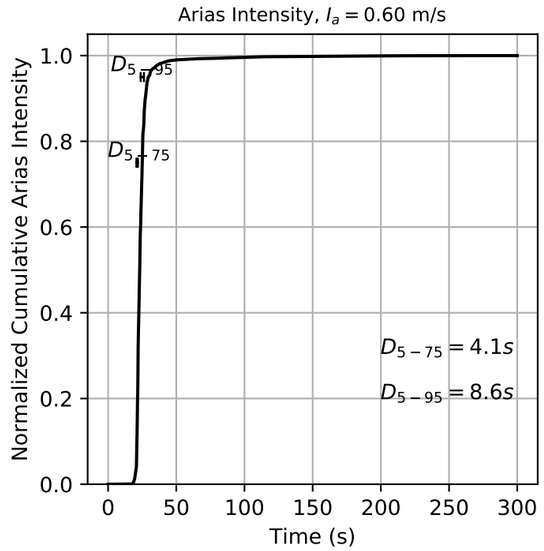

Notes:
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2. m/sec = meters per second
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5. s = seconds
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7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

IWATE, JAPAN 2008
MYGH02, NS
FEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-27

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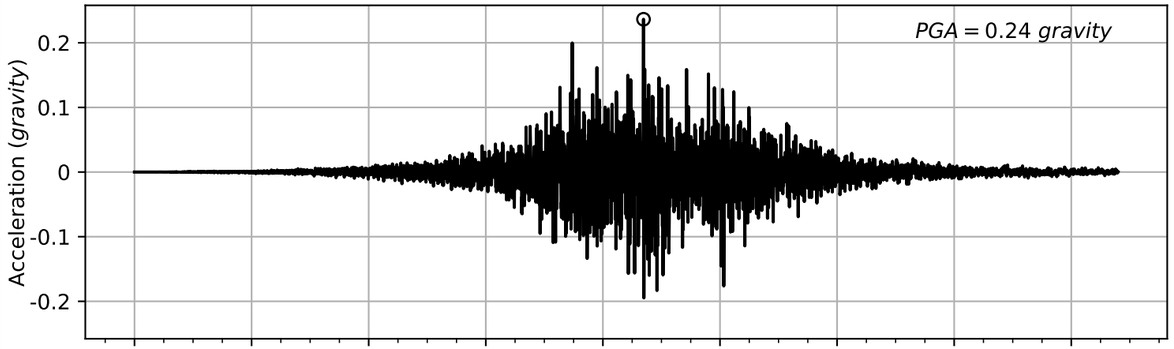

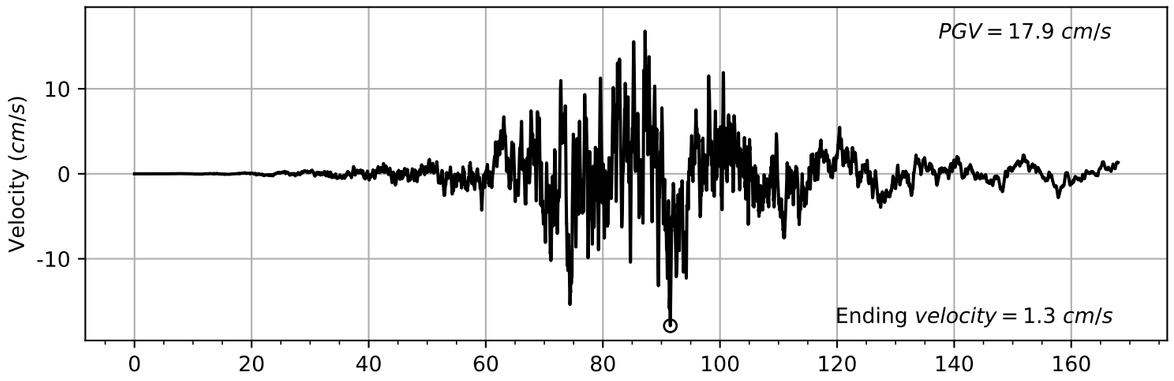

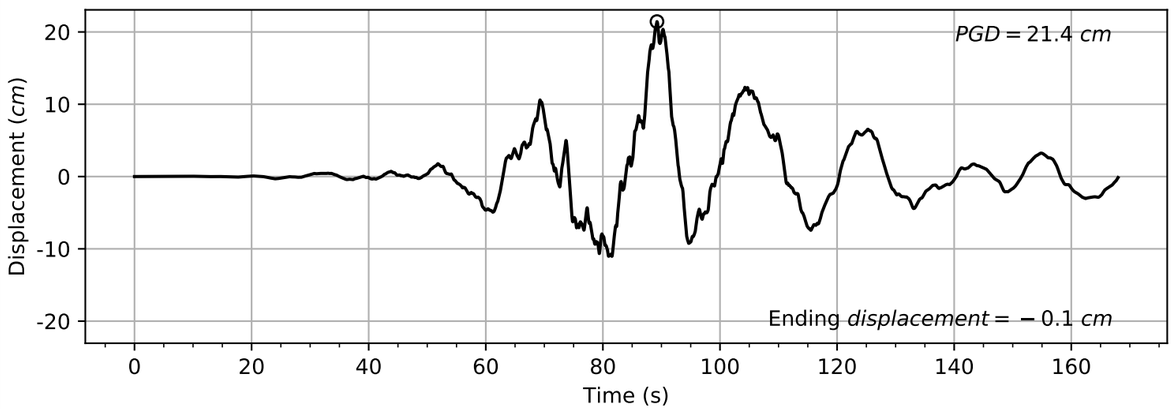

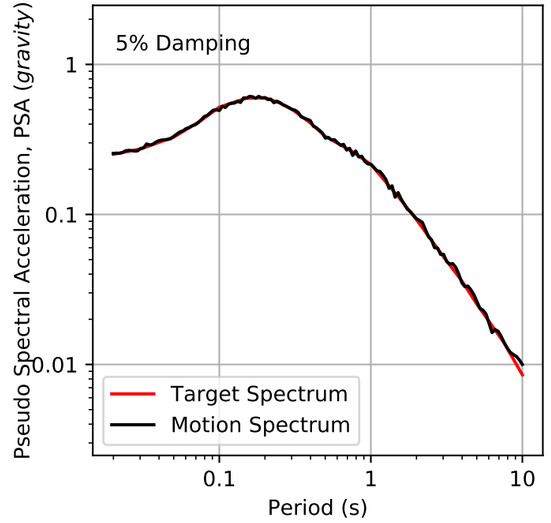

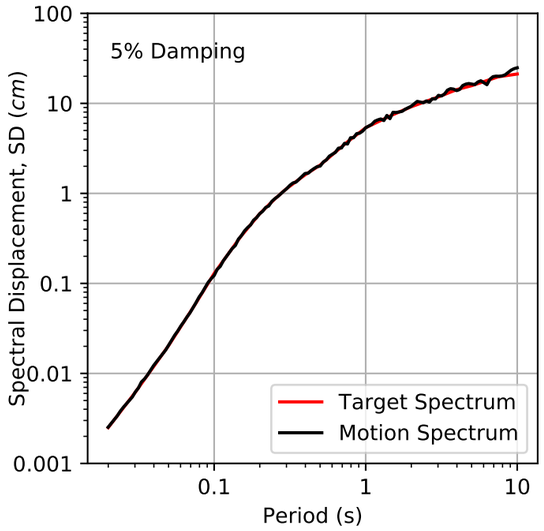

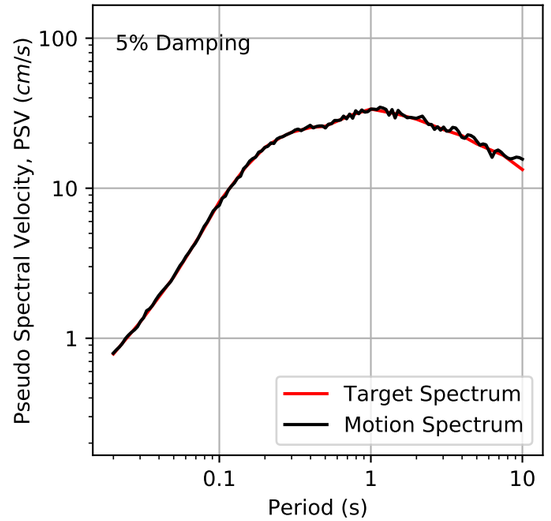

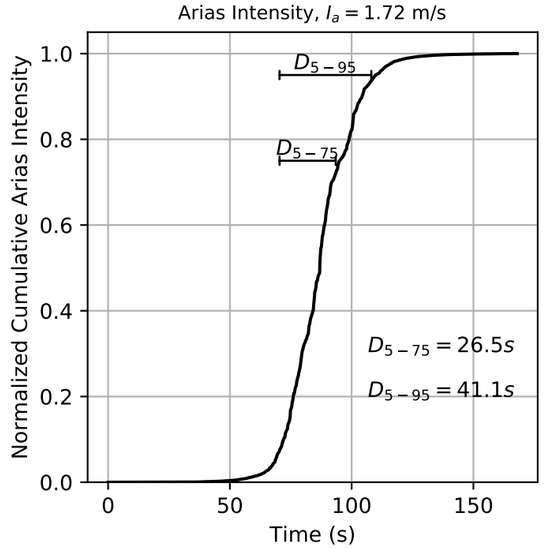

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
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4. cm/s = centimeters per second
5. s = seconds
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7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

MAULE, CHILE 2010
MUNI. SAN JOSE DE MAIPO, 090°
FEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-28

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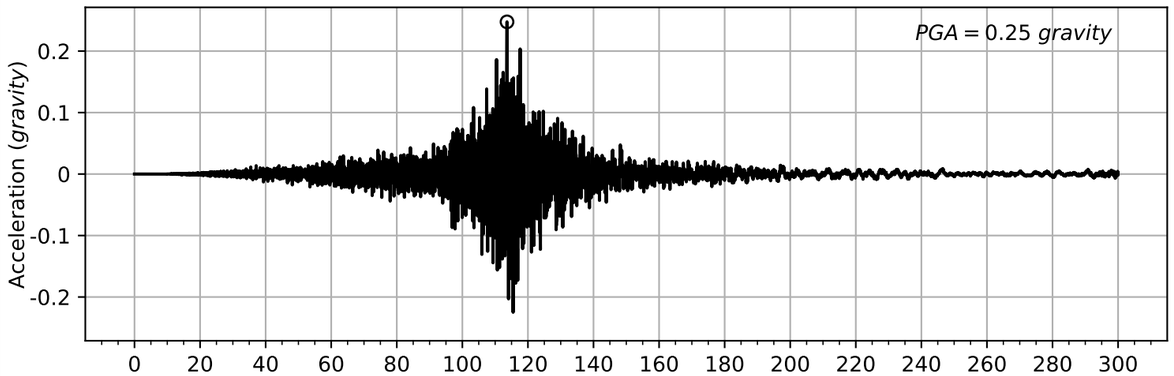

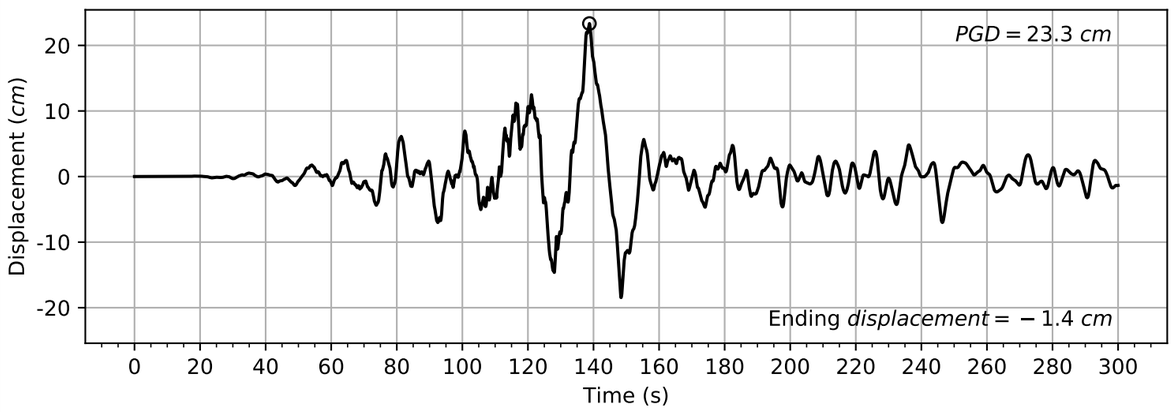

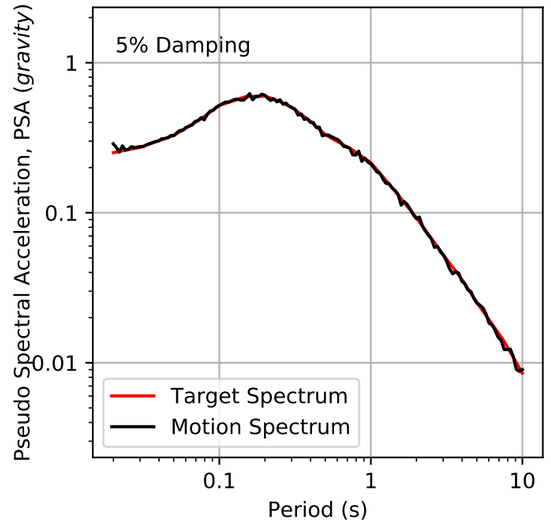

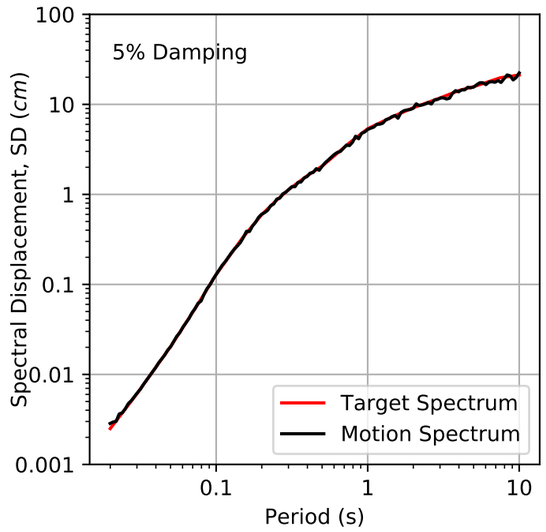

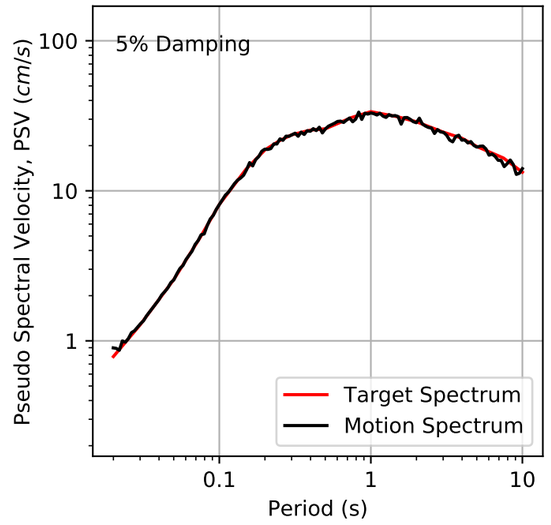

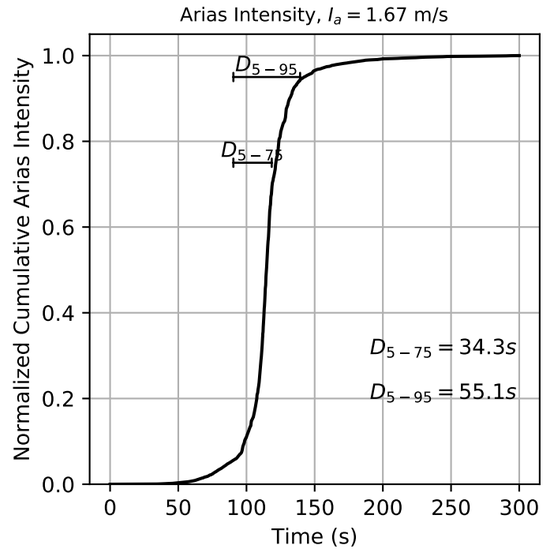

Notes:
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3. cm = centimeters
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8. Response spectra calculated using a linear interpolation of the acceleration ground motion

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

TOHOKU, JAPAN 2011
SAWARA, NS
FEE MATCHED TIME HISTORY

September 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-29

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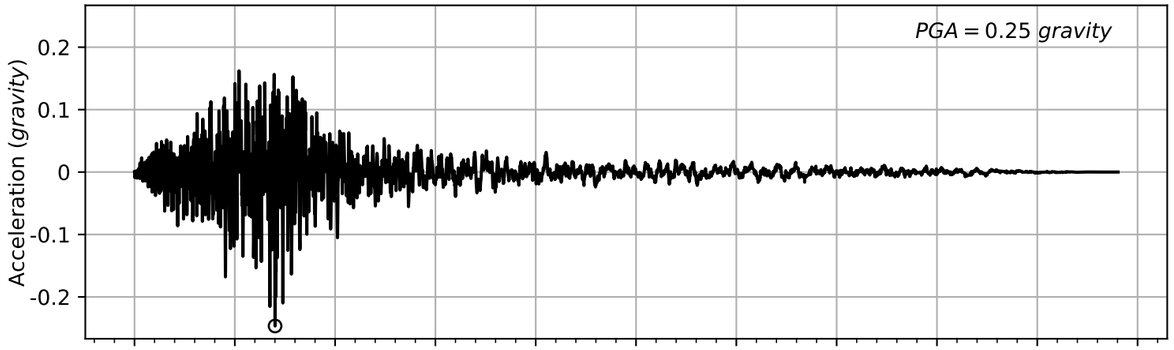

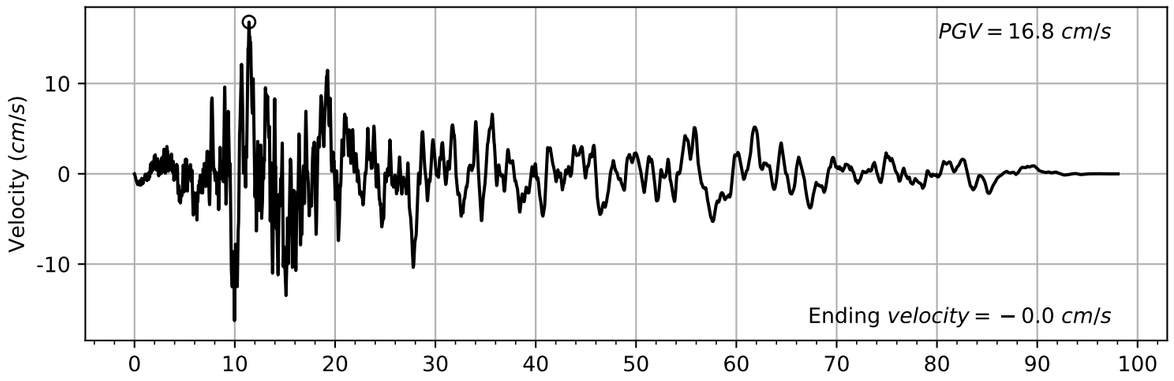

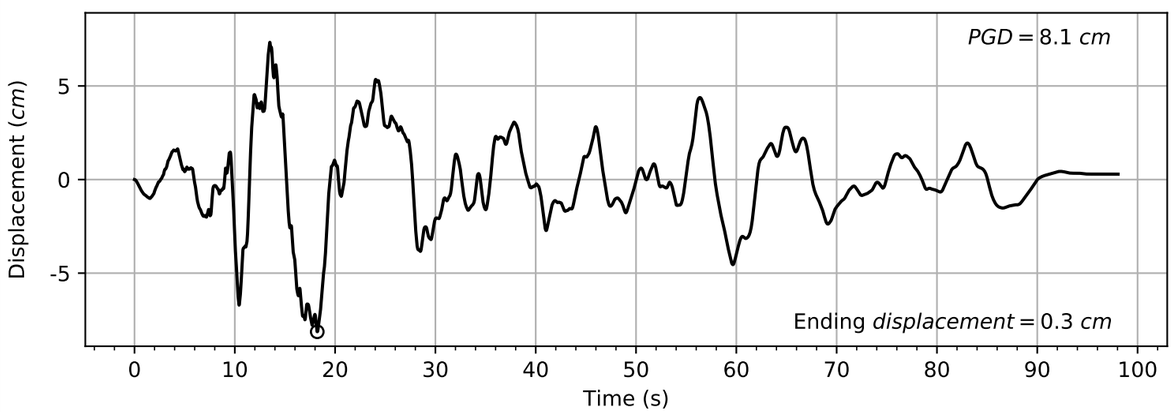

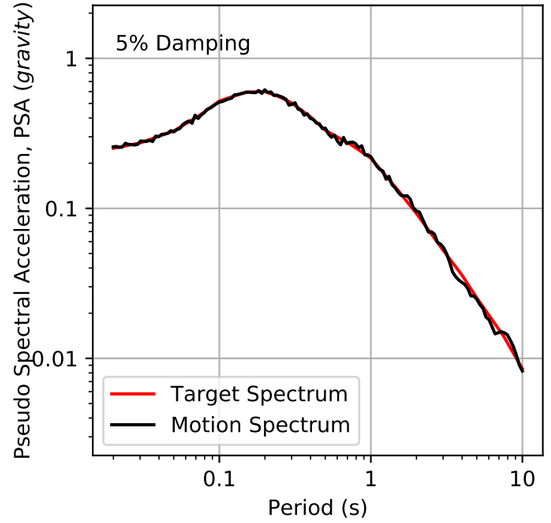

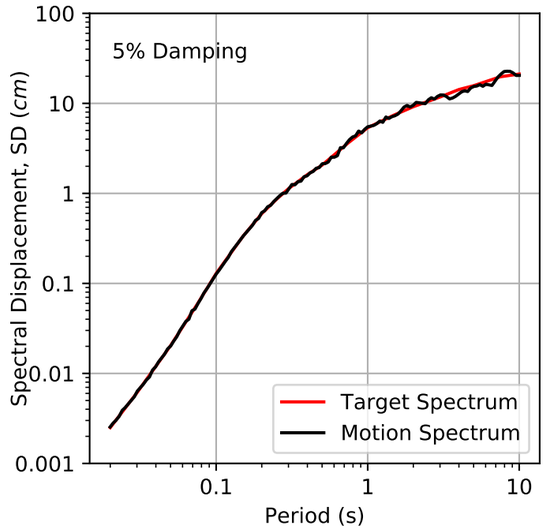

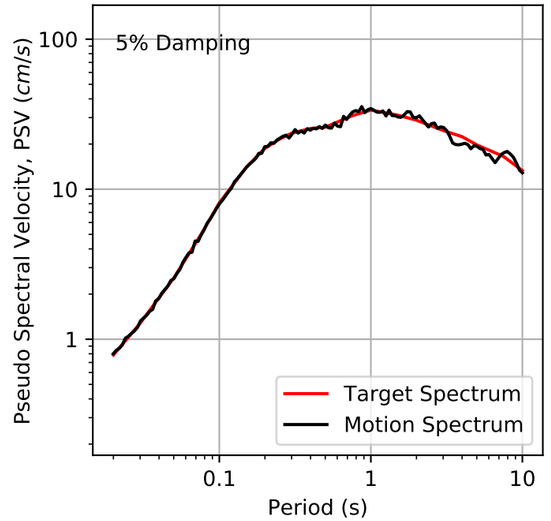

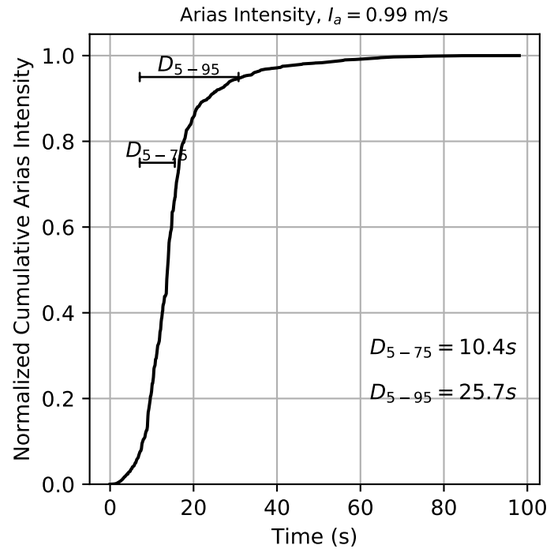

Notes:
1. PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement
2. m/sec = meters per second
3. cm = centimeters
4. cm/s = centimeters per second
5. s = seconds
6. D_{5-75} = significant duration from 5 to 75 percent of normalized cumulative Arias intensity
7. D_{5-95} = significant duration from 5 to 95 percent of normalized cumulative Arias intensity
8. Response spectra calculated using a linear interpolation of the acceleration ground motion

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NISQUALLY, WASHINGTON 2001
HALVERSON RES., 270°
FEE MATCHED TIME HISTORY

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FIG. A-30

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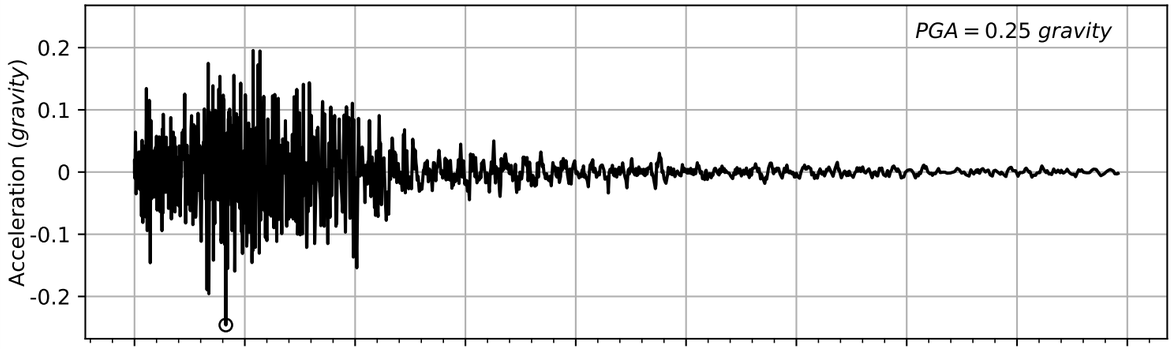

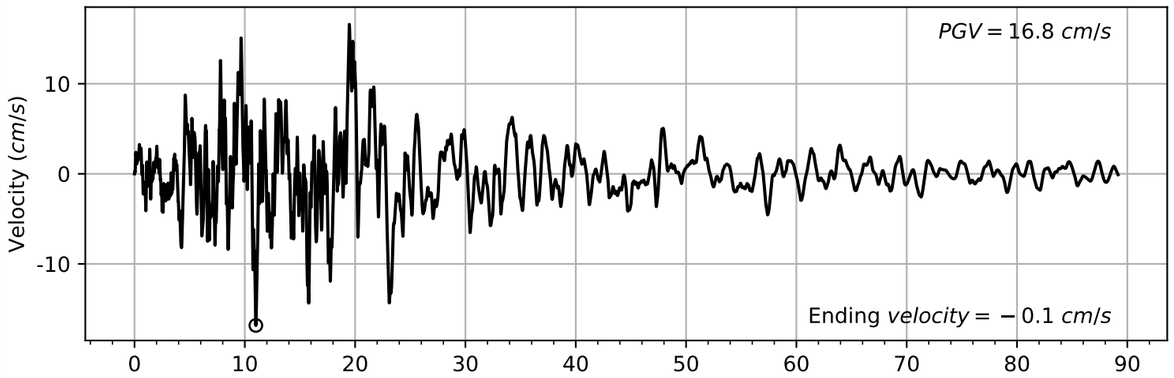

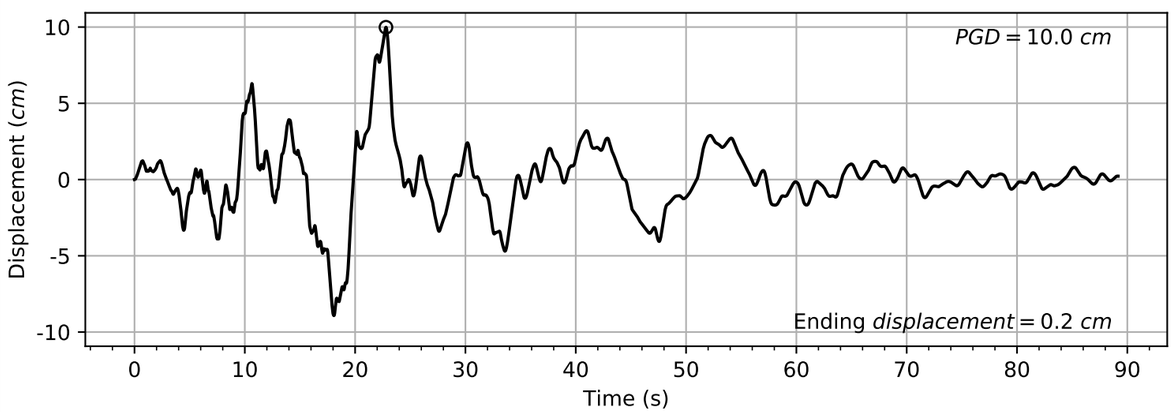

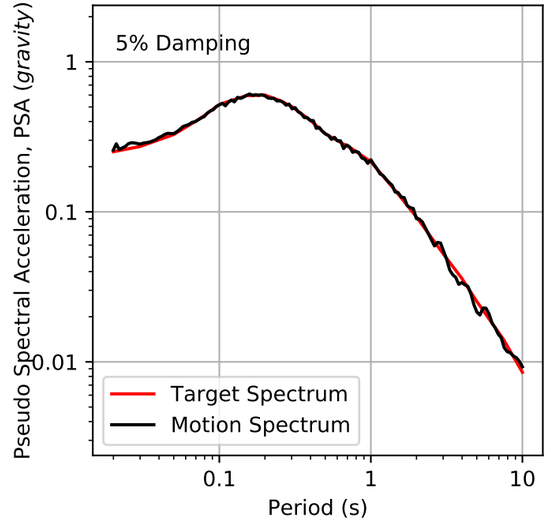

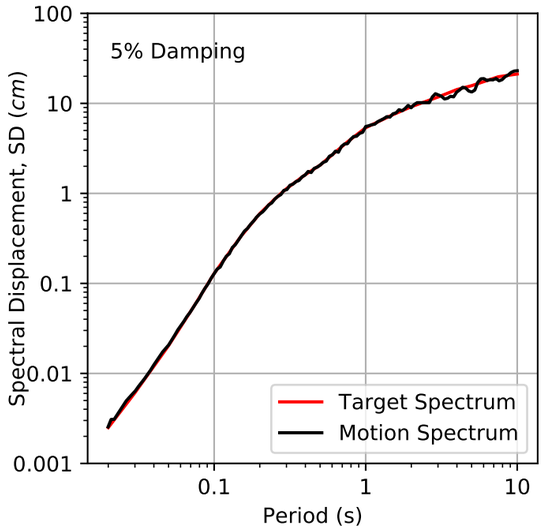

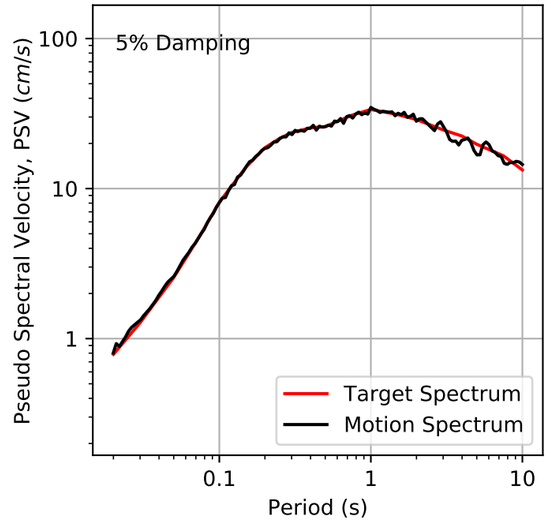

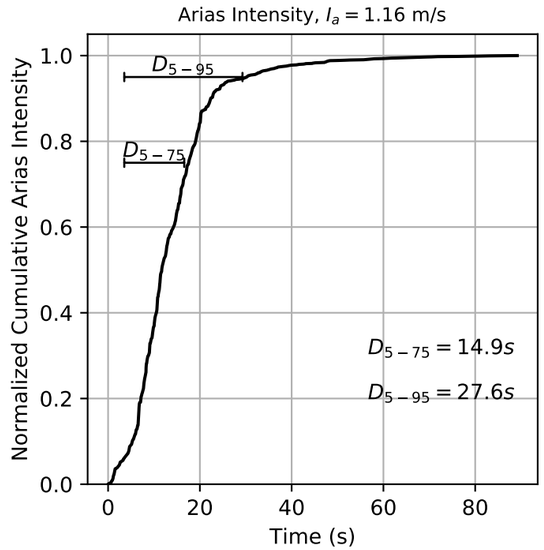

Notes:
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2. m/sec = meters per second
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FIG. A-31

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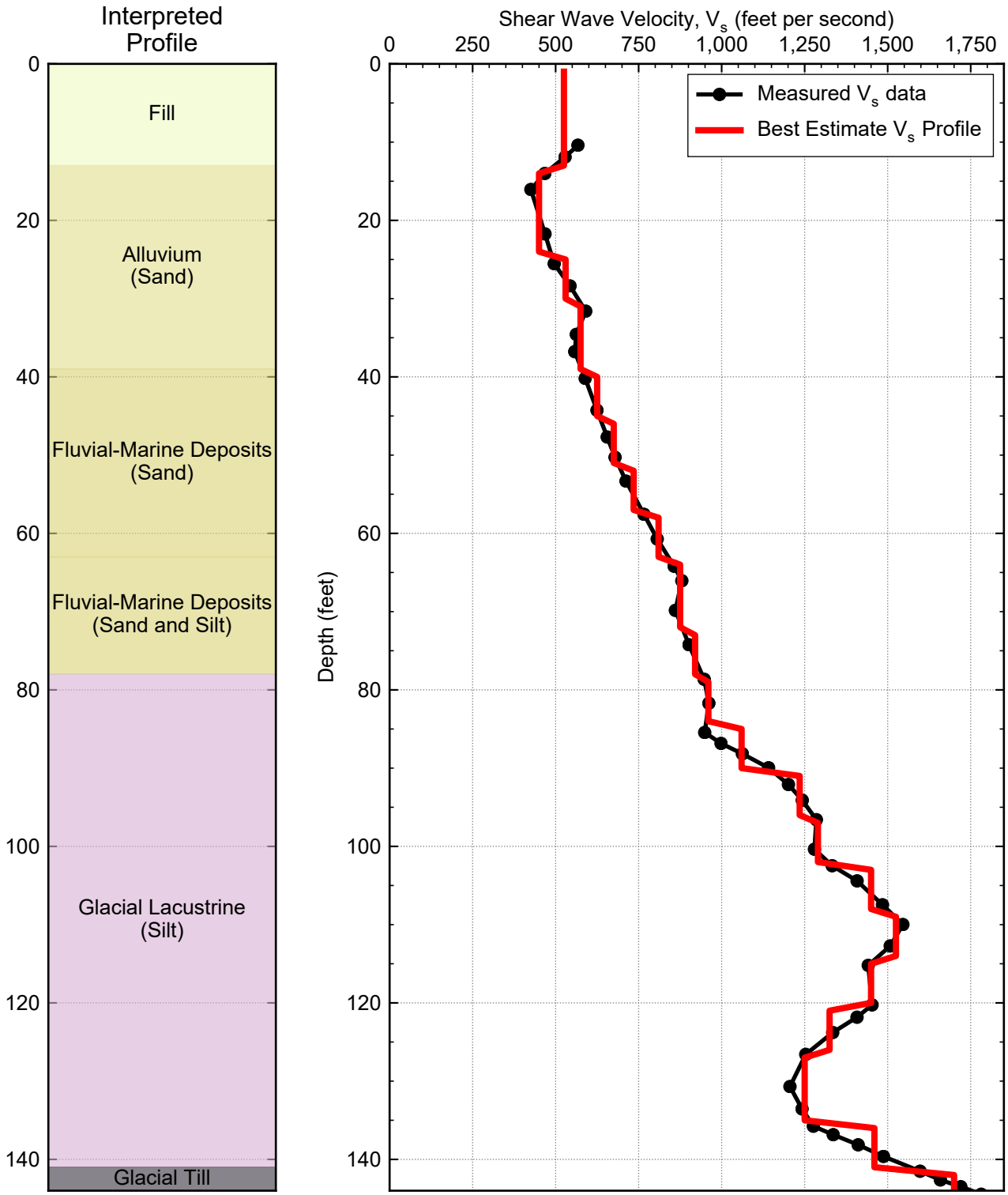

Note:
1. The natural site period estimated from the interpretted shear wave velocity profile was evaluated as approximately 0.7 seconds.

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SHEAR WAVE VELOCITY
INTERPRETED PROFILE

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FIG. A-32

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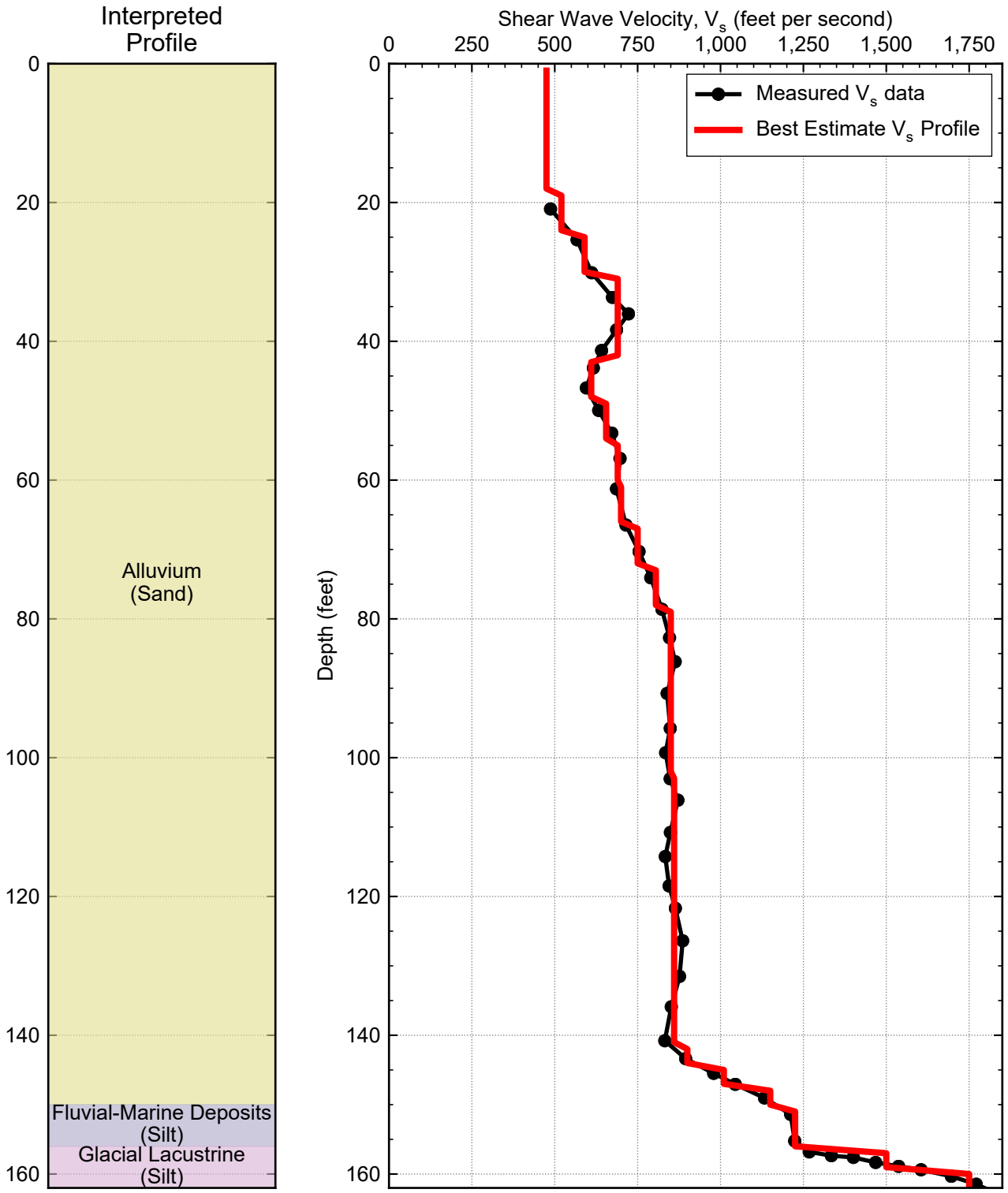

Note:
1. The natural site period estimated from the interpretted shear wave velocity profile was evaluated as approximately 0.9 seconds.

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SHEAR WAVE VELOCITY
INTERPRETED PROFILE

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FIG. A-33

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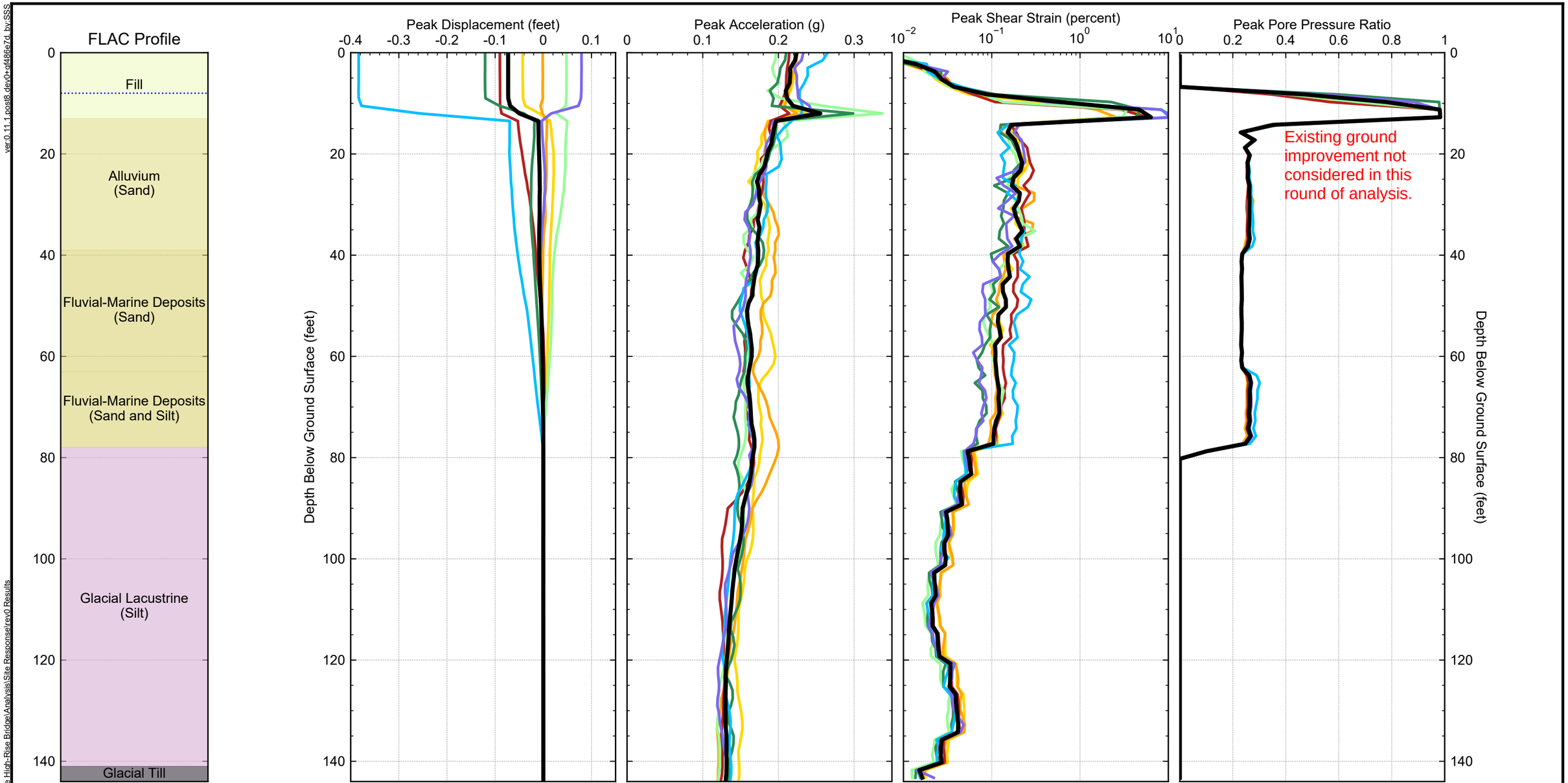

Source	Earthquake	Station	Component
Crustal	Cape Mendocino	Loleta FS	360 deg.
Crustal	Iwate	MYGH02	NS
Crustal	Loma Prieta	Anderson Dam	250 deg.
Intraslab	Nisqually	Halverson Res.	270 deg.
Intraslab	Olympia (1949)	Olympia Test Lab	356 deg.
CSZ	Maule	La Plantina	360 deg.
CSZ	Tohoku	SAWARA	NS
Average Response			

Note:
1. CSZ = Cascadia Subduction Zone; deg. = degrees; FEE = Functional Evaluation Earthquake; g = acceleration of gravity; WSDOT = Washington State Department of Transportation.Site response analyses were performed using FLAC v.8 (Itasca, 2018).

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FIG. A-34

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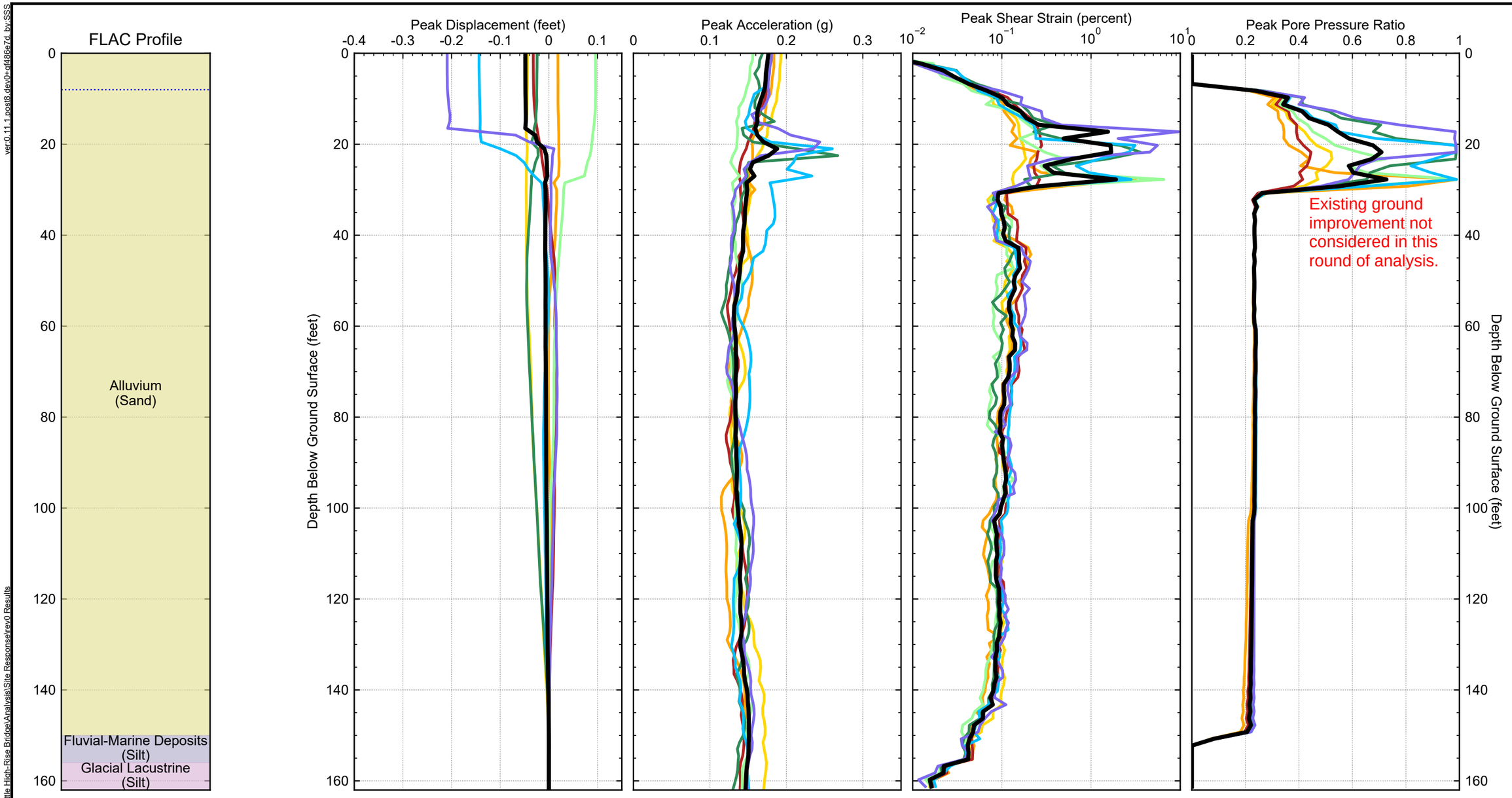

Source	Earthquake	Station	Component
Crustal	Cape Mendocino	Loleta FS	360 deg.
Crustal	Iwate	MYGH02	NS
Crustal	Loma Prieta	Anderson Dam	250 deg.
Intraslab	Nisqually	Halverson Res.	270 deg.
Intraslab	Olympia (1949)	Olympia Test Lab	356 deg.
CSZ	Maule	La Plantina	360 deg.
CSZ	Tohoku	SAWARA	NS
Average Response			

Note:
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FIG. A-35

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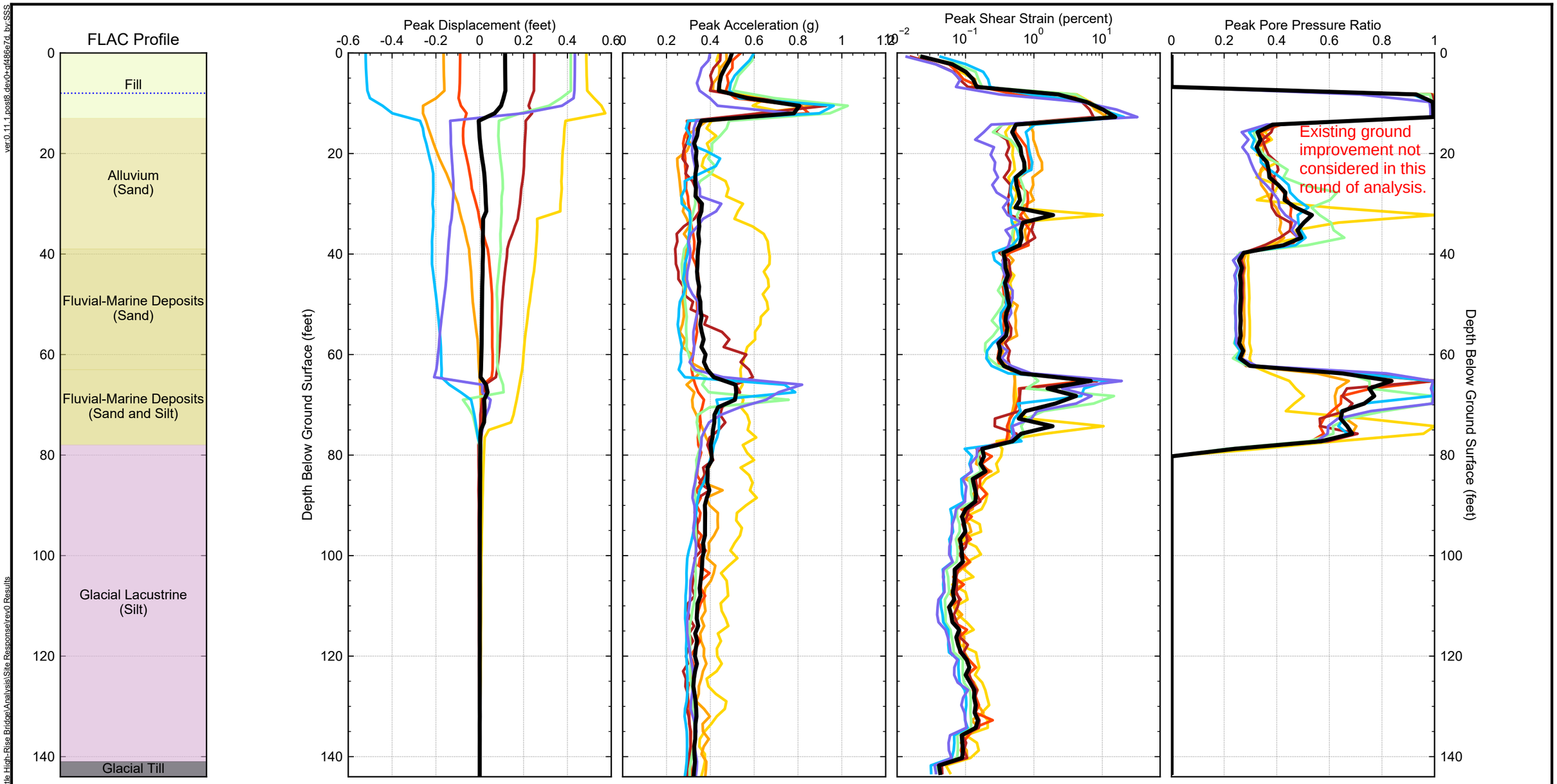

Source	Earthquake	Station	Component
Crustal	Cape Mendocino	Loleta FS	360 deg.
Crustal	Iwate	MYGH02	NS
Crustal	Loma Prieta	Anderson Dam	250 deg.
Crustal	Loma Prieta	Aloha Ave	90 deg.
Intraslab	Nisqually	Halverson Res.	270 deg.
CSZ	Maule	La Plantina	360 deg.
CSZ	Tohoku	SAWARA	NS
Average Response			

Notes:
1. CSZ = Cascadia Subduction Zone; deg. = degrees; g = acceleration of gravity; SEE = Safety Evaluation Earthquake; WSDOT = Washington State Department of Transportation.
2. Site response analyses were performed using FLAC v.8 (Itasca, 2018).

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WSDOT SEE
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FIG. A-36

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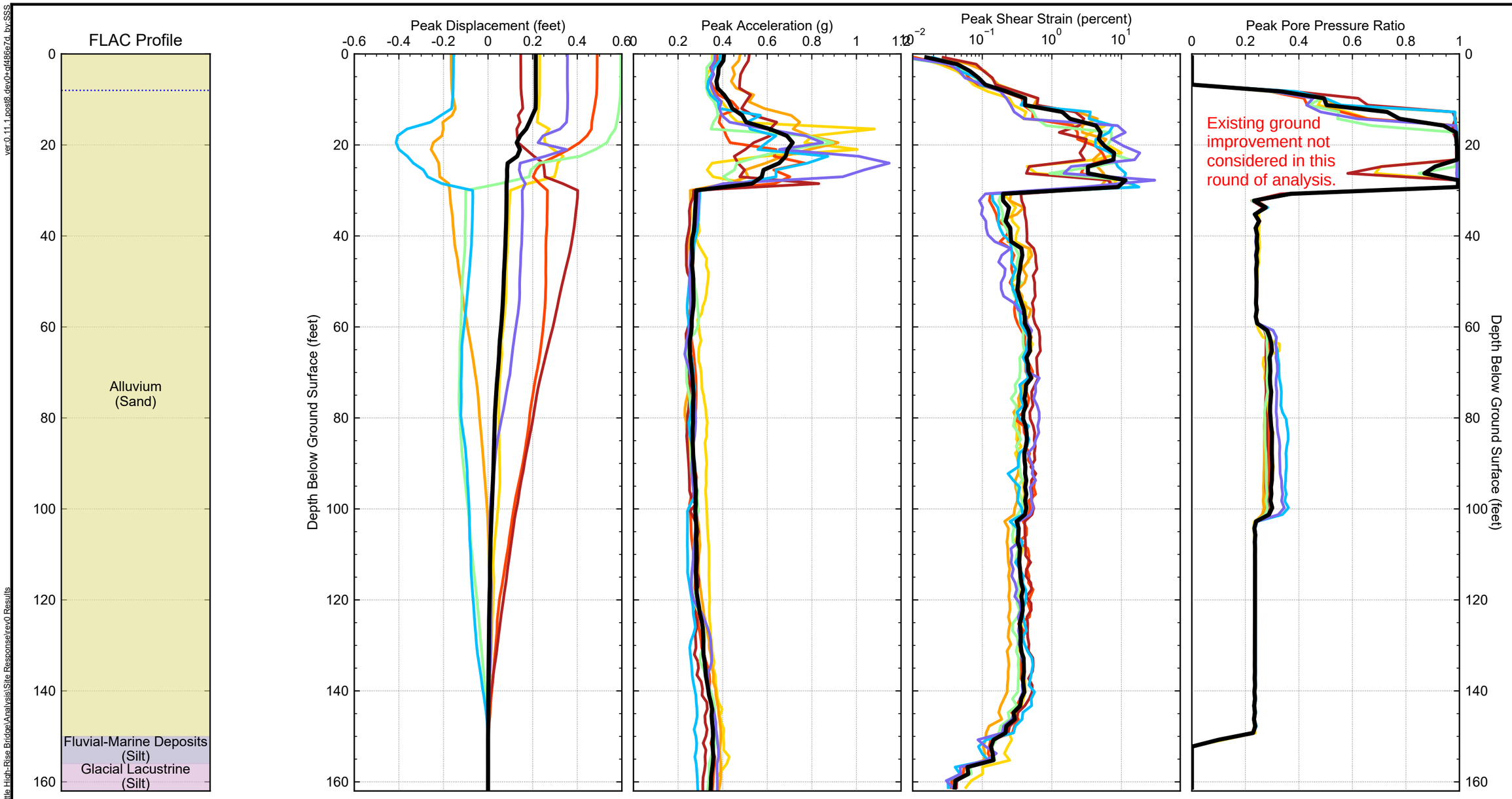

Source	Earthquake	Station	Component
Crustal	Cape Mendocino	Loleta FS	360 deg.
Crustal	Iwate	MYGH02	NS
Crustal	Loma Prieta	Anderson Dam	250 deg.
Crustal	Loma Prieta	Aloha Ave	90 deg.
Intraslab	Nisqually	Halverson Res.	270 deg.
CSZ	Maule	La Plantina	360 deg.
CSZ	Tohoku	SAWARA	NS
Average Response			

Notes:
1. CSZ = Cascadia Subduction Zone; deg. = degrees; g = acceleration of gravity; SEE = Safety Evaluation Earthquake; WSDOT = Washington State Department of Transportation.
2. Site response analyses were performed using FLAC v.8 (Itasca, 2018).

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FIG. A-37

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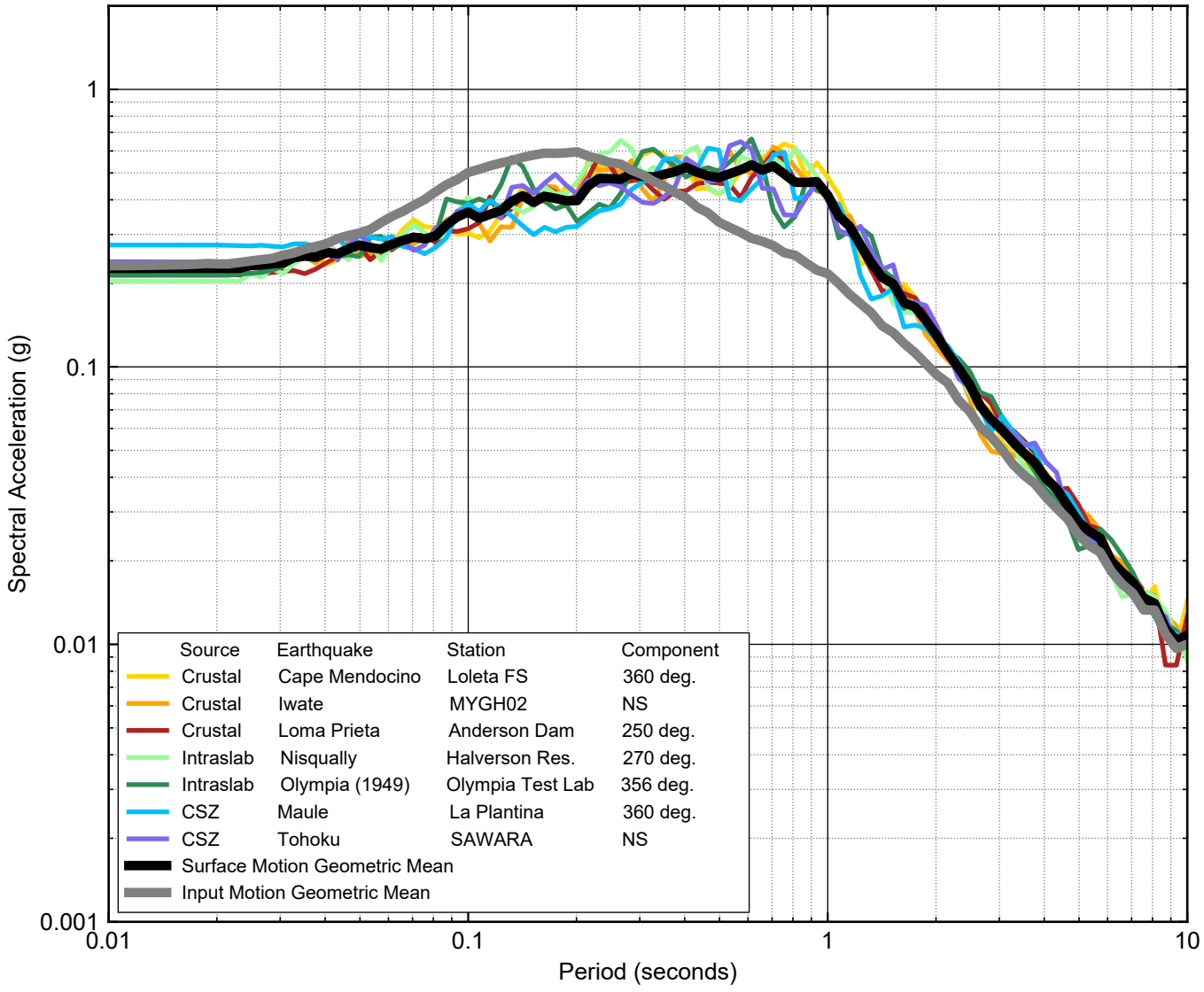

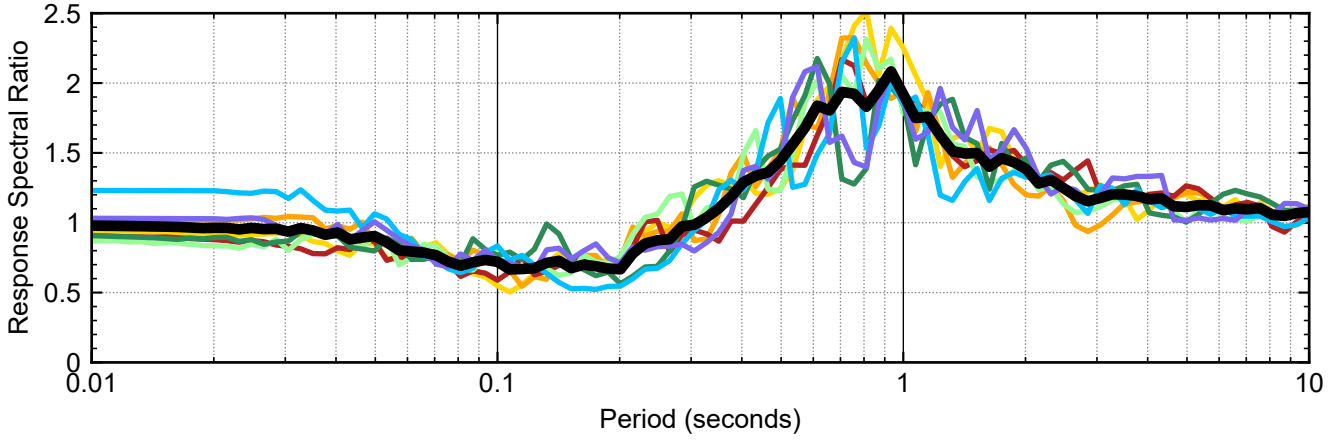

Notes:
1. CSZ = Cascadia Subduction Zone; deg. = degrees; FEE = Functional Evaluation Earthquake; g = acceleration of gravity; WSDOT = Washington State Department of Transportation.
2. Site response analyses were performed using FLAC v.8 (Itasca, 2018).
3. The response spectral ratio is defined as the ratio of the surface response spectra to the base response spectra.

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SURFACE RESPONSE SPECTRA

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FIG. A-38

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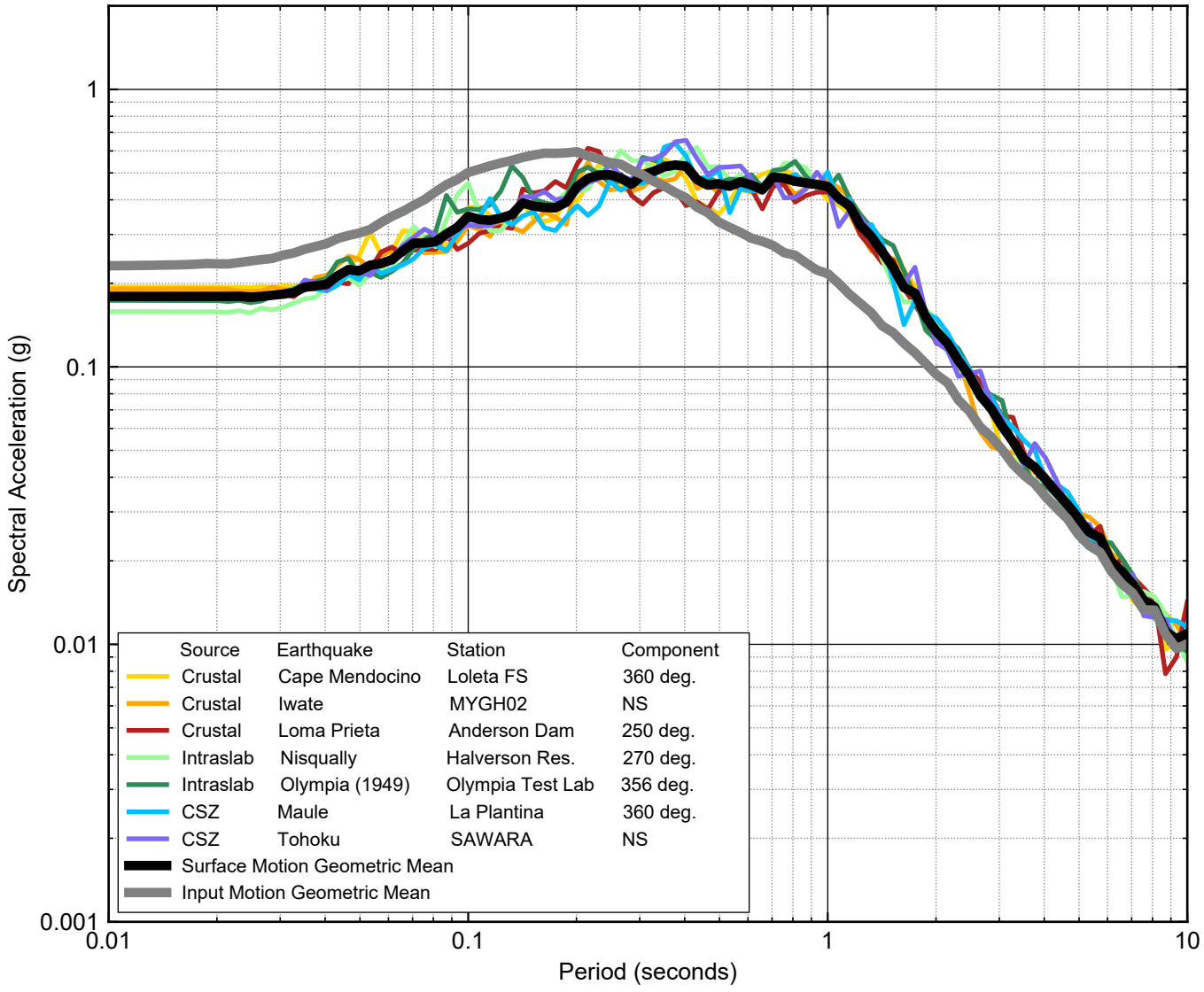

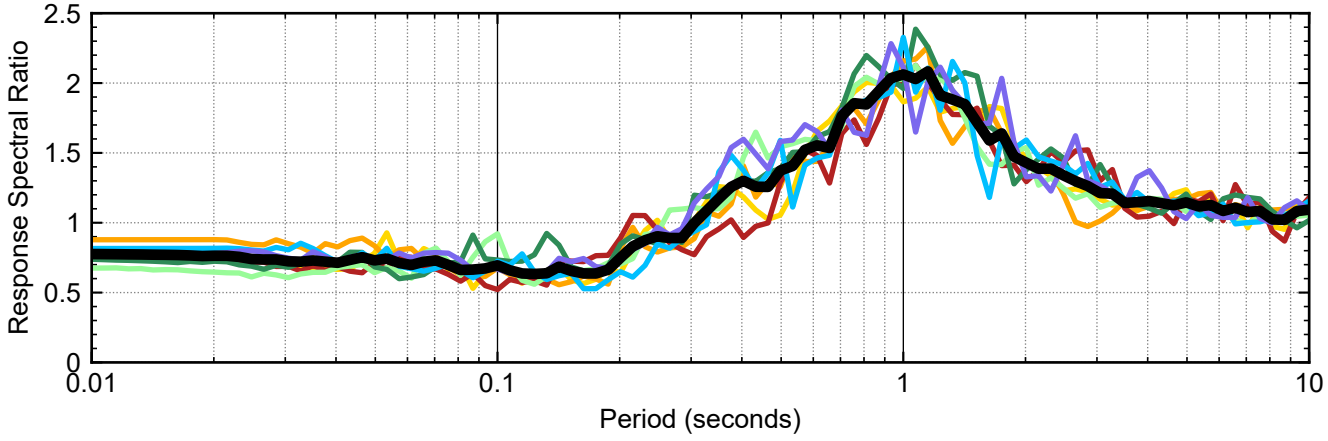

Notes:
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FIG. A-39

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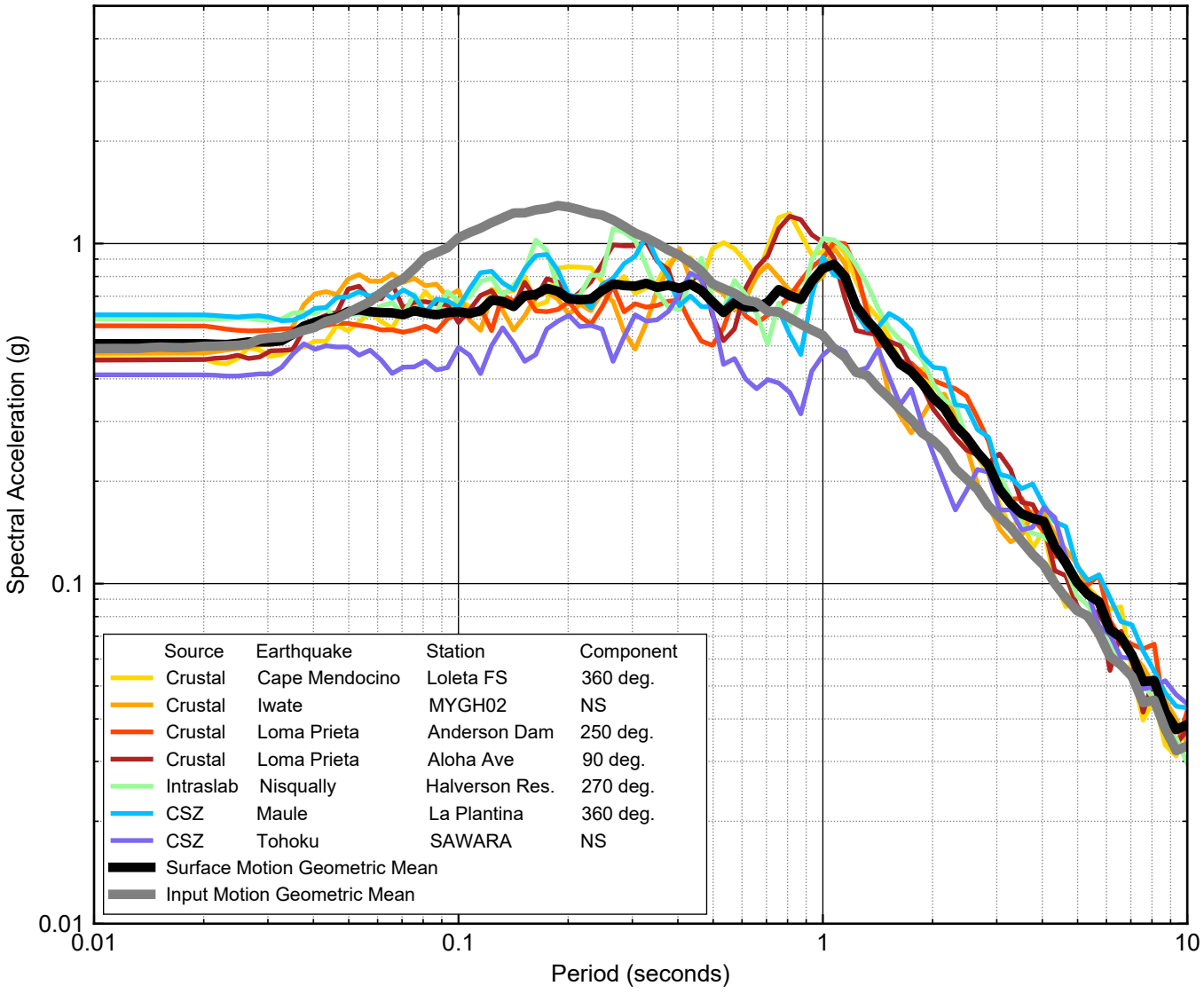

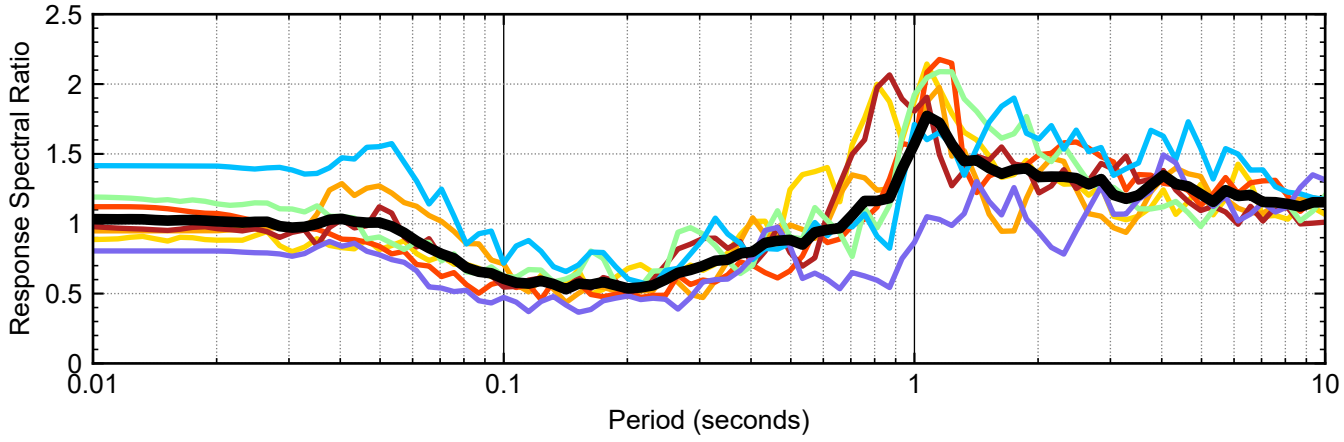

Notes:
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FIG. A-40

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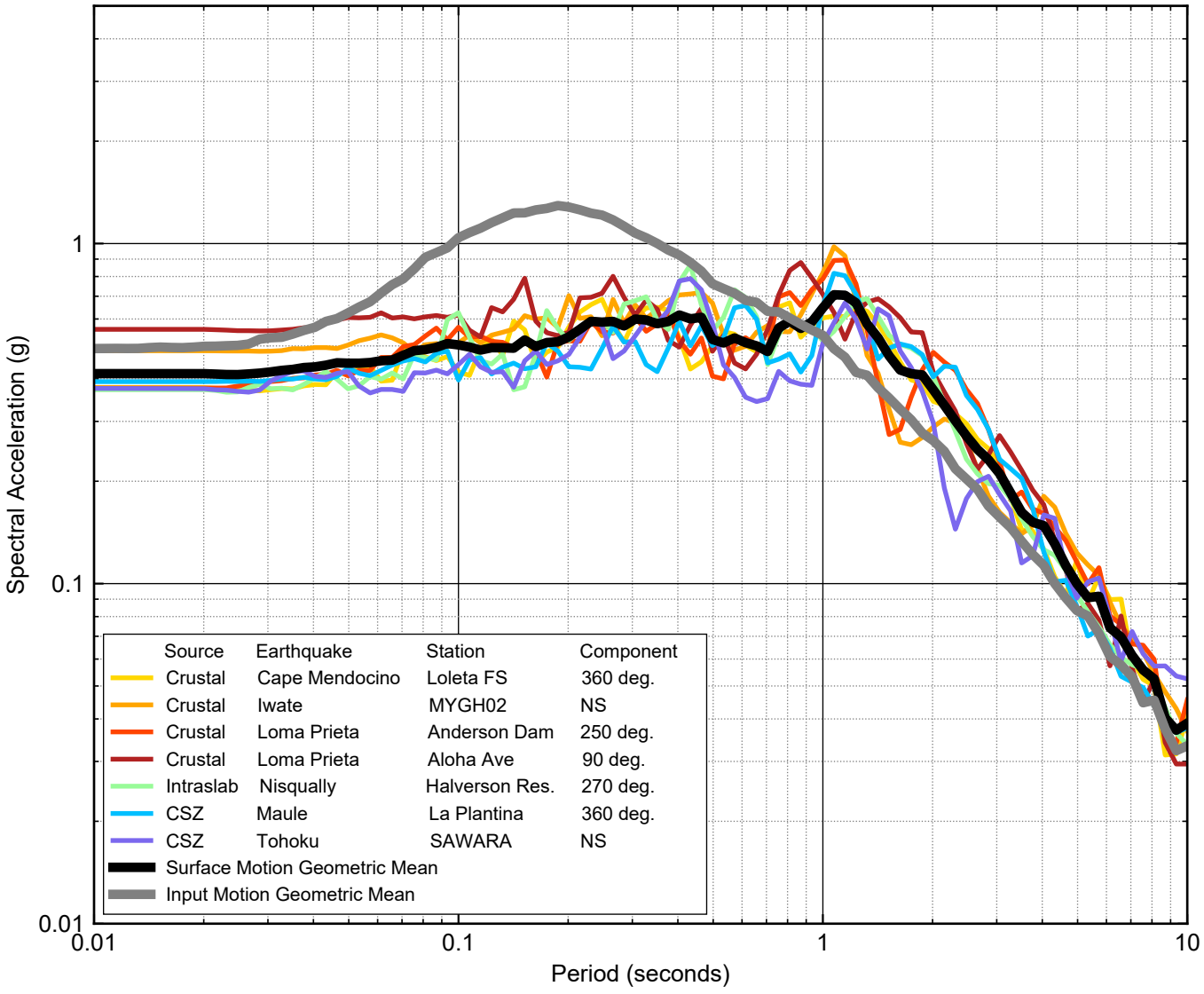

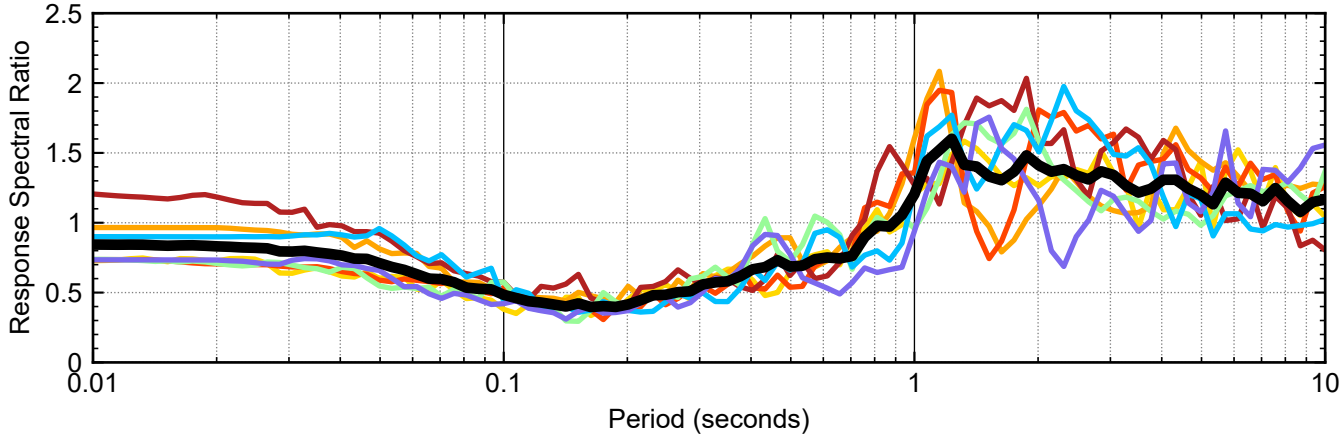

Notes:
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PIER 17 AND 18
WSDOT SEE
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FIG. A-41

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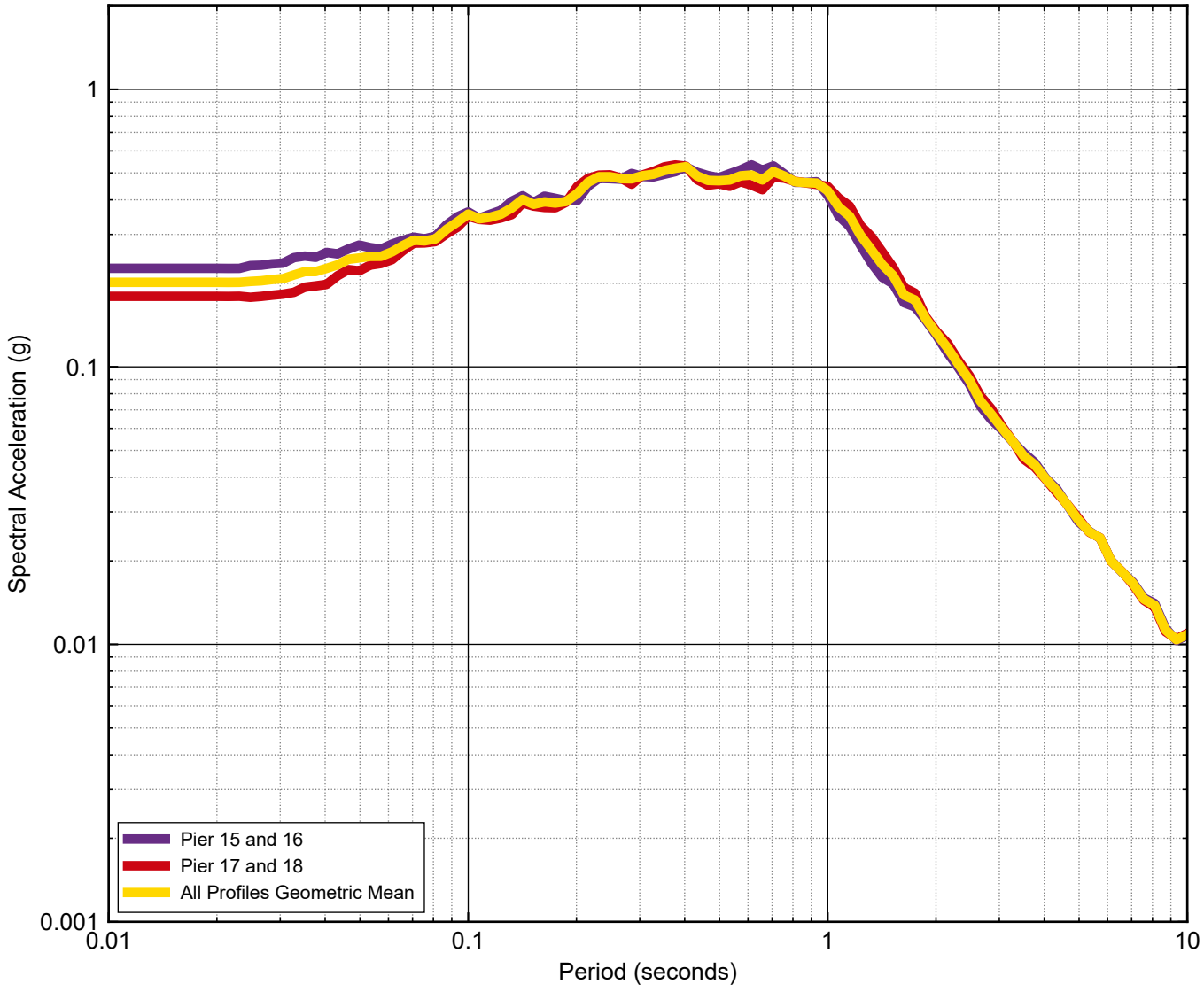

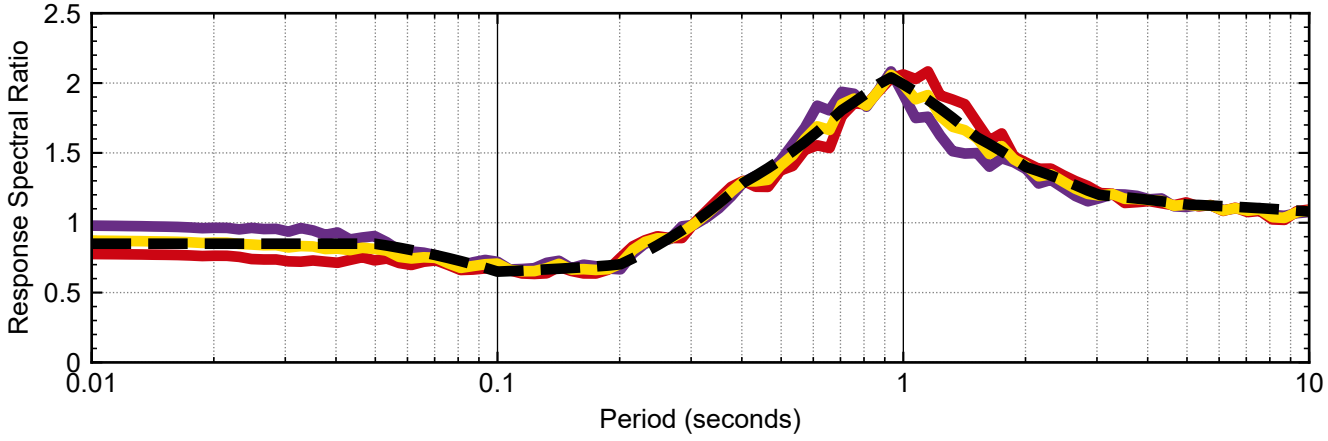

Notes:
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2. Site response analyses were performed using FLAC v.8 (Itasca, 2018).
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GEOMETRIC MEAN
SURFACE RESPONSE SPECTRA

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FIG. A-42

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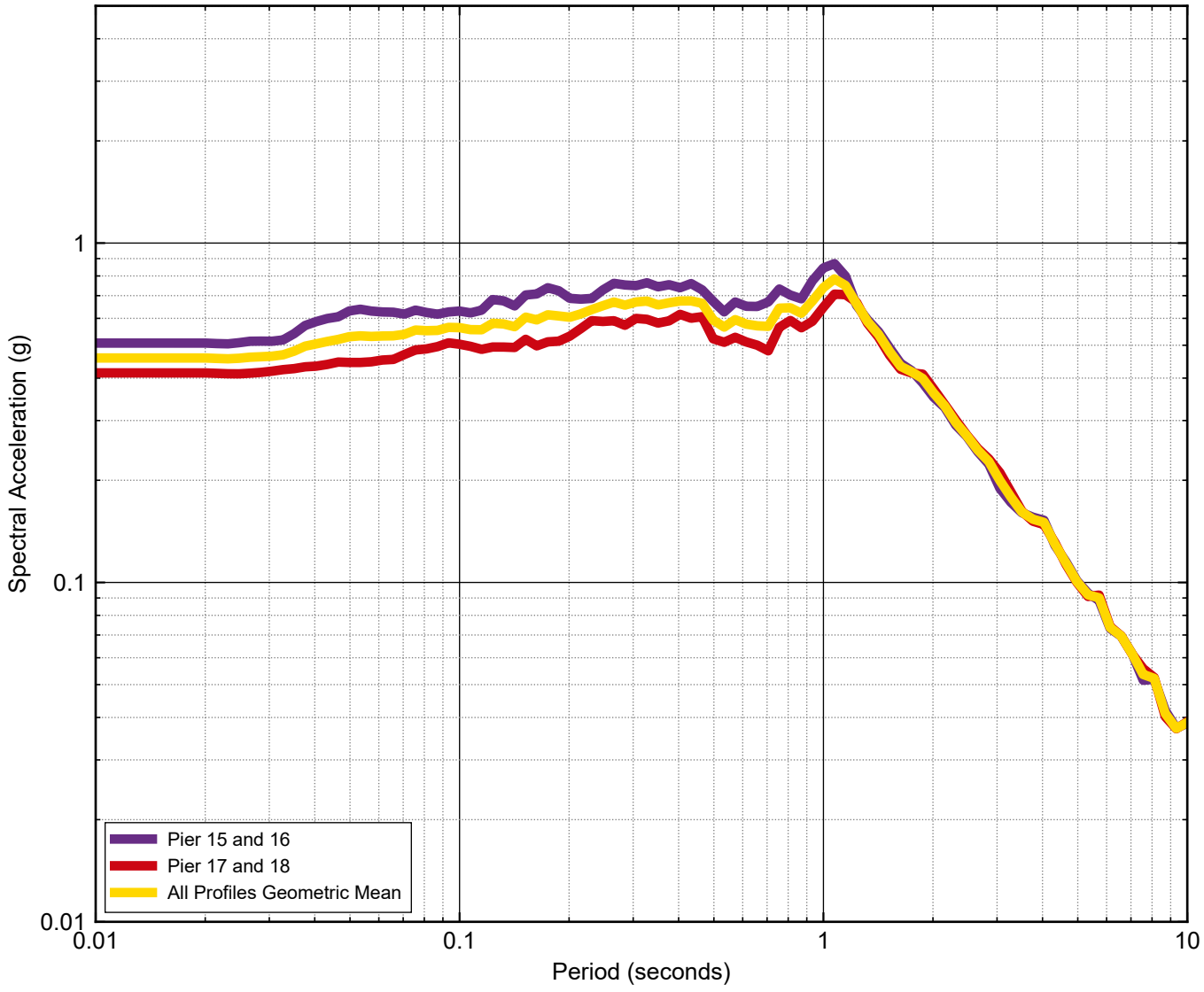

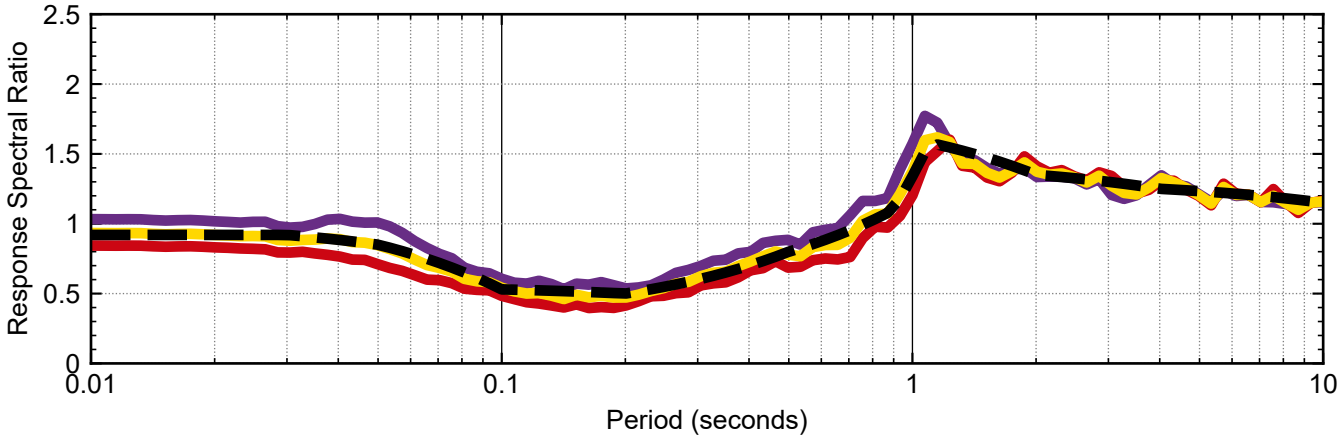

Notes:
1. CSZ = Cascadia Subduction Zone; deg. = degrees; g = acceleration of gravity; SEE = Safety Evaluation Earthquake; WSDOT = Washington State Department of Transportation.
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WSDOT SEE
GEOMETRIC MEAN
SURFACE RESPONSE SPECTRA

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FIG. A-43

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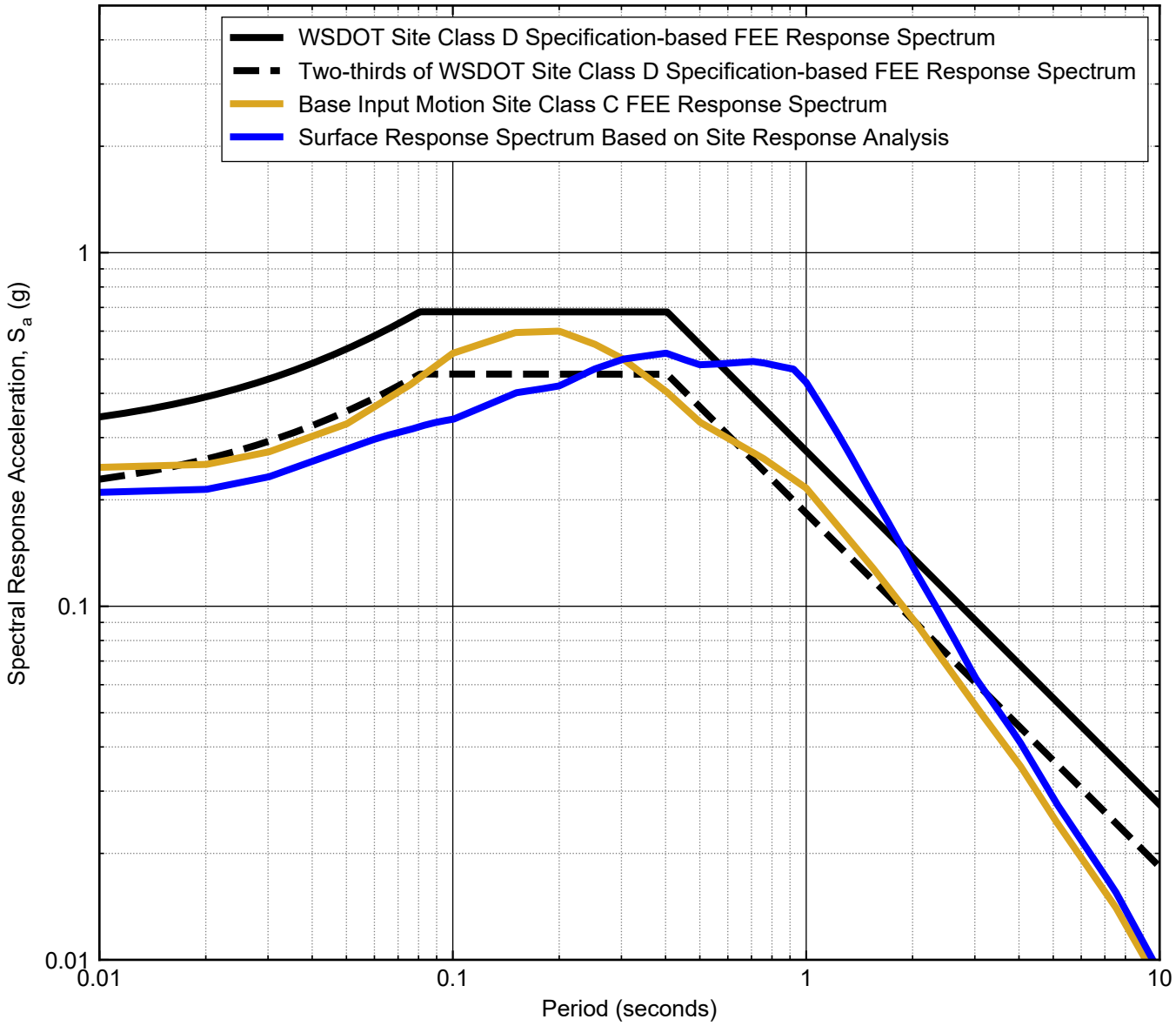

Note:
1. FEE = Functional Evaluation Earthquake; g = acceleration of gravity; WSDOT = Washington State Department of Transportation.

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WSDOT FEE
SITE RESPONSE ANALYSIS
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FIG. A-44

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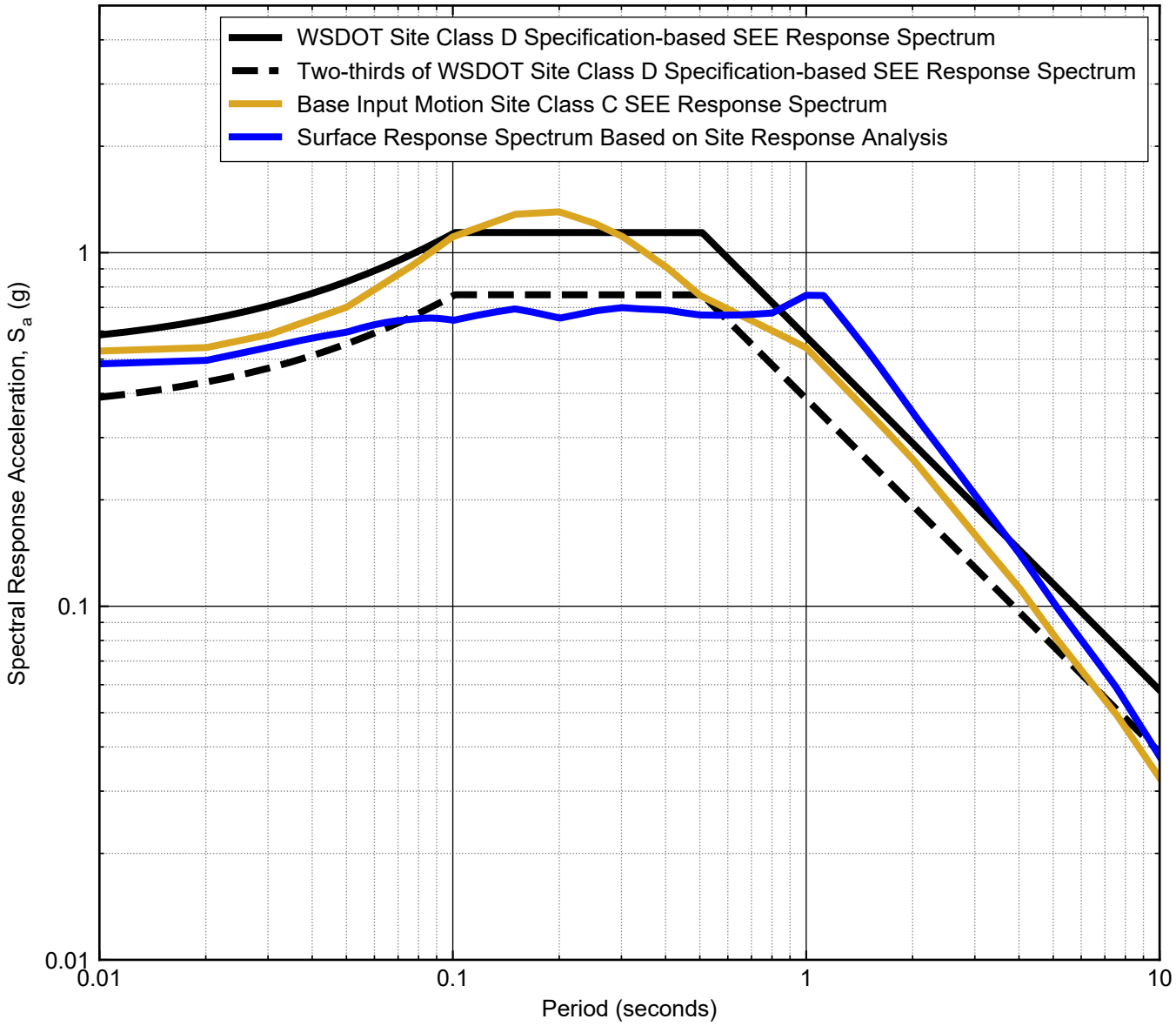

Note:
1. g = acceleration of gravity; SEE = Safety Evaluation Earthquake; WSDOT = Washington State Department of Transportation.

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

WSDOT SEE
SITE RESPONSE ANALYSIS
SURFACE RESPONSE SPECTRUM

October 2020 105492-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-45

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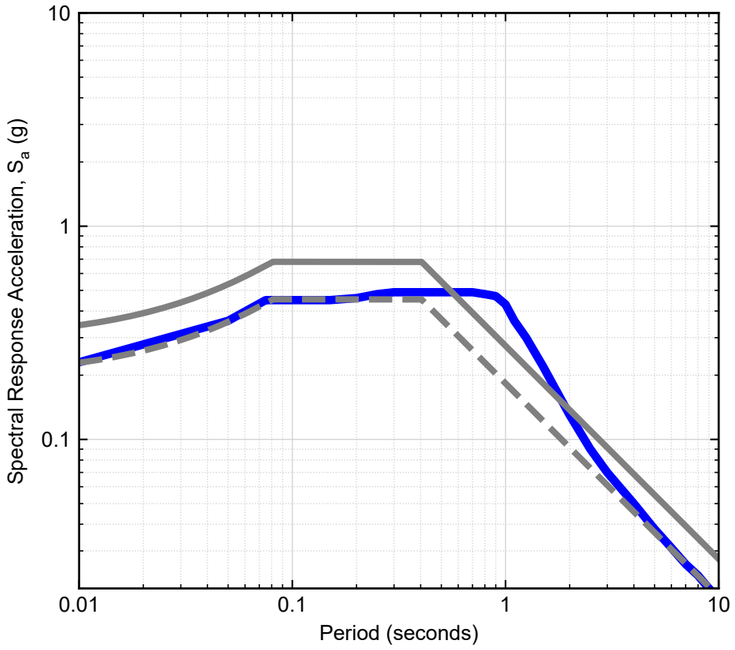

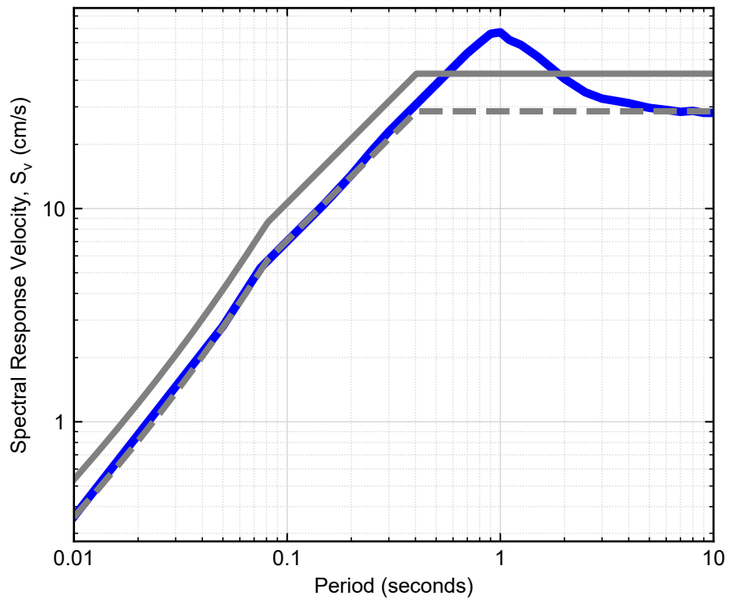

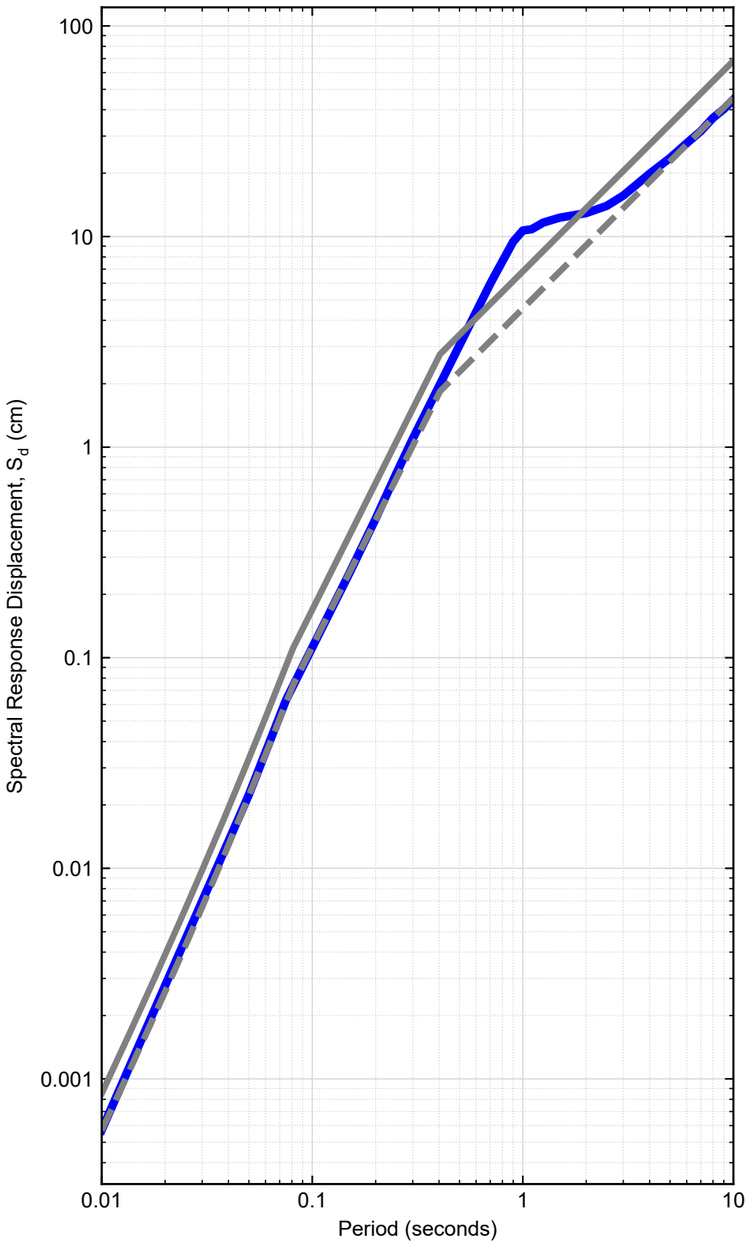

Recommended FEE Design Response Spectrum
WSDOT Site Class D Specification-based FEE Response Spectrum
Two-thirds of WSDOT Site Class D Specification-based FEE Design Spectrum

Notes:
1. cm = centimeters; cm/s = centimeters per second; FEE = Functional Evaluation Earthquake; g = acceleration of gravity; WSDOT = Washington State Department of Transportation.
2. The recommended design response spectrum was evaluated based on the WSDOT Geotechnical Design Manual and the results of our site response analyses.

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

ACCELERATION, VELOCITY, AND DISLACEMENT WSDOT FEE DESIGN RESPONSE SPECTRUM

October 2020 105492-001

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FIG. A-46

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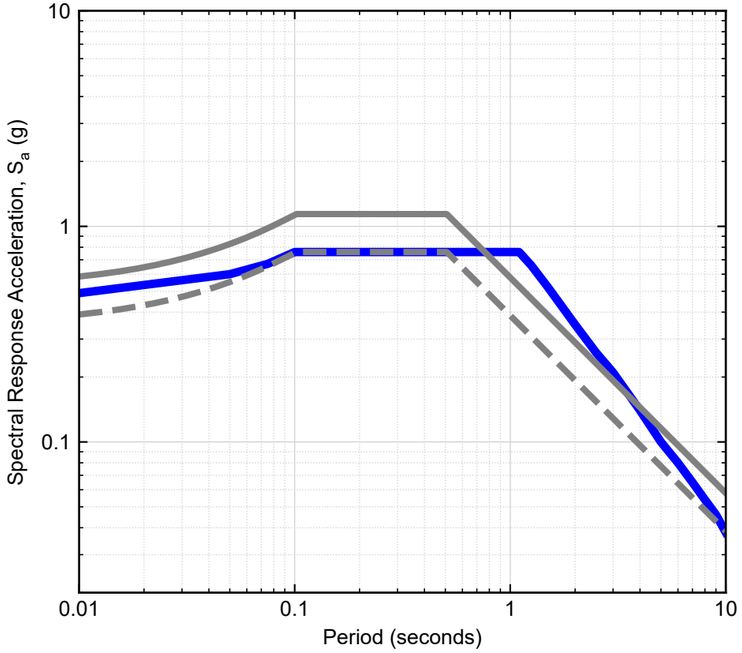

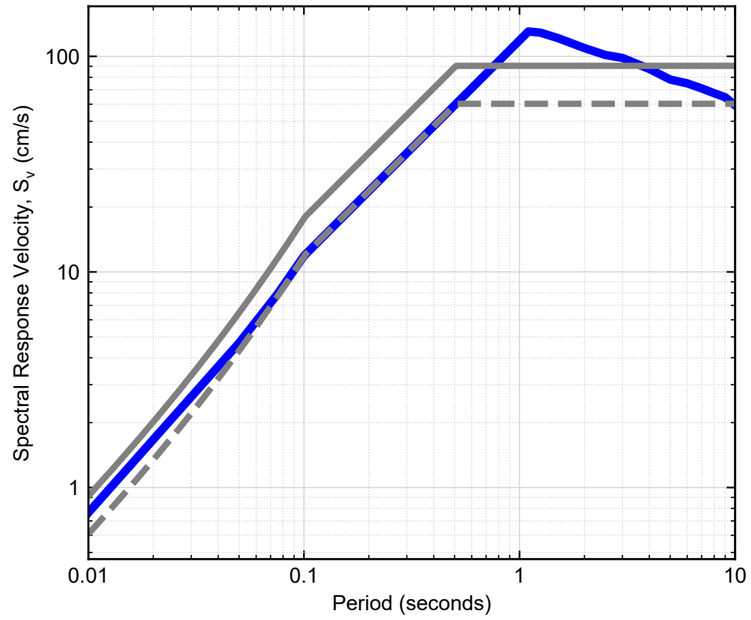

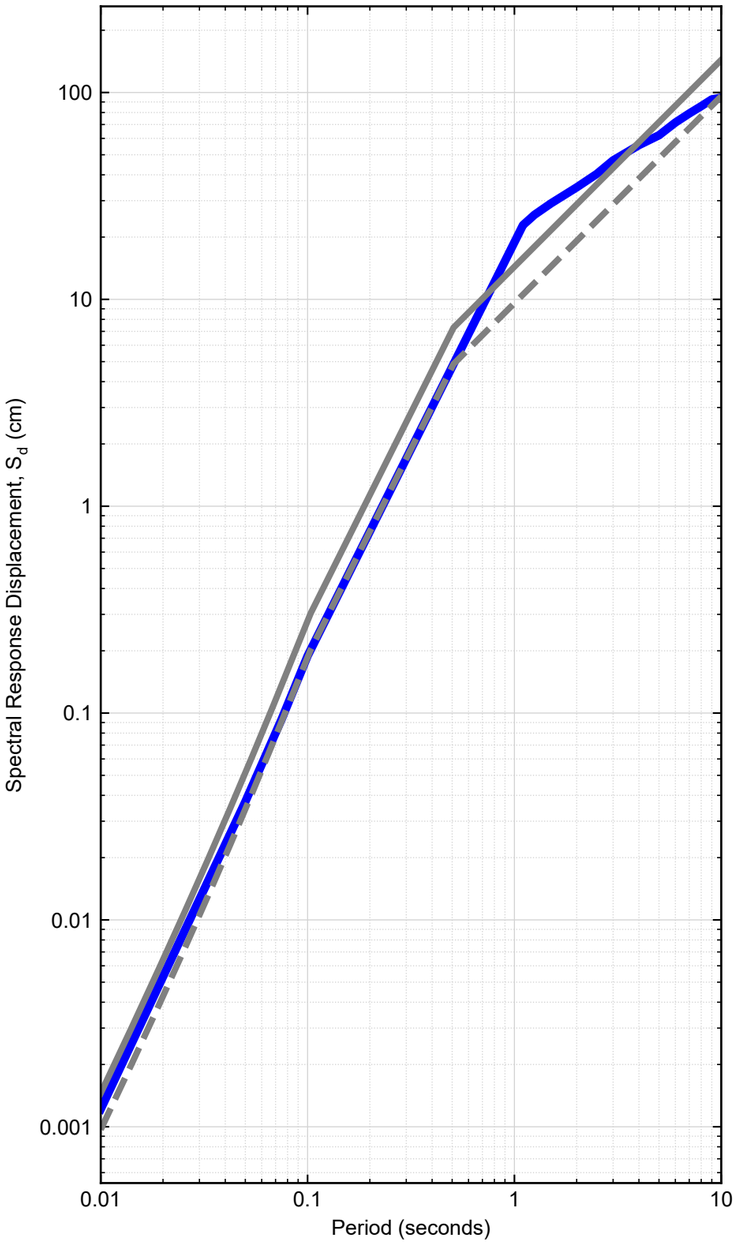

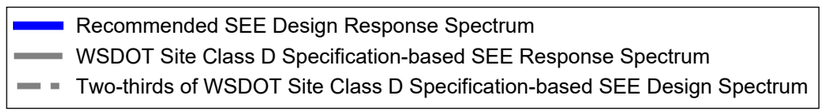

Notes:
1. cm = centimeters; cm/s = centimeters per second; g = acceleration of gravity; SEE = Safety Evaluation Earthquake; WSDOT = Washington State Department of Transportation.
2. The recommended design response spectrum was evaluated based on the WSDOT Geotechnical Design Manual and the results of our site response analyses.

SDOT
West Seattle Bridge Replacement Design
Seattle, Washington

ACCELERATION, VELOCITY, AND DISLACEMENT WSDOT SEE DESIGN RESPONSE SPECTRUM

October 2020 105492-001

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Geotechnical and Environmental Consultants

FIG. A-47

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IMPORTANT INFORMATION

Important Information

About Your Geotechnical Report

IMPORTANT INFORMATION

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

IMPORTANT INFORMATION

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 C

ROUGH ORDER OF MAGNITUDE COST BACKUP

APPENDIX C

ROUGH ORDER OF MAGNITUDE COST BACKUP

West Seattle High Rise Bridge

Capital Cost Estimate Summary

Revision Date: October 4, 2020

No.	Item Description		Alternative 1 Shore Restore LL Cost	Alternative 2 Direct Rehabilitation Cost	Alternative 4 Super Replacement (Concrete) Cost	Alternative 5 In-Kind Replacement Cost	Alternative 6 Tunnel Cost
I. CONSTRUCTION							
1.0	PREPARATION		$ 1,600,000	$ 900,000	$ 23,800,000	$ 29,000,000	$ -
2.0	CIVIL/ROADWAY		$ 6,042,000	$ 3,420,000	$ 20,454,000	$ 31,114,000	$ -
3.0	UTILITIES		$ 1,590,000	$ 900,000	$ 5,383,000	$ 8,188,000	$ -
4.0	STRUCTURE		$ 31,800,000	$ 18,000,000	$ 107,650,000	$ 163,754,000	$ -
5.0	TRAFFIC		$ 1,790,000	$ 664,000	$ 2,680,000	$ 2,960,000	$ -
6.0	MISCELLANEOUS CONSTRUCTION		$ 4,770,000	$ 2,700,000	$ 16,148,000	$ 24,564,000	$ -
	Construction without Mobilization		$ 47,592,000	$ 26,584,000	$ 176,115,000	$ 259,580,000	$ 814,000,000
	Mobilization	10%	$ 4,759,200	$ 2,658,400	$ 17,611,500	$ 25,958,000	$ 81,400,000
	Construction including Mobilization		$ 52,351,200	$ 29,242,400	$ 193,726,500	$ 285,538,000	$ 895,400,000
	Inflation						
	(Assume Construction in 2021)	3.0%	$ 1,570,536	$ 877,272	$ 5,811,795	$ 8,566,140	$ 26,862,000
	Construction Subtotal		$ 53,921,736	$ 30,119,672	$ 199,538,295	$ 294,104,140	$ 922,262,000
		Alt 1, 4,5 / Alt 2 / Alt 6					
	Design Contingency	40% / 20% / 40%	$ 21,568,694	$ 6,023,934.40	$ 79,815,318	$ 117,641,656	$ 368,904,800
	Allowance	20% / 10% / 40%	$ 10,784,347	$ 3,011,967	$ 39,907,659	$ 58,820,828	$ 368,904,800
	Subtotal Incl. Design Conting. & Allow.		$ 86,274,778	$ 39,155,574	$ 319,261,272	$ 470,566,624	$ 1,660,071,600
	Constr. Contingency	20%	$ 17,254,956	$ 7,831,115	$ 63,852,254	$ 94,113,325	$ 332,014,320
	Subtotal Incl. Constr. Contingency		$ 103,529,733	$ 46,986,688	$ 383,113,526	$ 564,679,949	$ 1,992,085,920
	Risk — Base Risk	0%	$ -	$ -	$ -	$ -	$ -
	Identfied Risk Item 1		$ -	$ 175,500,000	$ 229,500,000	$ 473,500,000	$ 269,000,000
	Identfied Risk Item 2		$ -	$ -			
	Identfied Risk Item 3						
	Identfied Risk Item 4						
	Subtotal Risk		$ -	$ 175,500,000	$ 229,500,000	$ 473,500,000	$ 269,000,000
	Construction Subtotal Incl. Risk		$ 103,529,733	$ 222,486,688	$ 612,613,526	$ 1,038,179,949	$ 2,261,085,920
	Sales Tax	0.00%	$ -	$ -	$ -	$ -	$ -
	Subtotal		$ 103,529,733	$ 222,486,688	$ 612,613,526	$ 1,038,179,949	$ 2,261,085,920
CONSTRUCTION TOTAL			**$ 103,530,000**	**$ 222,487,000**	**$ 612,614,000**	**$ 1,038,180,000**	**$ 2,261,086,000**

No.	Item Description	Alternative 1 Cost	Alternative 2 Cost	Alternative 4 Cost	Alternative 5 Cost	Alternative 6 Cost
II. RIGHT OF WAY						
1.0	Riverside Mill	$ 470,434	$ 470,434	$ 480,183	$ 470,434	$ -
2.0	3601 W Marg LLC	$ 14,140	$ 2,168	$ -	$ -	$ -
3.0	BNSF	$ 64,224	$ 66,250	$ 65,555	$ 64,224	$ -
4.0	DNR	$ 25,471	$ 66,250	$ 308,684	$ 165,913	$ -
5.0	Fee Acq. Riverside Mill, 5,000sf@$60psf	$ 300,000	$ 300,000	$ 300,000	$ 300,000	$ -
RIGHT OF WAY TOTAL		**$ 875,000**	**$ 906,000**	**$ 1,155,000**	**$ 1,001,000**	**$ -**

No.	Item Description		Alternative 1 Cost	Alternative 2 Cost	Alternative 4 Cost	Alternative 5 Cost	Alternative 6 Cost
III. OTHER VARIABLE							
	Other Variable	20%	$ 20,706,000	$ 10,032,400	$ 122,522,800	$ 207,636,000	$ 452,217,200
OTHER VARIABLE TOTAL			**$ 20,706,000**	**$ 10,033,000**	**$ 122,523,000**	**$ 207,636,000**	**$ 452,218,000**

CONSTRUCTION TOTAL	$ 103,530,000	$ 222,487,000	$ 612,614,000	$ 1,038,180,000	$ 2,261,086,000
RIGHT OF WAY TOTAL	$ 875,000	$ 906,000	$ 1,155,000	$ 1,001,000	$ -
OTHER VARIABLE TOTAL	$ 20,706,000	$ 10,033,000	$ 122,523,000	$ 207,636,000	$ 452,218,000
TOTAL	**$ 125,111,000**	**$ 233,426,000**	**$ 736,292,000**	**$ 1,246,817,000**	**$ 2,713,304,000**

LIFE CYCLE COST ESTIMATE

ROUGH ORDER OF MAGNITUDE COST BACKUP

TABLE - WSHB LIFE CYCLE COST ANALYSIS

UNADJUSTED (0% Net Rate)

Item	Alternative 1 Shore Restore LL	Alternative 2 Direct Rehabilitation	Alternative 4 Super Replacement (Concrete)	Alternative 5 In-Kind Replacement	Alternative 6 Tunnel
Initial Capital Investment	$ 125,111,000	$ 233,426,000	$ 736,292,000	$ 1,246,817,000	$ 2,713,304,000
Total Unadjusted Value of Maint.,Inspect.	$ 32,285,000	$ 40,510,000	$ 22,075,000	$ 29,570,000	$ 110,303,030
Total Unadjusted Value of Remaining Service Life	$ (124,443,200)	$ (637,771,400)	$ -	$ (15,614,867)	$ (217,064,320)
Total Unadjusted Value of Rehab.	$ 1,525,922,000	$ 1,279,809,000	$ 247,319,000	$ 281,951,000	$ 214,495,726
Total Unadjusted Value of Maint.,Inspect.,Rehab.	$ 1,433,763,800	$ 682,547,600	$ 269,394,000	$ 295,906,133	$ 107,734,436
Estimated Total Ownership Costs Unadjusted Value	$ 1,558,874,800	$ 915,973,600	$ 1,005,686,000	$ 1,542,723,133	$ 2,821,038,436

ADJUSTED (-3% Net Rate)

Item	Alternative 1 Shore Restore LL	Alternative 2 Direct Rehabilitation	Alternative 4 Super Replacement (Concrete)	Alternative 5 In-Kind Replacement	Alternative 6 Tunnel
Initial Capital Investment	$ 116,004,978	$ 216,336,066	$ 652,438,920	$ 1,104,822,458	$ 2,303,582,406
Total Adjusted Value of Maint.,Inspect.	$ 12,673,106	$ 16,990,111	$ 7,371,354	$ 10,008,954	$ 33,116,070
Total Adjusted Value of Remaining Service Life	$ (10,882,030)	$ (55,770,403)	$ -	$ (1,365,454)	$ (18,981,354)
Total Adjusted Value of Rehab.	$ 931,547,647	$ 340,421,416	$ 44,923,605	$ 52,520,307	$ 64,397,645
Total Adjusted Value of Maint.,Inspect.,Rehab.	$ 933,338,723	$ 301,641,125	$ 52,294,959	$ 61,163,807	$ 78,532,361
Estimated Total Ownership Costs Adjusted Value	$ 1,049,343,700	$ 517,977,190	$ 704,733,880	$ 1,165,986,266	$ 2,382,114,767

*Followed the guidelines from the FHWA Life-Cycle Cost Analysis Primer

*** The Initial Capital Investment (Raw) values for the five Alternatives, do not reflect the actual costs of the alternatives. Currently, the values placed in the spreadsheet are there to show how the spreadsheet works. To use this spreadsheet, the Intitial Capital Investment (Raw) value should come from the Capital

		Net Rate =	-3.00%

		Alternative 1 - Shore Restore LL							
		Maintenance (allowable annual)		Inspections		Rehabilitation/Replacement		Remaining Service Life (neg.) (end of design life)	
		% Initial Cost		% Initial Cost		% Initial Cost		% Initial Cost	
Year	Life Year	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted
2021	1	$ 590,000	$ 572,300	$150,000	$ 145,500	$ 31,277,750	$ 30,339,418		
2022	2	$ 590,000	$ 555,131	$150,000	$ 141,135	$ 31,277,750	$ 29,429,235		
2023	3	$ 590,000	$ 538,477	$150,000	$ 136,901	$ 31,277,750	$ 28,546,358		
2024	4	$ 590,000	$ 522,323	$150,000	$ 132,794	$ 31,277,750	$ 27,689,967		
2025	5	$ 590,000	$ 506,653	$150,000	$ 128,810		$ -		
2026	6	$ 590,000	$ 491,453	$150,000	$ 124,946		$ -		
2027	7	$ 590,000	$ 476,710	$150,000	$ 121,197		$ -		
2028	8	$ 590,000	$ 462,409	$150,000	$ 117,562		$ -		
2029	9		$ -		$ -	$ 88,860,000	$ 67,554,132		
2030	10		$ -		$ -	$ 291,663,750	$ 215,079,886		
2031	11		$ -		$ -	$ 291,663,750	$ 208,627,490		
2032	12		$ -		$ -	$ 291,663,750	$ 202,368,665		
2033	13		$ -		$ -	$ 291,663,750	$ 196,297,605		
2034	14	$295,000	$ 192,587	$200,000	$ 130,567		$ -		
2035	15	$295,000	$ 186,809		$ -		$ -		
2036	16	$295,000	$ 181,205	$200,000	$ 122,851		$ -		
2037	17	$295,000	$ 175,769		$ -		$ -		
2038	18	$295,000	$ 170,496	$200,000	$ 115,590		$ -		
2039	19	$295,000	$ 165,381		$ -		$ -		
2040	20	$295,000	$ 160,419	$200,000	$ 108,759		$ -		
2041	21	$295,000	$ 155,607		$ -		$ -		
2042	22	$295,000	$ 150,939	$200,000	$ 102,331		$ -		
2043	23	$295,000	$ 146,410		$ -		$ -		
2044	24	$295,000	$ 142,018	$200,000	$ 96,283		$ -		
2045	25	$295,000	$ 137,758		$ -		$ -		
2046	26	$295,000	$ 133,625	$200,000	$ 90,593		$ -		
2047	27	$295,000	$ 129,616		$ -		$ -		
2048	28	$295,000	$ 125,728	$200,000	$ 85,239		$ -		
2049	29	$295,000	$ 121,956		$ -		$ -		
2050	30	$295,000	$ 118,297	$200,000	$ 80,201		$ -		
2051	31	$295,000	$ 114,748		$ -		$ -		
2052	32	$295,000	$ 111,306	$200,000	$ 75,462		$ -		
2053	33	$295,000	$ 107,967		$ -		$ -		
2054	34	$295,000	$ 104,728	$200,000	$ 71,002		$ -		
2055	35	$295,000	$ 101,586		$ -		$ -		
2056	36	$295,000	$ 98,538	$200,000	$ 66,806		$ -		
2057	37	$295,000	$ 95,582		$ -		$ -		
2058	38	$295,000	$ 92,715	$200,000	$ 62,857	$ 11,544,000	$ 3,628,125		
2059	39	$295,000	$ 89,933		$ -		$ -		
2060	40	$295,000	$ 87,235	$200,000	$ 59,142		$ -		
2061	41	$295,000	$ 84,618		$ -		$ -		
2062	42	$295,000	$ 82,080	$200,000	$ 55,647		$ -		
2063	43	$295,000	$ 79,617		$ -		$ -		
2064	44	$295,000	$ 77,229	$200,000	$ 52,358		$ -		
2065	45	$295,000	$ 74,912		$ -		$ -		
2066	46	$295,000	$ 72,664	$200,000	$ 49,264		$ -		
2067	47	$295,000	$ 70,484		$ -		$ -		
2068	48	$295,000	$ 68,370	$200,000	$ 46,353		$ -		
2069	49	$295,000	$ 66,319		$ -		$ -		
2070	50	$295,000	$ 64,329	$200,000	$ 43,613		$ -		
2071	51	$295,000	$ 62,399		$ -		$ -		
2072	52	$295,000	$ 60,527	$200,000	$ 41,036		$ -		
2073	53	$295,000	$ 58,712		$ -		$ -		
2074	54	$295,000	$ 56,950	$200,000	$ 38,610		$ -		
2075	55	$295,000	$ 55,242		$ -		$ -		
2076	56	$295,000	$ 53,584	$200,000	$ 36,328		$ -		
2077	57	$295,000	$ 51,977		$ -		$ -		
2078	58	$295,000	$ 50,418	$200,000	$ 34,181		$ -		
2079	59	$295,000	$ 48,905		$ -		$ -		
2080	60	$295,000	$ 47,438	$200,000	$ 32,161		$ -		
2081	61	$295,000	$ 46,015		$ -		$ -		
2082	62	$295,000	$ 44,634	$200,000	$ 30,261		$ -		
2083	63	$295,000	$ 43,295		$ -	$ 258,863,000	$ 37,991,745		
2084	64	$295,000	$ 41,996	$200,000	$ 28,472		$ -		
2085	65	$295,000	$ 40,737		$ -		$ -		
2086	66	$295,000	$ 39,514	$200,000	$ 26,789		$ -		
2087	67	$295,000	$ 38,329		$ -		$ -		
2088	68	$295,000	$ 37,179	$200,000	$ 25,206		$ -		
2089	69	$295,000	$ 36,064		$ -		$ -		
2090	70	$295,000	$ 34,982	$200,000	$ 23,717		$ -		
2091	71	$295,000	$ 33,932		$ -		$ -		
2092	72	$295,000	$ 32,914	$200,000	$ 22,315		$ -		
2093	73	$295,000	$ 31,927		$ -		$ -		
2094	74	$295,000	$ 30,969	$200,000	$ 20,996		$ -		
2095	75	$295,000	$ 30,040		$ -		$ -		
2096	76	$295,000	$ 29,139	$200,000	$ 19,755		$ -		
2097	77	$295,000	$ 28,265		$ -		$ -		
2098	78	$295,000	$ 27,417	$200,000	$ 18,588		$ -		
2099	79	$295,000	$ 26,594		$ -		$ -		
2100	80	$295,000	$ 25,796		$ -		$ -	**$ (124,443,200.00)**	**$ (10,882,030)**
	Sum =	$ 24,485,000	$ 9,710,926	$ 7,800,000	$ 2,962,180	$ 1,651,033,000	$ 1,047,552,625	$ (124,443,200)	$ (10,882,030)

	Unadjusted	Adjusted
Total Costs Maint., Inspect. =	$ 32,285,000	$ 12,673,106
Total Remaining Service Life Cost =	$ (124,443,200)	$ (10,882,030)
Total Costs Initial Investment & Repair/Rehab =	$ 1,651,033,000	$ 1,047,552,625
Estimated Total Ownership Cost =	**$ 1,558,874,800**	**$ 1,049,343,700**

		Net Rate =	-3.00%

Alternative 2 - Direct Rehabilitation

		Maintenance (allowable annual)		Inspections		Rehabilitation/Replacement		Remaining Service Life (neg.) (end of design life)	
		% Initial Cost		% Initial Cost		% Initial Cost		% Initial Cost	
Year	Life Year	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted
2021	1	$490,000	$ 475,300	$150,000	$ 145,500	$ -	$ -		
2022	2	$490,000	$ 461,041	$150,000	$ 141,135	$ 116,713,000	$ 109,815,262		
2023	3	$490,000	$ 447,210	$150,000	$ 136,901	$ 116,713,000	$ 106,520,804		
2024	4	$490,000	$ 433,793	$150,000	$ 132,794	$ -	$ -		
2025	5	$490,000	$ 420,780	$150,000	$ 128,810	$ -	$ -		
2026	6	$490,000	$ 408,156	$150,000	$ 124,946	$ -	$ -		
2027	7	$490,000	$ 395,912	$150,000	$ 121,197	$ -	$ -		
2028	8	$490,000	$ 384,034	$150,000	$ 117,562	$ -	$ -		
2029	9	$490,000	$ 372,513	$150,000	$ 114,035	$ -	$ -		
2030	10	$490,000	$ 361,338	$150,000	$ 110,614	$300,000	$ 221,227		
2031	11	$490,000	$ 350,498	$150,000	$ 107,295	$ -	$ -		
2032	12	$490,000	$ 339,983	$150,000	$ 104,076	$ 24,600,000	$ 17,068,522		
2033	13	$490,000	$ 329,783	$150,000	$ 100,954	$ -	$ -		
2034	14	$490,000	$ 319,890	$150,000	$ 97,925	$ -	$ -		
2035	15	$490,000	$ 310,293	$150,000	$ 94,988	$ -	$ -		
2036	16	$490,000	$ 300,984	$150,000	$ 92,138	$ -	$ -		
2037	17	$490,000	$ 291,955	$150,000	$ 89,374	$ -	$ -		
2038	18	$490,000	$ 283,196	$150,000	$ 86,693	$ -	$ -		
2039	19	$490,000	$ 274,700	$150,000	$ 84,092	$ -	$ -		
2040	20	$490,000	$ 266,459	$150,000	$ 81,569	$300,000	$ 163,138		
2041	21	$490,000	$ 258,465	$150,000	$ 79,122	$ -	$ -		
2042	22	$490,000	$ 250,711	$150,000	$ 76,748	$ -	$ -		
2043	23	$490,000	$ 243,190	$150,000	$ 74,446	$ -	$ -		
2044	24	$490,000	$ 235,894	$150,000	$ 72,213	$ -	$ -		
2045	25	$490,000	$ 228,818	$150,000	$ 70,046	$ -	$ -		
2046	26	$490,000	$ 221,953	$150,000	$ 67,945	$ -	$ -		
2047	27	$490,000	$ 215,294	$150,000	$ 65,906	$ -	$ -		
2048	28	$490,000	$ 208,836	$150,000	$ 63,929	$ -	$ -		
2049	29	$490,000	$ 202,571	$150,000	$ 62,011	$ -	$ -		
2050	30	$490,000	$ 196,493	$150,000	$ 60,151	$300,000	$ 120,302		
2051	31	$490,000	$ 190,599	$150,000	$ 58,347	$ -	$ -		
2052	32	$490,000	$ 184,881	$150,000	$ 56,596	$ -	$ -		
2053	33	$490,000	$ 179,334	$150,000	$ 54,898	$ -	$ -		
2054	34	$490,000	$ 173,954	$150,000	$ 53,251	$ -	$ -		
2055	35	$490,000	$ 168,736	$150,000	$ 51,654	$ -	$ -		
2056	36	$490,000	$ 163,674	$150,000	$ 50,104	$ -	$ -		
2057	37	$490,000	$ 158,763	$150,000	$ 48,601	$ -	$ -		
2058	38	$490,000	$ 154,000	$150,000	$ 47,143	$ -	$ -		
2059	39	$490,000	$ 149,380	$150,000	$ 45,729	$ -	$ -		
2060	40	$490,000	$ 144,899	$150,000	$ 44,357	$ -	$ -		
2061	41	$490,000	$ 140,552	$150,000	$ 43,026	$ -	$ -		
2062	42	$490,000	$ 136,335	$150,000	$ 41,735	$ -	$ -		
2063	43		$ -		$ -	$ 367,773,750	$ 99,257,950		
2064	44		$ -		$ -	$ 291,663,750	$ 76,355,225		
2065	45		$ -		$ -	$ 291,663,750	$ 74,064,569		
2066	46		$ -		$ -	$ 291,663,750	$ 71,842,632		
2067	47	$295,000	$ 70,484	$200,000	$ 47,786	$ -	$ -		
2068	48	$295,000	$ 68,370		$ -	$ -	$ -		
2069	49	$295,000	$ 66,319	$200,000	$ 44,962	$ -	$ -		
2070	50	$295,000	$ 64,329		$ -	$ -	$ -		
2071	51	$295,000	$ 62,399	$200,000	$ 42,305	$ -	$ -		
2072	52	$295,000	$ 60,527		$ -	$ -	$ -		
2073	53	$295,000	$ 58,712	$200,000	$ 39,804	$ -	$ -		
2074	54	$295,000	$ 56,950		$ -	$ -	$ -		
2075	55	$295,000	$ 55,242	$200,000	$ 37,452	$ -	$ -		
2076	56	$295,000	$ 53,584		$ -	$ -	$ -		
2077	57	$295,000	$ 51,977	$200,000	$ 35,239	$ -	$ -		
2078	58	$295,000	$ 50,418		$ -	$ -	$ -		
2079	59	$295,000	$ 48,905	$200,000	$ 33,156	$ -	$ -		
2080	60	$295,000	$ 47,438		$ -	$ -	$ -		
2081	61	$295,000	$ 46,015	$200,000	$ 31,196	$ -	$ -		
2082	62	$295,000	$ 44,634		$ -	$ -	$ -		
2083	63	$295,000	$ 43,295	$200,000	$ 29,353	$ -	$ -		
2084	64	$295,000	$ 41,996		$ -	$ -	$ -		
2085	65	$295,000	$ 40,737	$200,000	$ 27,618	$ -	$ -		
2086	66	$295,000	$ 39,514		$ -	$ -	$ -		
2087	67	$295,000	$ 38,329	$200,000	$ 25,986	$ -	$ -		
2088	68	$295,000	$ 37,179		$ -	$ -	$ -		
2089	69	$295,000	$ 36,064	$200,000	$ 24,450	$ -	$ -		
2090	70	$295,000	$ 34,982		$ -	$ -	$ -		
2091	71	$295,000	$ 33,932	$200,000	$ 23,005	$ 11,544,000	$ 1,327,851		
2092	72	$295,000	$ 32,914		$ -	$ -	$ -		
2093	73	$295,000	$ 31,927	$200,000	$ 21,645	$ -	$ -		
2094	74	$295,000	$ 30,969		$ -	$ -	$ -		
2095	75	$295,000	$ 30,040	$200,000	$ 20,366	$ -	$ -		
2096	76	$295,000	$ 29,139		$ -	$ -	$ -		
2097	77	$295,000	$ 28,265	$200,000	$ 19,163	$ -	$ -		
2098	78	$295,000	$ 27,417		$ -	$ -	$ -		
2099	79	$295,000	$ 26,594	$200,000	$ 18,030	$ -	$ -		
2100	80	$295,000	$ 25,796	$200,000	$ 17,489		$ -	$ (637,771,400.00)	$ (55,770,403)
	Sum =	$ 30,610,000	$ 12,950,549	$ 9,900,000	$ 4,039,562	$ 1,513,235,000	$ 556,757,482	$ (637,771,400)	$ (55,770,403)

	Unadjusted	Adjusted
Total Costs Maint., Inspect. =	$ 40,510,000	$ 16,990,111
Total Remaining Service Life Cost =	$ (637,771,400)	$ (55,770,403)
Total Costs Initial Investment & Repair/Rehab =	$ 1,513,235,000	$ 556,757,482
Estimated Total Ownership Cost =	**$ 915,973,600**	**$ 517,977,190**

				Net Rate =	-3.00%				
		Alternative 4 - Super Replacement (Concrete)							
		Maintenance (allowable annual)		Inspections		Rehabilitation/Replacement		Remaining Service Life (neg.) (end of design life)	
		% Initial Cost		% Initial Cost		% Initial Cost		% Initial Cost	
Year	Life Year	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted
2021	1	$390,000	$ 378,300	$150,000	$ 145,500	$ -	$ -		
2022	2		$ -		$ -	$ 147,258,400	$ 138,555,429		
2023	3		$ -		$ -	$ 147,258,400	$ 134,398,766		
2024	4		$ -		$ -	$ 147,258,400	$ 130,366,803		
2025	5		$ -		$ -	$ 147,258,400	$ 126,455,799		
2026	6	$245,000	$ 204,078	$100,000	$ 83,297	$ 147,258,400	$ 122,662,125		
2027	7	$245,000	$ 197,956		$ -	$ -	$ -		
2028	8	$245,000	$ 192,017	$100,000	$ 78,374	$ -	$ -		
2029	9	$245,000	$ 186,257		$ -	$ -	$ -		
2030	10	$245,000	$ 180,669	$100,000	$ 73,742	$ -	$ -		
2031	11	$245,000	$ 175,249		$ -	$ -	$ -		
2032	12	$245,000	$ 169,991	$100,000	$ 69,384	$ -	$ -		
2033	13	$245,000	$ 164,892		$ -	$ -	$ -		
2034	14	$245,000	$ 159,945	$100,000	$ 65,284	$ -	$ -		
2035	15	$245,000	$ 155,147		$ -	$ -	$ -		
2036	16	$245,000	$ 150,492	$100,000	$ 61,425	$ -	$ -		
2037	17	$245,000	$ 145,977		$ -	$ -	$ -		
2038	18	$245,000	$ 141,598	$100,000	$ 57,795	$ -	$ -		
2039	19	$245,000	$ 137,350		$ -	$ -	$ -		
2040	20	$245,000	$ 133,230	$100,000	$ 54,379	$ -	$ -		
2041	21	$245,000	$ 129,233		$ -	$ -	$ -		
2042	22	$245,000	$ 125,356	$100,000	$ 51,166	$ -	$ -		
2043	23	$245,000	$ 121,595		$ -	$ -	$ -		
2044	24	$245,000	$ 117,947	$100,000	$ 48,142	$ -	$ -		
2045	25	$245,000	$ 114,409		$ -	$ -	$ -		
2046	26	$245,000	$ 110,977	$100,000	$ 45,297	$ -	$ -		
2047	27	$245,000	$ 107,647		$ -	$ -	$ -		
2048	28	$245,000	$ 104,418	$100,000	$ 42,620	$ -	$ -		
2049	29	$245,000	$ 101,285		$ -	$ -	$ -		
2050	30	$245,000	$ 98,247	$100,000	$ 40,101	$ -	$ -		
2051	31	$245,000	$ 95,299		$ -	$ -	$ -		
2052	32	$245,000	$ 92,440	$100,000	$ 37,731	$ -	$ -		
2053	33	$245,000	$ 89,667		$ -	$ -	$ -		
2054	34	$245,000	$ 86,977	$100,000	$ 35,501	$ -	$ -		
2055	35	$245,000	$ 84,368		$ -	$ -	$ -		
2056	36	$245,000	$ 81,837	$100,000	$ 33,403	$ -	$ -		
2057	37	$245,000	$ 79,382		$ -	$ -	$ -		
2058	38	$245,000	$ 77,000	$100,000	$ 31,429	$ -	$ -		
2059	39	$245,000	$ 74,690		$ -	$ -	$ -		
2060	40	$245,000	$ 72,450	$100,000	$ 29,571	$ -	$ -		
2061	41	$245,000	$ 70,276		$ -	$ -	$ -		
2062	42	$245,000	$ 68,168	$100,000	$ 27,824	$ -	$ -		
2063	43	$245,000	$ 66,123		$ -	$ -	$ -		
2064	44	$245,000	$ 64,139	$100,000	$ 26,179	$ -	$ -		
2065	45	$245,000	$ 62,215		$ -	$ -	$ -		
2066	46	$245,000	$ 60,348	$100,000	$ 24,632	$ -	$ -		
2067	47	$245,000	$ 58,538		$ -	$ -	$ -		
2068	48	$245,000	$ 56,782	$100,000	$ 23,176	$ -	$ -		
2069	49	$245,000	$ 55,078		$ -	$ -	$ -		
2070	50	$245,000	$ 53,426	$100,000	$ 21,807	$ -	$ -		
2071	51	$245,000	$ 51,823		$ -	$ -	$ -		
2072	52	$245,000	$ 50,269	$100,000	$ 20,518	$ -	$ -		
2073	53	$245,000	$ 48,760		$ -	$ -	$ -		
2074	54	$245,000	$ 47,298	$100,000	$ 19,305	$ -	$ -		
2075	55	$245,000	$ 45,879		$ -	$ -	$ -		
2076	56	$245,000	$ 44,502	$100,000	$ 18,164	$ 247,319,000	$ 44,923,605		
2077	57	$245,000	$ 43,167		$ -	$ -	$ -		
2078	58	$245,000	$ 41,872	$100,000	$ 17,091	$ -	$ -		
2079	59	$245,000	$ 40,616		$ -	$ -	$ -		
2080	60	$245,000	$ 39,398	$100,000	$ 16,081	$ -	$ -		
2081	61	$245,000	$ 38,216		$ -	$ -	$ -		
2082	62	$245,000	$ 37,069	$100,000	$ 15,130	$ -	$ -		
2083	63	$245,000	$ 35,957		$ -	$ -	$ -		
2084	64	$245,000	$ 34,878	$100,000	$ 14,236	$ -	$ -		
2085	65	$245,000	$ 33,832		$ -	$ -	$ -		
2086	66	$245,000	$ 32,817	$100,000	$ 13,395	$ -	$ -		
2087	67	$245,000	$ 31,833		$ -	$ -	$ -		
2088	68	$245,000	$ 30,878	$100,000	$ 12,603	$ -	$ -		
2089	69	$245,000	$ 29,951		$ -	$ -	$ -		
2090	70	$245,000	$ 29,053	$100,000	$ 11,858	$ -	$ -		
2091	71	$245,000	$ 28,181		$ -	$ -	$ -		
2092	72	$245,000	$ 27,336	$100,000	$ 11,157	$ -	$ -		
2093	73	$245,000	$ 26,516		$ -	$ -	$ -		
2094	74	$245,000	$ 25,720	$100,000	$ 10,498	$ -	$ -		
2095	75	$245,000	$ 24,949		$ -	$ -	$ -		
2096	76	$245,000	$ 24,200	$100,000	$ 9,878	$ -	$ -		
2097	77	$245,000	$ 23,474		$ -	$ -	$ -		
2098	78	$245,000	$ 22,770	$100,000	$ 9,294	$ -	$ -		
2099	79	$245,000	$ 22,087		$ -	$ -	$ -		
2100	80	$245,000	$ 21,424		$ -	$ -	$ -		
	Sum =	$ 18,375,000	$ 6,109,889	$ 3,700,000	$ 1,261,466	$ 983,611,000	$ 697,362,525	$ -	$ -

	Unadjusted	Adjusted
Total Costs Maint., Inspect. =	$ 22,075,000	$ 7,371,354
Total Remaining Service Life Cost =	$ -	$ -
Total Costs Initial Investment & Repair/Rehab =	$ 983,611,000	$ 697,362,525
Estimated Total Ownership Cost =	**$ 1,005,686,000**	**$ 704,733,880**

		Net Rate =	-3.00%

Alternative 5 - In-Kind Replacement

		Maintenance (allowable annual)		Inspections		Rehabilitation/Replacement		Remaining Service Life (neg.) (end of design life)	
		% Initial Cost		% Initial Cost		% Initial Cost		% Initial Cost	
Year	Life Year	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted
2021	1	$390,000	$ 378,300	$150,000	$ 145,500	$ -	$ -		
2022	2		$ -		$ -	$ 249,363,400	$ 234,626,023		
2023	3		$ -		$ -	$ 249,363,400	$ 227,587,242		
2024	4		$ -		$ -	$ 249,363,400	$ 220,759,625		
2025	5		$ -		$ -	$ 249,363,400	$ 214,136,836		
2026	6		$ -		$ -	$ 249,363,400	$ 207,712,731		
2027	7	$295,000	$ 238,355	$200,000	$ 161,597	$ -	$ -		
2028	8	$295,000	$ 231,204		$ -	$ -	$ -		
2029	9	$295,000	$ 224,268	$200,000	$ 152,046	$ -	$ -		
2030	10	$295,000	$ 217,540		$ -	$ -	$ -		
2031	11	$295,000	$ 211,014	$200,000	$ 143,060	$ -	$ -		
2032	12	$295,000	$ 204,683		$ -	$ -	$ -		
2033	13	$295,000	$ 198,543	$200,000	$ 134,605	$ -	$ -		
2034	14	$295,000	$ 192,587		$ -	$ -	$ -		
2035	15	$295,000	$ 186,809	$0	$ -	$ -	$ -		
2036	16	$295,000	$ 181,205	$200,000	$ 122,851	$ -	$ -		
2037	17	$295,000	$ 175,769		$ -	$ -	$ -		
2038	18	$295,000	$ 170,496	$200,000	$ 115,590	$ -	$ -		
2039	19	$295,000	$ 165,381		$ -	$ -	$ -		
2040	20	$295,000	$ 160,419	$200,000	$ 108,759	$ -	$ -		
2041	21	$295,000	$ 155,607		$ -	$ -	$ -		
2042	22	$295,000	$ 150,939	$200,000	$ 102,331	$ -	$ -		
2043	23	$295,000	$ 146,410		$ -	$ -	$ -		
2044	24	$295,000	$ 142,018	$200,000	$ 96,283	$ -	$ -		
2045	25	$295,000	$ 137,758		$ -	$ -	$ -		
2046	26	$295,000	$ 133,625	$200,000	$ 90,593	$ -	$ -		
2047	27	$295,000	$ 129,616		$ -	$ -	$ -		
2048	28	$295,000	$ 125,728	$200,000	$ 85,239	$ -	$ -		
2049	29	$295,000	$ 121,956		$ -	$ -	$ -		
2050	30	$295,000	$ 118,297	$200,000	$ 80,201	$ -	$ -		
2051	31	$295,000	$ 114,748		$ -	$ 11,544,000	$ 4,490,349		
2052	32	$295,000	$ 111,306	$200,000	$ 75,462	$ -	$ -		
2053	33	$295,000	$ 107,967		$ -	$ -	$ -		
2054	34	$295,000	$ 104,728	$200,000	$ 71,002	$ -	$ -		
2055	35	$295,000	$ 101,586		$ -	$ -	$ -		
2056	36	$295,000	$ 98,538	$200,000	$ 66,806	$ -	$ -		
2057	37	$295,000	$ 95,582		$ -	$ -	$ -		
2058	38	$295,000	$ 92,715	$200,000	$ 62,857	$ -	$ -		
2059	39	$295,000	$ 89,933		$ -	$ -	$ -		
2060	40	$295,000	$ 87,235	$200,000	$ 59,142	$ -	$ -		
2061	41	$295,000	$ 84,618		$ -	$ -	$ -		
2062	42	$295,000	$ 82,080	$200,000	$ 55,647	$ -	$ -		
2063	43	$295,000	$ 79,617		$ -	$ -	$ -		
2064	44	$295,000	$ 77,229	$200,000	$ 52,358	$ -	$ -		
2065	45	$295,000	$ 74,912		$ -	$ -	$ -		
2066	46	$295,000	$ 72,664	$200,000	$ 49,264	$ -	$ -		
2067	47	$295,000	$ 70,484		$ -	$ -	$ -		
2068	48	$295,000	$ 68,370	$200,000	$ 46,353	$ -	$ -		
2069	49	$295,000	$ 66,319		$ -	$ -	$ -		
2070	50	$295,000	$ 64,329	$200,000	$ 43,613	$ -	$ -		
2071	51	$295,000	$ 62,399		$ -	$ -	$ -		
2072	52	$295,000	$ 60,527	$200,000	$ 41,036	$ -	$ -		
2073	53	$295,000	$ 58,712		$ -	$ -	$ -		
2074	54	$295,000	$ 56,950	$200,000	$ 38,610	$ -	$ -		
2075	55	$295,000	$ 55,242		$ -	$ -	$ -		
2076	56	$295,000	$ 53,584	$200,000	$ 36,328	$ 258,863,000	$ 47,020,484		
2077	57	$295,000	$ 51,977		$ -	$ -	$ -		
2078	58	$295,000	$ 50,418	$200,000	$ 34,181	$ -	$ -		
2079	59	$295,000	$ 48,905		$ -	$ -	$ -		
2080	60	$295,000	$ 47,438	$200,000	$ 32,161	$ -	$ -		
2081	61	$295,000	$ 46,015		$ -	$ -	$ -		
2082	62	$295,000	$ 44,634	$200,000	$ 30,261	$ -	$ -		
2083	63	$295,000	$ 43,295		$ -	$ -	$ -		
2084	64	$295,000	$ 41,996	$200,000	$ 28,472	$ -	$ -		
2085	65	$295,000	$ 40,737		$ -	$ -	$ -		
2086	66	$295,000	$ 39,514	$200,000	$ 26,789	$ -	$ -		
2087	67	$295,000	$ 38,329		$ -	$ -	$ -		
2088	68	$295,000	$ 37,179	$200,000	$ 25,206	$ -	$ -		
2089	69	$295,000	$ 36,064		$ -	$ -	$ -		
2090	70	$295,000	$ 34,982	$200,000	$ 23,717	$ -	$ -		
2091	71	$295,000	$ 33,932		$ -	$ -	$ -		
2092	72	$295,000	$ 32,914	$200,000	$ 22,315	$ -	$ -		
2093	73	$295,000	$ 31,927		$ -	$ -	$ -		
2094	74	$295,000	$ 30,969	$200,000	$ 20,996	$ -	$ -		
2095	75	$295,000	$ 30,040		$ -	$ -	$ -		
2096	76	$295,000	$ 29,139	$200,000	$ 19,755	$ -	$ -		
2097	77	$295,000	$ 28,265		$ -	$ -	$ -		
2098	78	$295,000	$ 27,417	$200,000	$ 18,588	$ -	$ -		
2099	79	$295,000	$ 26,594		$ -	$ -	$ -		
2100	80	$295,000	$ 25,796		$ -	$ 11,544,000	$ 1,009,474	$ (15,614,866.67)	$ (1,365,454)
	Sum =	$ 22,220,000	$ 7,489,378	$ 7,350,000	$ 2,519,576	$ 1,528,768,000	$ 1,157,342,765	$ (15,614,867)	$ (1,365,454)

$ 29,570,000

	Unadjusted	Adjusted
Total Costs Maint., Inspect. =	$ 29,570,000	$ 10,008,954
Total Remaining Service Life Cost =	$ (15,614,867)	$ (1,365,454)
Total Costs Initial Investment & Repair/Rehab =	$ 1,528,768,000	$ 1,157,342,765
Estimated Total Ownership Cost =	**$ 1,542,723,133**	**$ 1,165,986,266**

					Net Rate =	-3.00%					
		Alternative 6 - Tunnel									
		Maintenance (allowable annual)		Inspections		Rehabilitation/Replacement				Remaining Service Life (neg.) (end of design life)	
		% Initial Cost		% Initial Cost		% Initial Cost				% Initial Cost	
Year	Life Year	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted			Unadjusted	Adjusted
2021	1		$ -		$ -	$ 271,330,400	$ 263,190,488				
2022	2		$ -		$ -	$ 271,330,400	$ 255,294,773				
2023	3		$ -		$ -	$ 271,330,400	$ 247,635,930				
2024	4		$ -		$ -	$ 271,330,400	$ 240,206,852				
2025	5		$ -		$ -	$ 271,330,400	$ 233,000,647				
2026	6		$ -		$ -	$ 271,330,400	$ 226,010,627				
2027	7		$ -		$ -	$ 271,330,400	$ 219,230,308				
2028	8		$ -		$ -	$ 271,330,400	$ 212,653,399				
2029	9		$ -		$ -	$ 271,330,400	$ 206,273,797				
2030	10		$ -		$ -	$ 271,330,400	$ 200,085,583				
2031	11	$ 1,575,758	$ 1,127,142	$ -	$ -	$ 3,064,225	$ 2,191,844				
2032	12	$ 1,575,758	$ 1,093,327	$ -	$ -	$ 3,064,225	$ 2,126,089				
2033	13	$ 1,575,758	$ 1,060,528	$ -	$ -	$ 3,064,225	$ 2,062,306				
2034	14	$ 1,575,758	$ 1,028,712	$ -	$ -	$ 3,064,225	$ 2,000,437				
2035	15	$ 1,575,758	$ 997,850	$ -	$ -	$ 3,064,225	$ 1,940,424				
2036	16	$ 1,575,758	$ 967,915	$ -	$ -	$ 3,064,225	$ 1,882,211				
2037	17	$ 1,575,758	$ 938,877	$ -	$ -	$ 3,064,225	$ 1,825,745				
2038	18	$ 1,575,758	$ 910,711	$ -	$ -	$ 3,064,225	$ 1,770,973				
2039	19	$ 1,575,758	$ 883,390	$ -	$ -	$ 3,064,225	$ 1,717,843				
2040	20	$ 1,575,758	$ 856,888	$ -	$ -	$ 3,064,225	$ 1,666,308				
2041	21	$ 1,575,758	$ 831,181	$ -	$ -	$ 3,064,225	$ 1,616,319				
2042	22	$ 1,575,758	$ 806,246	$ -	$ -	$ 3,064,225	$ 1,567,829				
2043	23	$ 1,575,758	$ 782,059	$ -	$ -	$ 3,064,225	$ 1,520,794				
2044	24	$ 1,575,758	$ 758,597	$ -	$ -	$ 3,064,225	$ 1,475,171				
2045	25	$ 1,575,758	$ 735,839	$ -	$ -	$ 3,064,225	$ 1,430,915				
2046	26	$ 1,575,758	$ 713,764	$ -	$ -	$ 3,064,225	$ 1,387,988				
2047	27	$ 1,575,758	$ 692,351	$ -	$ -	$ 3,064,225	$ 1,346,348				
2048	28	$ 1,575,758	$ 671,580	$ -	$ -	$ 3,064,225	$ 1,305,958				
2049	29	$ 1,575,758	$ 651,433	$ -	$ -	$ 3,064,225	$ 1,266,779				
2050	30	$ 1,575,758	$ 631,890	$ -	$ -	$ 3,064,225	$ 1,228,776				
2051	31	$ 1,575,758	$ 612,933	$ -	$ -	$ 3,064,225	$ 1,191,912				
2052	32	$ 1,575,758	$ 594,545	$ -	$ -	$ 3,064,225	$ 1,156,155				
2053	33	$ 1,575,758	$ 576,709	$ -	$ -	$ 3,064,225	$ 1,121,470				
2054	34	$ 1,575,758	$ 559,408	$ -	$ -	$ 3,064,225	$ 1,087,826				
2055	35	$ 1,575,758	$ 542,625	$ -	$ -	$ 3,064,225	$ 1,055,192				
2056	36	$ 1,575,758	$ 526,347	$ -	$ -	$ 3,064,225	$ 1,023,536				
2057	37	$ 1,575,758	$ 510,556	$ -	$ -	$ 3,064,225	$ 992,830				
2058	38	$ 1,575,758	$ 495,240	$ -	$ -	$ 3,064,225	$ 963,045				
2059	39	$ 1,575,758	$ 480,382	$ -	$ -	$ 3,064,225	$ 934,153				
2060	40	$ 1,575,758	$ 465,971	$ -	$ -	$ 3,064,225	$ 906,129				
2061	41	$ 1,575,758	$ 451,992	$ -	$ -	$ 3,064,225	$ 878,945				
2062	42	$ 1,575,758	$ 438,432	$ -	$ -	$ 3,064,225	$ 852,577				
2063	43	$ 1,575,758	$ 425,279	$ -	$ -	$ 3,064,225	$ 826,999				
2064	44	$ 1,575,758	$ 412,521	$ -	$ -	$ 3,064,225	$ 802,189				
2065	45	$ 1,575,758	$ 400,145	$ -	$ -	$ 3,064,225	$ 778,124				
2066	46	$ 1,575,758	$ 388,141	$ -	$ -	$ 3,064,225	$ 754,780				
2067	47	$ 1,575,758	$ 376,496	$ -	$ -	$ 3,064,225	$ 732,137				
2068	48	$ 1,575,758	$ 365,202	$ -	$ -	$ 3,064,225	$ 710,172				
2069	49	$ 1,575,758	$ 354,246	$ -	$ -	$ 3,064,225	$ 688,867				
2070	50	$ 1,575,758	$ 343,618	$ -	$ -	$ 3,064,225	$ 668,201				
2071	51	$ 1,575,758	$ 333,310	$ -	$ -	$ 3,064,225	$ 648,155				
2072	52	$ 1,575,758	$ 323,310	$ -	$ -	$ 3,064,225	$ 628,711				
2073	53	$ 1,575,758	$ 313,611	$ -	$ -	$ 3,064,225	$ 609,849				
2074	54	$ 1,575,758	$ 304,203	$ -	$ -	$ 3,064,225	$ 591,554				
2075	55	$ 1,575,758	$ 295,077	$ -	$ -	$ 3,064,225	$ 573,807				
2076	56	$ 1,575,758	$ 286,224	$ -	$ -	$ 3,064,225	$ 556,593				
2077	57	$ 1,575,758	$ 277,638	$ -	$ -	$ 3,064,225	$ 539,895				
2078	58	$ 1,575,758	$ 269,308	$ -	$ -	$ 3,064,225	$ 523,698				
2079	59	$ 1,575,758	$ 261,229	$ -	$ -	$ 3,064,225	$ 507,987				
2080	60	$ 1,575,758	$ 253,392	$ -	$ -	$ 3,064,225	$ 492,748				
2081	61	$ 1,575,758	$ 245,791	$ -	$ -	$ 3,064,225	$ 477,965				
2082	62	$ 1,575,758	$ 238,417	$ -	$ -	$ 3,064,225	$ 463,626				
2083	63	$ 1,575,758	$ 231,264	$ -	$ -	$ 3,064,225	$ 449,718				
2084	64	$ 1,575,758	$ 224,326	$ -	$ -	$ 3,064,225	$ 436,226				
2085	65	$ 1,575,758	$ 217,597	$ -	$ -	$ 3,064,225	$ 423,139				
2086	66	$ 1,575,758	$ 211,069	$ -	$ -	$ 3,064,225	$ 410,445				
2087	67	$ 1,575,758	$ 204,737	$ -	$ -	$ 3,064,225	$ 398,132				
2088	68	$ 1,575,758	$ 198,595	$ -	$ -	$ 3,064,225	$ 386,188				
2089	69	$ 1,575,758	$ 192,637	$ -	$ -	$ 3,064,225	$ 374,602				
2090	70	$ 1,575,758	$ 186,858	$ -	$ -	$ 3,064,225	$ 363,364				
2091	71	$ 1,575,758	$ 181,252	$ -	$ -	$ 3,064,225	$ 352,463				
2092	72	$ 1,575,758	$ 175,814	$ -	$ -	$ 3,064,225	$ 341,889				
2093	73	$ 1,575,758	$ 170,540	$ -	$ -	$ 3,064,225	$ 331,633				
2094	74	$ 1,575,758	$ 165,424	$ -	$ -	$ 3,064,225	$ 321,684				
2095	75	$ 1,575,758	$ 160,461	$ -	$ -	$ 3,064,225	$ 312,033				
2096	76	$ 1,575,758	$ 155,647	$ -	$ -	$ 3,064,225	$ 302,672				
2097	77	$ 1,575,758	$ 150,978	$ -	$ -	$ 3,064,225	$ 293,592				
2098	78	$ 1,575,758	$ 146,448	$ -	$ -	$ 3,064,225	$ 284,784				
2099	79	$ 1,575,758	$ 142,055	$ -	$ -	$ 3,064,225	$ 276,241				
2100	80	$ 1,575,758	$ 137,793	$ -	$ -	$ 3,064,225	$ 267,953	$	(217,064,320.00)	$	(18,981,354)
	Sum =	$ 110,303,030	$ 33,116,070	$ -	$ -	$ 2,927,799,726	$ 2,367,980,051	$	(217,064,320)	$	(18,981,354)

$ 110,303,030

	Unadjusted	Adjusted
Total Costs Maint., Inspect. =	$ 110,303,030	$ 33,116,070
Total Remaining Service Life Cost =	$ (217,064,320)	$ (18,981,354)
Total Costs Initial Investment & Repair/Rehab =	$ 2,927,799,726	$ 2,367,980,051
Estimated Total Ownership Cost =	**$ 2,821,038,436**	**$ 2,382,114,767**

APPENDIX D

RISK ASSESSMENT MATRIX

SUMMARY

This appendix contains a record of the identified risks associated with the various rehabilitation and replacement alternatives. The risk registry is a living document that should be managed throughout the life of the project. To date, the risk registry has been informed by various subject matter experts within WSP, SDOT, and the TAP, and is based on a comprehensive contribution from experts and non-experts alike, with the objective of capturing intrinsic, technical risks, as well as extrinsic risks situated within the broader context of this project.

Risks are never removed from the registry; however, they may be closed out if a management strategy or solution has been identified. The risk registry is structured as follows:

- **Identification:** identifying the possible risks that may have significant impacts on the project.
- **Qualitative Analysis:** determining a risk factor rating based on a qualitative assessment of the probability of the identified risk occurring, and the associated impact of the identified risk. The assessment assigns ratings of very high, high, medium, low, or very low, and answers the following questions:
  - *Probability*: What is the likelihood of the identified risk occurring?
  - *Impact*: What level of impact would the identified risk have on the project if it occurred?
- **Strategy:** developing a strategy, or strategies, for managing the identified risk items. Items could involve primary and secondary plans, and be dependent upon the level of severity. The team discusses and agrees upon strategies. Risk strategy categories include avoiding, transferring, mitigating, and accepting, as defined below:
  - *Avoid*: changes the project plan to eliminate the risk. This would be done by adjusting the scope, schedule, and/or budget.
  - *Transfer*: shifts, but does not eliminate, the risk and responsibility to a specialized third party.
  - *Mitigate*: reduces the probability and/or effect of the risk to an acceptable level.
  - *Accept*: accepts a "do nothing strategy" until/if the risk occurs, at which point it would be dealt with. Contingencies are considered within this strategy.

The register identifies a planned response, or action, to address each risk. Planned responses may change throughout the life of the project as more details become available and the design progresses.

- **Monitor:** developing a plan to monitor and control the risk. Monitoring and controlling risk continues through the life of the project. As the design progresses, details become more evident. This allows the team to reassess and monitor the planned responses, add/re-analyze/edit the identified risks, and evaluate the registry. The registry identifies actions

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taken to-date, planned actions and when the actions are anticipated to occur, and who is responsible for the actions.

- **Monetization:** identifying risks as monetized or not. Monetized risks are assigned a dollar value. Non-monetized risks may be schedule-related and intended to be mitigated as the design progresses; stakeholder coordination-related intended to be mitigated through a detailed coordination plan; etc. Some risks that could be monetized may not be monetized, because they are indirect costs to the project, or covered by assumptions made in the cost estimating for the project. These are noted in the risk registry. Where risks are monetized, the approach to monetizing the risk is included in the registry. All monetized risk values include an absolute cost calculation, and then a factored value, which is the product of the risk factor rating and the absolute cost calculation.

The CBA's risk registry includes more than 40 items, eight of which have been monetized. This list will likely grow in future project phases, once a rehabilitation or replacement decision is made and there is a clear path forward.

West Seattle High Rise Bridge - Risk Management Matrix

ITEM CLOSED

RISK MANAGEMENT MATRIX

Risk Identification			Qualitative Analysis			Risk Response		Monitoring & Controlling	
		(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)
Risk Number	Category	Specific Risks or Opportunity	Probability Table IV (VH, H, M, L, VL)	Impact Table V (a&b) (VH, H, M, L, VL)	Overall Rating Table VI (H, M, L)	Strategy (Avoid, Transfer, Mitigate, Accept)	Planned Response	Assigned Responsible Person	Current Status, Date
1	Schedule	Securing funding to avoid delays in construction	**M**	**H**	M	Accept	Identified as an attribute and are identifying the available funding sources and differences between alternatives.	CITY	CLOSED FOR PURPOSES OF THE CBA. The funding opportunities are included as an attribute and thus will not be monetized. Funding opportunities should be tracked/managed as the project progresses
2	Public/Political	If project stakeholders are not well informed as the project proceeds, then design decisions will be in jeopardy and project delays could occur. Particulary Port of Seattle (Tunnel Option)	**H**	**H**	H	Accept	Meet with the key stakeholders, including TAP and CTF, as part of the CBA process to keep the stakeholders informed and updated on when key decisions or input from them is required.	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed that the City will continue to keep project stakeholders informed, and this will be tracked as a risk for future phases of the project.
3	Schedule	Delays in getting agency approval could lead to overall project delays	**VL**	**H**	M	Accept	Post CBA a detailed design schedule with appropriate agency approval periods should be developed	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed the City will develop a detailed design schedule with the appropriate agency approval periods in later phases of work.
4	Cost	Geotechnical standards for seismic acceleration will likely change in next couple years, which may change design and sizing of components. This is relation to Subduction Zones, Near Fault Effects, and Basin Effects.	**VH**	**M**	H	Accept	Rehabilitation alternatives are treated as a normal structure following FHWA SRM and SDOT BSRPPC document. Replacement alternatives are being considered essential structures following multi-hazard level ground motions and ductility demand requirements in WSDOT. This is assumed for CBA purposes only. City will need to determine project specific criteria for a replacement structure if that is the direction chosen. For Alt #6, this could impact the ovaling and racking deformations of the tunnel	WSP	Parameter Study - Conducted site specific hazard analysis and got S&W input on anticipated spectral acceleration magnification relative to site specific hazard. Assumed not applicable for Alt #2, the foundation will be capacity protected and the ductility demands aren't too high due to the height of the columns, thus, it is not anticipated to have much of an impact (potentially some potential savings within the superstructure - ignored).
5	Schedule	BNSF third quarter moratorium. If required on this project could impact the construction schedule	**M**	**M**	M	Accept	Consider if the moratorium would fall on the critical path. If so, overall construction schedule will consider.	WSP	CLOSED FOR PURPOSES OF THE CBA. Considered in construction durations specified. Should be considered for future phases of work.

Risk Identification			Qualitative Analysis			Risk Response		Monitoring & Controlling	
		(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)
Risk Number	Category	Specific Risks or Opportunity	Probability Table IV (VH, H, M, L, VL)	Impact Table V (a&b) (VH, H, M, L, VL)	Overall Rating Table VI (H, M, L)	Strategy (Avoid, Transfer, Mitigate, Accept)	Planned Response	Assigned Responsible Person	Current Status, Date
6	Schedule	Swing bridge operations interfere with temporary shoring, foundation strengthening, and demoltion methods	H	H	H	Avoid	Avoid shoring and demolition concepts that would impact operations of the swing bridge and navigation channel.	WSP	CLOSED FOR PURPOSES OF THE CBA. Assuming overhead shoring/demolition methods and accounting for in the reported capital costs for Alt #1
7	Stakeholders	Commitments to stakeholders may get lost and not incorporated into the design, which could cause strained relationships and delays in approvals.	M	M	M	Accept	Keep a running log of commitments and agreements, which can be reviewed on a regular basis and ensure they are incorporated into the design process.	CITY	CLOSED FOR PURPOSES OF THE CBA. A log of stakeholder correspondence and commitments should be kept throughout future phases of work.
8	Stakeholders	Service requirements for utilities that will need to be re-routed	M	M	M	Accept	Meet with utilities to discuss	CITY	CLOSED FOR PURPOSES OF THE CBA. Utility coordination efforts should commence in future phases of work. Allowance has been included in the capital cost estimate based on a percentage of the structure cost.
9	Technical	Bridge Stabilization measures don't react as predicted by analytical modeling (uncertainty in expansion pier restraint, uncertainty in post-tensioning effective stress, uncertainty in locked-in forces (i.e. creep effects)), etc.	M	M	M	Accept	Developed a detailed demolition concept in parallel with the CBA	WSP	Conducted analyses; conducted condition testing, installed instrumentation system, correlating measured data to analytical data, will observe how bridge reacts to the stabilization measures before proceeding with further repairs (if deemed as the desirable path forward through the CBA process)

Risk Identification			Qualitative Analysis			Risk Response		Monitoring & Controlling	
		(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)
Risk Number	Category	Specific Risks or Opportunity	Probability Table IV (VH, H, M, L, VL)	Impact Table V (a&b) (VH, H, M, L, VL)	Overall Rating Table VI (H, M, L)	Strategy (Avoid, Transfer, Mitigate, Accept)	Planned Response	Assigned Responsible Person	Current Status, Date
10	Technical	Fixed alignment of BNSF RR tracks relative to existing foundation systems; restricts where an in-kind replacement bridge foundation can be placed on the east side of the west channel.	H	H	H	Accept	Assume a split outriggered foundation system with an elevated drop crossbeam for Alternative #5.	WSP	CLOSED FOR PURPOSES OF THE CBA. Calculated material quantities and included them in the cost estimate based on the planned response. This should be looked at further in future phases of work when actual structure types and layouts are investigated.
11	Technical	Definition of Importance classificatiion of the bridge and the associated ductility demand/strain limits	H	H	H	Mitigate	Due to critical nature of the corridor, the bridge classification will be essential for the CBA.	WSP	Conducting a parameter study to understand material differences between essential and critical classifications. Using ductility demand requirements in WSDOT as the criteria for the CBA. Future work will require that the City define the actual project specific criteria if an essential or critical classification is chosen. Baseline assumption for the CBA analysis is that the replacement bridge is essential. Assumed not applicable for Alt #2, the foundation will be capacity protected and the ductility demands aren't too high due to the height of the columns, thus, it is not anticipated to have much of an impact (potentially some potential savings within the superstructure - ignored).
12	Environmental	FAA height restrictions	H	VH	H	Transfer	For consideration beyond the CBA. Consideration of alternate structure type or configuration to comply with requirements.	CITY	CLOSED FOR PURPOSES OF THE CBA. Identified in the CBA report as a risk with significant cost implications and a key next step for future phases of work should a replacement bridge be evaluated.

Risk Identification			Qualitative Analysis			Risk Response		Monitoring & Controlling	
		(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)
Risk Number	Category	Specific Risks or Opportunity	Probability Table IV (VH, H, M, L, VL)	Impact Table V (a&b) (VH, H, M, L, VL)	Overall Rating Table VI (H, M, L)	Strategy (Avoid, Transfer, Mitigate, Accept)	Planned Response	Assigned Responsible Person	Current Status, Date
13	Environmental	As part of the Bridge Permit, the USCG requires additional vertical clearance over the west Duwamish Waterway - requiring a modified height/profile of the bridge and revisions to approaches and ramps.	**M**	**VH**	H	Transfer	For consideration beyond the CBA. If Sound Transit is accomodated, this will require a complete corridor replacement due to grade limitations.	CITY	Will identify this in the CBA report as a risk with significant cost implications. City has started correspondence with USCG
14	Technical	Fiber optics that serve SDOT's transportation operations division. If bridge is replaced (Alt 4 & 5), need to relocate.	**H**	**M**	M	Accept	City to coordinate	CITY	CLOSED FOR PURPOSES OF THE CBA. For CBA purposes, assumed lump sum value in cost estimate
15	Technical	What happens to swing bridge controls, if bridge is replaced (Alt 4 & 5)	**H**	**M**	M	Accept	City to coordinate	CITY	CLOSED FOR PURPOSES OF THE CBA. For CBA purposes, assumed lump sum value in cost estimate
16	Technical	Technical feasibility existing foundations can accommodate lateral spreading and inertial load effects	**M**	**VH**	H	Accept	WSP to assess existing foundations for lateral spreading loads based on S&W lateral spreading results	WSP	WSP has setup NLS PO models and received preliminary lateral spreading loads from S&W. Initial findings show that the existing foundation systems may not need strengthened.
17	Public/Political	Public/political/media scrutiny of the project reveals or raises issues during design or construction	**M**	**M**	M	Accept	Ensure full transparency and no surprises approach to design and construction	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed the City will continue to engage the public and press throughout the various phases of the project.
18	Stakeholders	Delay in obtaining RR permits could cause schedule issues	**L**	**M**	M	Avoid	Ensure submittals are on time and that negotiations take place early	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed the City will develop a detailed design schedule with the appropriate RR approval periods in later phases of work.
19	Public/Political	Equity-related: lengthy time to restore traffic could maximize impacts on marginalized communities, increasing social inequality	**M**	**M**	M	Avoid	Manage schedule and inform public of all delays. Minimize impacts through MOT and mobility strategies	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed the City will continue to engage the public and address impacts to marginalized communities throughout the various phases of the project.
20	Stakeholders	Early coordination with Sound Transit could provide effeciencies on both sides in terms of the new light rail line to West Seattle (shared costs).	**M**	**M**	M	Accept	Coordinate with Sound Transit early on alternatives. This includes a better understanding of ST criteria (i.e. design life and O&M expenditures, etc), and also an understanding of potential deficiencies with the approach structures	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed that the City will continue to coordinate with Sound Transit, and this will be tracked as a risk for future phases of the project.

Risk Identification			Qualitative Analysis			Risk Response		Monitoring & Controlling	
		(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)
Risk Number	Category	Specific Risks or Opportunity	Probability Table IV (VH, H, M, L, VL)	Impact Table V (a&b) (VH, H, M, L, VL)	Overall Rating Table VI (H, M, L)	Strategy (Avoid, Transfer, Mitigate, Accept)	Planned Response	Assigned Responsible Person	Current Status, Date
21	Public/Political	Communities serving detour routes do not support preferred alternative due to closure time.	**M**	**H**	M	Mitigate	Determine public outreach strategy.	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed the City will continue to engage the public and community regarding impacts throughout the various phases of the project.
22	Environmental	Tribal consultation/coordination extends time frame to issue permits.	**M**	**H**	M	Mitigate	Proactive coordination with Tribes to address issues ahead of permit submittals.	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed the City will develop a detailed design schedule with the appropriate agency/tribal approval periods in later phases of work.
23	Environmental	NEPA/SEPA decisions (such as the EA/EIS for an in-kind replacement bridge) are appealed	**L**	**H**	M	Accept	Maintain a robust public engagement process to proactively address concerns and reduce the possibility of an appeal.	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed the City will continue to engage the public and community regarding impacts throughout the various phases of the project.
24	Environmental	Permit requirements exceed what was expected. For example, requiring nest surveys of the nearby heron rookery or for other nests within the Greenway, that delays the start of construction. Or construction methods or remediation of soils related to Lower Duwamish Superfund cleanup add cost/time.	**L**	**H**	M	Mitigate	Early and continuous coordination with permitting agencies to avoid scope and schedule "surprises" once permits are issued.	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed the City will develop a detailed design schedule with the appropriate agency coordination and approval periods in later phases of work.
25	Environmental	Design changes after permits have been obtained require modifying permits.	M	**M**	M	Mitigate	Bring on the contractor and/or conduct constructability reviews prior to applying for permits.	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed that constructability reviews will be incorporated into future phases of work.
26	Environmental	Design changes during construction require stopping work while permits are modified/resubmitted.	M	**M**	M	Mitigate	Bring on the contractor and conduct constructability reviews prior to applying for permits.	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed that constructability reviews will be incorporated into future phases of work. Consideration of environmental impacts should be made when making decisions if a change will be made during construction
27	Environmental	Tribal and ESA impacts of working in and around wetted perimeteter of the OHWM	**M**	**H**	M	Mitigate	Proactive coordination with Tribes to address issues ahead of permit submittals.	CITY	CLOSED FOR PURPOSES OF THE CBA. It is assumed the City will develop a detailed design schedule with the appropriate agency/tribal approval periods in later phases of work.
28	Technical	Fixed alignment of BNSF RR tracks relative to existing foundation systems; potentially restricting future foundation retrofit . Current Sound Transit prefered alternative has foundation 15' clear from existing battered piles.	**H**	**H**	H	Accept	Accounted for with assumptions made within the CBA and calcuated costs (i.e. independent Sound Transit crossing exists by 2032).	WSP	CLOSED FOR PURPOSES OF THE CBA. Captured cost impacts and carry forward bridge replacement alternatives based on assuming this is a constraint.
29	Schedule	Swing bridge may be stuck in open position(full or partial) causing delay in construciton schedule due to obstruction?	**M**	**L**	M	Accept	This could happen in any of the alternatives, but would be most impactful to work required in and around the OHWM.	CITY	CLOSED FOR PURPOSES OF THE CBA. Could quantity difference between alternatives based on construction duration.

Risk Identification			Qualitative Analysis			Risk Response		Monitoring & Controlling	
		(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)
Risk Number	Category	Specific Risks or Opportunity	Probability Table IV (VH, H, M, L, VL)	Impact Table V (a&b) (VH, H, M, L, VL)	Overall Rating Table VI (H, M, L)	Strategy (Avoid, Transfer, Mitigate, Accept)	Planned Response	Assigned Responsible Person	Current Status, Date
30	Schedule	Earthquake happens during construction causing damage to the bridge	L	H	M	Accept	The probability of the design earthquake occuring during construction is low, however, could have significant impacts if it did occur.	CITY	CLOSED FOR PURPOSES OF THE CBA. Acknowleding the risk, but taking no action. The risk would exist while constructing what is needed to be constructed to mitigate for the EQ concerns, thus, adding more risk.
31	Schedule	Construction efficiency due to concurrent/overlapping Sound Transit construction	M	M	M	Transfer	Track with Risk Item #20	CITY	CLOSED FOR PURPOSES OF THE CBA. City to Track with Risk Item #20 for future phases of work
32	Stakeholders	We see no risk / monetization for impact of construction period on local businesses; these will vary greatly between the different alternatives and should be captured somewhere as a risk unless SDOT believes this is adequately covered by being an attribute	M	H	M	Accept	Local business impacts are being accounted for in the measurable definition of the attribute. The measurables are inclusive of construction duration and thus the attribute could be monetized as part of future phases of work, or for consideration when making repair/replace decisions informed by the CBA.	CITY	CLOSED FOR PURPOSES OF THE CBA. The City will develop monetized social economic impacts as part of future phases of work. This limitation of the CBA should be understood and is listed in the CBA assumptions. Qualitatively speaking, considering the opportunity cost of money, restoring service to the corridor sooner now is more advantageous than assesment of closures in teh future. Thus, the replacement alternatives would be more impactful than Alts 1 and 2
33	Schedule	COVID outbreak causes lockdown. Work stops.	H	M	M	Accept	Covid-19 specific considerations are not being addressed as part of the CBA.	CITY	CLOSED FOR PURPOSES OF THE CBA. The City will decide on how to address Covid-19 impacts as part of future phases of work.
34	Schedule	Impact due to delays in ROW purchase; varies by alternative; will have differing value for each	M	M	M	Accept	Develop a list/exhibit of potentially impacted properties and durations to inform the City's real property group and determine ROM costs of impacts	CITY	CLOSED FOR PURPOSES OF THE CBA. The City to continue coordination with real property group in future phases of work.
35	Schedule	Unexpected soil conditions (see Bertha in SR99 Tunnel) slow project and increases cost (Option 6 most impacted)	L	H	M	Accept	There is a lot of existing subsurface information that already exists. Based on this information, geotechnical design recommendations have been provided.	WSP	CLOSED FOR PURPOSES OF THE CBA. The City will conduct additional explorations as recommended by the design team in future phases of work.
36	Quality	As a "Lack of Resources" or "Quality" category: The TAP recommends monetizing the opportunity of blended options rather than simply focusing on unique, separate options. There can be benefit to blending options (e.g. Options 1 or 2 combined with replacement options) that would open traffic sooner, reduce risk and potentially benefit the situation. We do not believe this has been considered nor monetized.	L	L	L	Accept	The alternative defined to-date already are inclusive of various different work activities; see alternative definitions in the CBA report.	WSP	CLOSED FOR PURPOSES OF THE CBA.
37	Environmental	Requirement that USCG/Port requires greater channel depth, requiring drop in tunnel profile	L	M	M	Accept	Assumed a conservative dredge depth. City to coordinate with USCG/Port to determine appropriate dredge depths	CITY	CLOSED FOR PURPOSES OF THE CBA. The City will coordinate with USCG/Port in future phases of the project.
38	Environmental (HAZ MAT)	Risk of encountering contaminated soils during dredging	M	H	M	Accept	Additional subsurface explorations could mitigate this risk.	WSP	CLOSED FOR PURPOSES OF THE CBA. The City will conduct additional explorations as recommended by the design team in future phases of work.

Risk Identification			Qualitative Analysis			Risk Response		Monitoring & Controlling	
		(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)
Risk Number	Category	Specific Risks or Opportunity	Probability Table IV (VH, H, M, L, VL)	Impact Table V (a&b) (VH, H, M, L, VL)	Overall Rating Table VI (H, M, L)	Strategy (Avoid, Transfer, Mitigate, Accept)	Planned Response	Assigned Responsible Person	Current Status, Date
39	Schedule	An existing casting facility with suitable depth cannot be located and a new facility must be developed	VH	H	H	Accept	Conduct a study - determine potential cross section variants for the ITT, investigate capabilities of existing casting facililties and potential locations for a site specific casting facility, and then establish actual cross section and casting facility needs	CITY	CLOSED FOR PURPOSES OF THE CBA. The City will conduct a casting facility study post CBA.
40	Technical	The site is located in a mapped tsunami inundation zone - the design to-date has not accounted for this. Future work should consider potential inundation heights and account for them in design	VH	H	H	Accept	Set portals to be above potential tsunami inundation heights.	CITY	CLOSED FOR PURPOSES OF THE CBA. The City will determine tsunami inundation heights in future phases of work.
41	Technical	For the tunnel alternative, the civil tie-ins with existing facilities/roadways were not developed. The actual tie-ins will have an impact on the actual alignment and profile used	VH	M	H	Accept	Actual development of the alignment and profile will conduct detailed considerations of tie-ins with existing roadway facilities	CITY	CLOSED FOR PURPOSES OF THE CBA. The City will conduct detailed civil tie-in considerations in future phases of work.
42	Technical	Existing foundations need to be seismically retrofitted based on formal seismic assessment	H	H	H	Accept	Future formal seismic assessment of the existing bridge may yield potential deficiencies.	WSP	CLOSED FOR PURPOSES OF THE CBA. The City will conduct a detailed formal seismic assessment of the exisitng bridge in the future should Alternative #2 be carried forward
43	Technical	Rehabiliation of the existing bridge does not perform as expected and the corridor needs to be closed ahead of the 40-year service life and the bridge replaced.	VL	VH	M	Accept	Recognize a desired O&M cost (above actual) within the Alt 2 estimate. Conduct more refined calculations for Alt 2 than other alternatives to define technical feasibility.	WSP	CLOSED FOR THE PURPOSES OF THE CBA. The City will hold CBA determination contingent on predictable behavior being exhibited by the bridge through completion of the bridge stabilization measures, and a period of time thereafter; install a fiber optic system for asset management purposes; conduct necessary maintenance of the external PT and CFRP systems installed on the bridge, and continue to physically inspect the bridge.
44		Unknown structure type risk - some structure types being considred are fracture critical and require a more intensive and frequent hands-on inspection	M	H	M	Accept	These higher than normal inspection and maintenance costs should be recognized if such features are included in the design	WSP	CLOSED FOR PURPOSES OF THE CBA. The City will continue to track risks specific to the structure type as deisgn develops.
45		Unknown structure type risk - Some alterantives making use of existing foundatoins may utilitize base isolation. Base isolation would include higher initial costs for bearings and higher routine maintenance costs	M	M	M	Accept	These higher initial and maintenance costs should be recognized if such features are included in the design	WSP	CLOSED FOR PURPOSES OF THE CBA. The City will continue to track risks specific to the structure type as deisgn develops.

Risk Identification			Qualitative Analysis			Risk Response		Monitoring & Controlling	
		(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)
Risk Number	Category	Specific Risks or Opportunity	Probability Table IV (VH, H, M, L, VL)	Impact Table V (a&b) (VH, H, M, L, VL)	Overall Rating Table VI (H, M, L)	Strategy (Avoid, Transfer, Mitigate, Accept)	Planned Response	Assigned Responsible Person	Current Status, Date
46		Risk that a rehabilitated structure will not serve it's full expected life as a result of insufficient maintenance due to limited funds	**M**	M	M	Accept	Idealizaed O&M costs have been included in the cost estimate.	WSP	CLOSED FOR PURPOSES OF THE CBA.
47		The superstructure replacement alternative has been assumed to have a 75 year design life, however as the original substructure will be used there is a riks that it will not reach this full design life.	**M**	**H**	M	Accept	Post CBA if a replacement option reusing existing elements is chosen risk cost should be incorporated.	WSP	CLOSED FOR PURPOSES OF THE CBA.

Last Revised: 10/15/20

Table IV

Risk/Opportunity Probability Ranking

Ranking	Probability of Risk Materializing
VH	*80 – 99%*
H	*60 – 79%*
M	*40 – 59%*
L	*20 – 39%*
VL	*1 – 19%*

TABLE V(a)

Evaluating Impact of a Risk on Major Project Objectives

Objective	IMPACT	VL	L	M	H	VH
	Scope	Scope change Barely noticeable	Changes in project features with small effect on schedule, budget or customer satisfaction	Changes in project features with moderate effect on schedule, budget or customer satisfaction	Scope does not meet stakeholders needs	Scope does not meet project purpose and need
	Schedule	Insignificant schedule slippage	Low Schedule slippage	Delay of deliverable or milestone	Delay of critical path deliverable or milestone	project delivery outside fiscal year, funding cycle or construction season or does not meet political commitments
	Budget	Insignificant cost increase	<10% cost increase	10 – 20% cost increase	20 – 40% cost increase	>40% cost increase
	Customer Satisfaction	Minor customer dissatisfaction				Project does not meet customer's needs

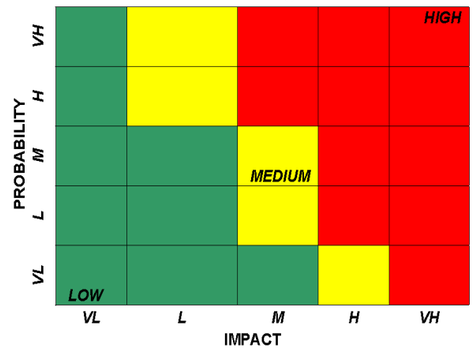
TABLE V(b)

Evaluating Impact of an Opportunity on Major Project Objectives

Objective	IMPACT	VL	L	M	H	VH
	Scope					
	Schedule					
	Budget					
	Customer Satisfaction					

TABLE VI

Overall Rating Matrix

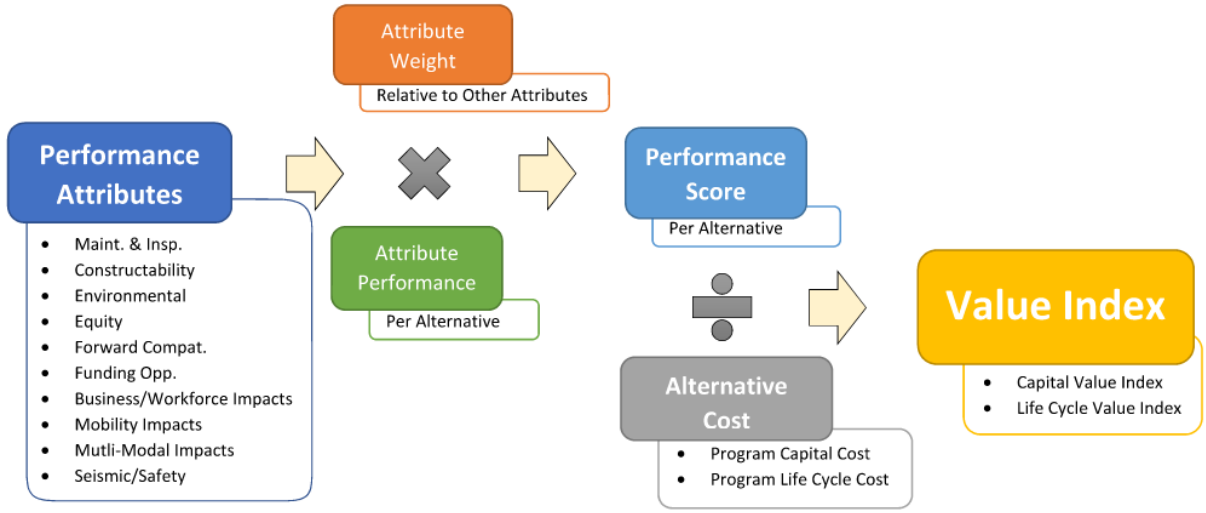


APPENDIX E

EVALUATION MATRIX

SUMMARY

The findings of the CBA are presented as value indices, a measure indicating return on investment, and mathematically represented by dividing the performance score by the cost of a given alternative. The intent of this process is to minimize subjective evaluation of the alternatives, providing an objective and qualitative method of comparison. The process and its inputs are illustrated by the figure below:



This appendix contains the raw input values which were used during the process for developing the alternative performance score and the resulting value indices. Further discussion of the development of alternatives, attributes, weights, etc. can be found in the main body of the Cost-Benefit Analysis Report.

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ATTRIBUTE WEIGHT INPUT

EVALUATION MATRIX

PERFORMANCE CRITERIA MATRIX

West Seattle Bridge Rehabilitation/Replacement Study

Which evaluation criterion contributes more to the overall success of the project?

											TOTAL	%	
Bridge Maintenance, Inspection & Operation	**A**	1/8	2/7	4/5	3/6	2/7	3/6	2/7	2/7	0/9	3.1	5.7%	Bridge Maintenance, Inspection & Operation
Constructability	**B**	5/4	6/3	6/3	8/1	7/2	4/5	5/4	2/7		6.7	12.1%	Constructability
Environmental	**C**	5/4	6/3	6/3	6/3	3/6	4/5	3/6			5.9	10.7%	Environmental
Equity	**D**	5/4	6/3	6/3	5/4	5/4	2/7				5.6	10.1%	Equity
Forward Compatibility	**E**	5/4	5/4	3/6	6/3	2/7					5.1	9.3%	Forward Compatibility
Funding Opportunities	F	6/3	4/5	3/6	2/7						4.7	8.5%	Funding Opportunities
Business and Workplace Impacts	G	2/7	4/5	1/8							4.1	7.5%	Local Business Impacts
Mobility Impacts	H	7/2	2/7								6.4	11.7%	Mobility Impacts
Multi-modal Impacts	I	1/8									5.1	9.3%	Multi-modal Impacts
Seismic/Safety	J										8.3	15.2%	Seismic/Safety
											55.0	100%	

CTF

PERFORMANCE CRITERIA MATRIX

West Seattle Bridge Rehabilitation/Replacement Study

Which evaluation criterion contributes more to the overall success of the project?

SDOT

										TOTAL	%	
Bridge Maintenance, Inspection & Operation	**A**	10 / 10	11 / 9	5 / 15	11 / 9	8 / 12	14 / 6	5 / 15	8 / 12	6 / 14	4.9	8.9%	Bridge Maintenance, Inspection & Operation
Constructability	**B**	9 / 11	7 / 13	11 / 9	9 / 11	13 / 7	7 / 13	7 / 13	6 / 14		5.0	9.0%	Constructability
Environmental	**C**	5 / 15	10 / 10	10 / 10	9 / 11	9 / 11	10 / 10	3 / 17			4.8	8.7%	Environmental
Equity	**D**	15 / 5	13 / 7	14 / 6	10 / 10	11 / 9	5 / 15				6.6	11.9%	Equity
Forward Compatibility	**E**	6 / 14	8 / 12	5 / 15	5 / 15	2 / 18					4.0	7.2%	Forward Compatibility
Funding Opportunities	F	10 / 10	7 / 13	8 / 12	2 / 18						5.1	9.2%	Funding Opportunities
Business and Workplace Impacts	G	2 / 18	8 / 12	2 / 18							4.2	7.6%	Local Business Impacts
Mobility Impacts	H	13 / 7	6 / 14								6.7	12.2%	Mobility Impacts
Multi-modal Impacts	I	4 / 16									5.7	10.4%	Multi-modal Impacts
Seismic/Safety	J										8.2	14.9%	Seismic/Safety
											55.0	100%	

PERFORMANCE CRITERIA MATRIX

West Seattle Bridge Rehabilitation/Replacement Study

Which evaluation criterion contributes more to the overall success of the project?

											TOTAL	%	
Bridge Maintenance, Inspection & Operation	**A**	5/2	6/1	7/0	5/2	4/3	3/4	0/7	3/4	5/2	6.4	11.7%	Bridge Maintenance, Inspection & Operation
Constructability	**B**	4/3	7/0	5/2	3/4	4/3	0/7	5/2	4/3		5.9	10.6%	Constructability
Environmental	**C**	4/3	2/5	2/5	1/6	1/6	1/6	1/6			3.3	6.0%	Environmental
Equity	**D**	1/6	2/5	1/6	0/7	2/5	0/7				2.3	4.2%	Equity
Forward Compatibility	**E**	4/3	2/5	0/7	4/3	3/4					5.0	9.1%	Forward Compatibility
Funding Opportunities	F	3/4	0/7	4/3	5/2						5.6	10.1%	Funding Opportunities
Business and Workplace Impacts	G	1/6	6/1	2/5							6.3	11.4%	Local Business Impacts
Mobility Impacts	H	7/0	6/1								9.6	17.4%	Mobility Impacts
Multi-modal Impacts	I	2/5									4.7	8.6%	Multi-modal Impacts
Seismic/Safety	J										6.0	10.9%	Seismic/Safety
											55.0	100%	

TAP

PERFORMANCE ATTRIBUTE SCORES

EVALUATION MATRIX

WSHB
Attribute Matrix
Prepared by WSP USA

Baseline

VALUE MATRIX

West Seattle Bridge Rehabilitation/Replacement Study

	Evaluation Criteria	Attribute Weight	Alternative	Performance Rating				Total Performance	Accelerated Construction Sensitivity			
				SDOT Avg	TAP	WSP	Average		Custom Weight	Average	Sensitivity Study	Total Performance
A	**Bridge Maintenance, Inspection & Operation**	8.8	Temporary Shoring to Restore Live Load (3 to 5 years)	5.00	1	5	3.67	32.1		8.8	5	43.8
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	43.8			5	43.8
			Accelerated Superstructure Replacement (50 to 75 years)	8.33	7	7	7.44	65.1			7	61.3
			Accelerated Bridge Replacement (75 years)	7.67	7	7	7.22	63.2			7	61.3
			Off-Alignment Immersed Tube Tunnel (75+ years)	3.67	3	3	3.22	28.2			3	26.3
B	**Constructability**	10.6	Temporary Shoring to Restore Live Load (3 to 5 years)	2.33	3	3	2.78	29.4		10.6	3	31.8
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	53.0			5	53.0
			Accelerated Superstructure Replacement (50 to 75 years)	5.67	3	7	5.22	55.3			7	74.1
			Accelerated Bridge Replacement (75 years)	4.33	3	5	4.11	43.5			5	53.0
			Off-Alignment Immersed Tube Tunnel (75+ years)	2.33	1	3	2.11	22.4			3	31.8
C	**Environmental**	8.5	Temporary Shoring to Restore Live Load (3 to 5 years)	3.00	3	5	3.67	31.1		8.5	5	42.3
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	42.3			5	42.3
			Accelerated Superstructure Replacement (50 to 75 years)	4.33	3	7	4.78	40.5			7	59.3
			Accelerated Bridge Replacement (75 years)	3.67	1	7	3.89	32.9			7	59.3
			Off-Alignment Immersed Tube Tunnel (75+ years)	2.00	1	1	1.33	11.3			1	8.5
D	**Equity**	8.7	Temporary Shoring to Restore Live Load (3 to 5 years)	2.33	3	3	2.78	24.2		8.7	3	26.2
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	43.6			5	43.6
			Accelerated Superstructure Replacement (50 to 75 years)	4.33	3	5	4.11	35.9			5	43.6
			Accelerated Bridge Replacement (75 years)	3.67	1	5	3.22	28.1			5	43.6
			Off-Alignment Immersed Tube Tunnel (75+ years)	1.00	1	1	1.00	8.7			1	8.7
E	**Forward Compatibility**	8.5	Temporary Shoring to Restore Live Load (3 to 5 years)	2.33	1	3	2.11	18.0		8.5	5	42.6
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	42.6			5	42.6
			Accelerated Superstructure Replacement (50 to 75 years)	7.00	7	7	7.00	59.7			7	59.7
			Accelerated Bridge Replacement (75 years)	8.33	9	7	8.11	69.1			7	59.7
			Off-Alignment Immersed Tube Tunnel (75+ years)	7.00	7	3	5.67	48.3			3	25.6
F	**Funding Opportunities**	9.3	Temporary Shoring to Restore Live Load (3 to 5 years)	3.67	5	7	5.22	48.4		9.3	7	64.9
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	46.3			5	46.3
			Accelerated Superstructure Replacement (50 to 75 years)	4.33	3	7	4.78	44.3			7	64.9
			Accelerated Bridge Replacement (75 years)	5.00	1	7	4.33	40.2			7	64.9
			Off-Alignment Immersed Tube Tunnel (75+ years)	5.00	1	5	3.67	34.0			5	46.3
G	**Business and Workforce Impacts**	8.8	Temporary Shoring to Restore Live Load (3 to 5 years)	3.67	3	5	3.89	34.4		8.8	5	44.2
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	44.2			5	44.2
			Accelerated Superstructure Replacement (50 to 75 years)	3.67	3	5	3.89	34.4			5	44.2
			Accelerated Bridge Replacement (75 years)	3.00	1	3	2.33	20.6			3	26.5
			Off-Alignment Immersed Tube Tunnel (75+ years)	1.00	1	3	1.67	14.7			3	26.5
H	**Mobility Impacts**	13.8	Temporary Shoring to Restore Live Load (3 to 5 years)	3.00	3	3	3.00	41.3		13.8	3	41.3
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	68.8			5	68.8
			Accelerated Superstructure Replacement (50 to 75 years)	4.33	3	5	4.11	56.6			5	68.8
			Accelerated Bridge Replacement (75 years)	3.67	3	5	3.89	53.5			5	68.8
			Off-Alignment Immersed Tube Tunnel (75+ years)	1.00	3	3	2.33	32.1			3	41.3
I	**Multi-modal Impacts**	9.4	Temporary Shoring to Restore Live Load (3 to 5 years)	4.33	5	3	4.11	38.7		9.4	3	28.2
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	47.0			5	47.0
			Accelerated Superstructure Replacement (50 to 75 years)	8.33	7	5	6.78	63.8			5	47.0
			Accelerated Bridge Replacement (75 years)	5.67	7	5	5.89	55.4			5	47.0
			Off-Alignment Immersed Tube Tunnel (75+ years)	4.00	7	3	4.67	43.9			3	28.2
J	**Seismic/Safety**	13.7	Temporary Shoring to Restore Live Load (3 to 5 years)	3.67	3	7	4.56	62.2		13.7	7	95.6
			Direct Strengthening to Restore Live Load (40 years)	5.00	5	5	5.00	68.3			5	68.3
			Accelerated Superstructure Replacement (50 to 75 years)	7.67	7	9	7.89	107.7			9	122.9
			Accelerated Bridge Replacement (75 years)	8.33	9	9	8.78	119.9			9	122.9
			Off-Alignment Immersed Tube Tunnel (75+ years)	8.00	9	9	8.67	118.4			9	122.9
		100							0.0			

WSHB
Attribute Matrix
Prepared by WSP USA

VALUE MATRIX

West Seattle Bridge Rehabilitation/Replacement Study

Baseline

Evaluation Criteria	Attribute Weight	Alternative	Performance Rating				Total Performance	Accelerated Construction Sensitivity			
			SDOT Avg	TAP	WSP	Average		Custom Weight	Average	Sensitivity Study	Total Performance

OVERALL PERFORMANCE - AVERAGE

		Performance (P)		Program Capital Cost (less long-term maint)		Capital Value Index (P/C)		Program Life Cycle Capital Cost		Life Cycle Value Index (P/C)	
				Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted
1	Temporary Shoring to Restore Live Load (3 to 5 years)	360		$ 125.1	$ 125.1	2.88	2.88	$ 1,558.9	$ 1,049.3	0.23	0.34
2	Direct Strengthening to Restore Live Load (40 years)	500		$ 233.4	$ 233.4	2.14	2.14	$ 916.0	$ 518.0	0.55	0.97
4	Accelerated Superstructure Replacement (50 to 75 years)	563		$ 736.3	$ 736.3	0.76	0.76	$ 1,005.7	$ 704.7	0.56	0.80
5	Accelerated Bridge Replacement (75 years)	527		$ 1,246.8	$ 1,246.8	0.42	0.42	$ 1,542.7	$ 1,166.0	0.34	0.45
6	Off-Alignment Immersed Tube Tunnel (75+ years)	362		$ 2,713.3	$ 2,713.3	0.13	0.13	$ 2,821.0	$ 2,382.1	0.13	0.15
		563.2 Max				2.88 Max		Max		0.56 Max	

OVERALL PERFORMANCE - TAP

		Performance (P)	Program Capital Cost (less long-term maint)		Capital Value Index (P/C)		Program Life Cycle Capital Cost		Life Cycle Value Index (P/C)	
			Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted
1	Temporary Shoring to Restore Live Load (3 to 5 years)	303	$ 125.1	$ 125.1	2.42	2.42	$ 1,558.9	$ 1,049.3	0.19	0.29
2	Direct Strengthening to Restore Live Load (40 years)	500	$ 233.4	$ 233.4	2.14	2.14	$ 916.0	$ 518.0	0.55	0.97
4	Accelerated Superstructure Replacement (50 to 75 years)	461	$ 736.3	$ 736.3	0.63	0.63	$ 1,005.7	$ 704.7	0.46	0.65
5	Accelerated Bridge Replacement (75 years)	435	$ 1,246.8	$ 1,246.8	0.35	0.35	$ 1,542.7	$ 1,166.0	0.28	0.37
6	Off-Alignment Immersed Tube Tunnel (75+ years)	362	$ 2,713.3	$ 2,713.3	0.13	0.13	$ 2,821.0	$ 2,382.1	0.13	0.15
		500.0 Max			2.42 Max				0.55 Max	

OVERALL PERFORMANCE - WSP

		Performance (P)	Program Capital Cost (less long-term maint)		Capital Value Index (P/C)		Program Life Cycle Capital Cost		Life Cycle Value Index (P/C)	
			Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted
1	Temporary Shoring to Restore Live Load (3 to 5 years)	444	$ 125.1	$ 125.1	3.55	3.55	$ 1,558.9	$ 1,049.3	0.28	0.42
2	Direct Strengthening to Restore Live Load (40 years)	500	$ 233.4	$ 233.4	2.14	2.14	$ 916.0	$ 518.0	0.55	0.97
4	Accelerated Superstructure Replacement (50 to 75 years)	646	$ 736.3	$ 736.3	0.88	0.88	$ 1,005.7	$ 704.7	0.64	0.92
5	Accelerated Bridge Replacement (75 years)	607	$ 1,246.8	$ 1,246.8	0.49	0.49	$ 1,542.7	$ 1,166.0	0.39	0.52
6	Off-Alignment Immersed Tube Tunnel (75+ years)	366	$ 2,713.3	$ 2,713	0.13	0.13	$ 2,821.0	$ 2,382.1	0.13	0.15
		645.8 Max			3.55 Max				0.64 Max	

OVERALL PERFORMANCE - SDOT

		Performance (P)	Program Capital Cost (less long-term maint)		Capital Value Index (P/C)		Program Life Cycle Capital Cost		Life Cycle Value Index (P/C)	
			Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted
1	Temporary Shoring to Restore Live Load (3 to 5 years)	333	$ 125.1	$ 125.1	2.66	2.66	$ 1,558.9	$ 1,049.3	0.21	0.32
2	Direct Strengthening to Restore Live Load (40 years)	500	$ 233.4	$ 233.4	2.14	2.14	$ 916.0	$ 518.0	0.55	0.97
4	Accelerated Superstructure Replacement (50 to 75 years)	582	$ 736.3	$ 736.3	0.79	0.79	$ 1,005.7	$ 704.7	0.58	0.83
5	Accelerated Bridge Replacement (75 years)	538	$ 1,246.8	$ 1,246.8	0.43	0.43	$ 1,542.7	$ 1,166.0	0.35	0.46
6	Off-Alignment Immersed Tube Tunnel (75+ years)	358	$ 2,713.3	$ 2,713	0.13	0.13	$ 2,821.0	$ 2,382	0.13	0.15
		582.4 Max			2.66 Max				0.58 Max	

OVERALL PERFORMANCE - SENSITIVITY

		Performance (P)	Program Capital Cost (less long-term maint)		Capital Value Index (P/C)		Program Life Cycle Capital Cost		Life Cycle Value Index (P/C)	
			Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted	Unadjusted	Adjusted
1	Temporary Shoring to Restore Live Load (3 to 5 years)	461	$ 125.1	$ 125.1	3.68	3.68	$ 1,558.9	$ 1,049.3	0.30	0.44
2	Direct Strengthening to Restore Live Load (40 years)	500	$ 233.4	$ 233.4	2.14	2.14	$ 916.0	$ 518.0	0.55	0.97
4	Accelerated Superstructure Replacement (50 to 75 years)	646	$ 736.3	$ 736.3	0.88	0.88	$ 1,005.7	$ 704.7	0.64	0.92
5	Accelerated Bridge Replacement (75 years)	607	$ 1,246.8	$ 1,246.8	0.49	0.49	$ 1,542.7	$ 1,166.0	0.39	0.52
6	Off-Alignment Immersed Tube Tunnel (75+ years)	366	$ 2,713.3	$ 2,713.3	0.13	0.13	$ 2,821.0	$ 2,382.1	0.13	0.15
		645.8 Max			3.68 Max		Max		0.64 Max	

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Attribute Matrix
Prepared by WSP USA

VALUE MATRIX - SDOT Input

West Seattle Bridge Rehabilitation/Replacement Study

	Evaluation Criteria	Attribute Weight	Alternative	Performance Rating				
				Donahue	Izzo	Teo	Average	Max difference
A	**Bridge Maintenance, Inspection & Operation**	8.8	Temporary Shoring to Restore Live Load (3 to 5 years)	5	9	1	5.00	8
			Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
			Accelerated Superstructure Replacement (50 to 75 years)	9	7	9	8.33	2
			Accelerated Bridge Replacement (75 years)	9	7	7	7.67	2
			Off-Alignment Immersed Tube Tunnel (75+ years)	7	3	1	3.67	6
B	**Constructability**	10.6	Temporary Shoring to Restore Live Load (3 to 5 years)	3	1	3	2.33	2
			Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
			Accelerated Superstructure Replacement (50 to 75 years)	5	3	9	5.67	6
			Accelerated Bridge Replacement (75 years)	1	3	9	4.33	8
			Off-Alignment Immersed Tube Tunnel (75+ years)	1	3	3	2.33	2
C	**Environmental**	8.5	Temporary Shoring to Restore Live Load (3 to 5 years)	3	1	5	3.00	4
			Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
			Accelerated Superstructure Replacement (50 to 75 years)	5	3	5	4.33	2
			Accelerated Bridge Replacement (75 years)	1	3	7	3.67	6
			Off-Alignment Immersed Tube Tunnel (75+ years)	3	1		2.00	2

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Attribute Matrix
Prepared by WSP USA

VALUE MATRIX - SDOT Input

West Seattle Bridge Rehabilitation/Replacement Study

	Evaluation Criteria	Attribute Weight	Alternative	Performance Rating				
				Donahue	Izzo	Teo	Average	Max difference
D	**Equity**	8.7	Temporary Shoring to Restore Live Load (3 to 5 years)	5	1	1	2.33	4
			Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
			Accelerated Superstructure Replacement (50 to 75 years)	3	3	7	4.33	4
			Accelerated Bridge Replacement (75 years)	1	3	7	3.67	6
			Off-Alignment Immersed Tube Tunnel (75+ years)	1	1		1.00	0
E	**Forward Compatibility**	8.5	Temporary Shoring to Restore Live Load (3 to 5 years)	5	1	1	2.33	4
			Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
			Accelerated Superstructure Replacement (50 to 75 years)	5	7	9	7.00	4
			Accelerated Bridge Replacement (75 years)	9	7	9	8.33	2
			Off-Alignment Immersed Tube Tunnel (75+ years)	7	7		7.00	0
F	**Funding Opportunities**	9.3	Temporary Shoring to Restore Live Load (3 to 5 years)	5	1	5	3.67	4
			Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
			Accelerated Superstructure Replacement (50 to 75 years)	3	5	5	4.33	2
			Accelerated Bridge Replacement (75 years)	7	3	5	5.00	4
			Off-Alignment Immersed Tube Tunnel (75+ years)	7	3		5.00	4

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Attribute Matrix
Prepared by WSP USA

VALUE MATRIX - SDOT Input

West Seattle Bridge Rehabilitation/Replacement Study

	Evaluation Criteria	Attribute Weight	Alternative	Performance Rating				
				Donahue	Izzo	Teo	Average	Max difference
G	**Local Business Impacts**	8.8	Temporary Shoring to Restore Live Load (3 to 5 years)	5	1	5	3.67	4
			Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
			Accelerated Superstructure Replacement (50 to 75 years)	3	3	5	3.67	2
			Accelerated Bridge Replacement (75 years)	1	3	5	3.00	4
			Off-Alignment Immersed Tube Tunnel (75+ years)	1	1		1.00	0
H	**Mobility Impacts**	13.8	Temporary Shoring to Restore Live Load (3 to 5 years)	5	1	3	3.00	4
			Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
			Accelerated Superstructure Replacement (50 to 75 years)	3	3	7	4.33	4
			Accelerated Bridge Replacement (75 years)	1	3	7	3.67	6
			Off-Alignment Immersed Tube Tunnel (75+ years)	1	1		1.00	0
I	**Multi-modal Impacts**	9.4	Temporary Shoring to Restore Live Load (3 to 5 years)	5	1	7	4.33	6
			Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
			Accelerated Superstructure Replacement (50 to 75 years)	9	7	9	8.33	2
			Accelerated Bridge Replacement (75 years)	1	7	9	5.67	8
			Off-Alignment Immersed Tube Tunnel (75+ years)	1	7		4.00	6

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Attribute Matrix
Prepared by WSP USA

VALUE MATRIX - SDOT Input

West Seattle Bridge Rehabilitation/Replacement Study

Evaluation Criteria	Attribute Weight	Alternative	Performance Rating				
			Donahue	Izzo	Teo	Average	Max difference
Seismic/Safety	13.7	Temporary Shoring to Restore Live Load (3 to 5 years)	5	5	1	3.67	4
		Direct Strengthening to Restore Live Load (40 years)	5	5	5	5.00	0
		Accelerated Superstructure Replacement (50 to 75 years)	9	7	7	7.67	2
		Accelerated Bridge Replacement (75 years)	9	7	9	8.33	2
		Off-Alignment Immersed Tube Tunnel (75+ years)	9	7		8.00	2

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Attribute Matrix
Prepared by WSP USA

VALUE MATRIX - TAP Input

West Seattle Bridge Rehabilitation/Replacement Study

	Evaluation Criteria	Attribute Weight	Alternative	TAP	Total Performance
A	**Bridge Maintenance, Inspection & Operation**	8.8	Temporary Shoring to Restore Live Load (3 to 5 years)	**1**	8.8
			Direct Strengthening to Restore Live Load (40 years)	**5**	43.8
			Accelerated Superstructure Replacement (50 to 75 years)	**7**	61.3
			Accelerated Bridge Replacement (75 years)	**7**	61.3
			Off-Alignment Immersed Tube Tunnel (75+ years)	**3**	26.3
B	**Constructability**	10.6	Temporary Shoring to Restore Live Load (3 to 5 years)	**3**	31.8
			Direct Strengthening to Restore Live Load (40 years)	**5**	53.0
			Accelerated Superstructure Replacement (50 to 75 years)	**3**	31.8
			Accelerated Bridge Replacement (75 years)	**3**	31.8
			Off-Alignment Immersed Tube Tunnel (75+ years)	**1**	10.6
C	**Environmental**	8.5	Temporary Shoring to Restore Live Load (3 to 5 years)	**3**	25.4
			Direct Strengthening to Restore Live Load (40 years)	**5**	42.3
			Accelerated Superstructure Replacement (50 to 75 years)	**3**	25.4
			Accelerated Bridge Replacement (75 years)	**1**	8.5
			Off-Alignment Immersed Tube Tunnel (75+ years)	**1**	8.5

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Attribute Matrix
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VALUE MATRIX - TAP Input

West Seattle Bridge Rehabilitation/Replacement Study

	Evaluation Criteria	Attribute Weight	Alternative	TAP	Total Performance
D	**Equity**	8.7	Temporary Shoring to Restore Live Load (3 to 5 years)	**3**	26.2
			Direct Strengthening to Restore Live Load (40 years)	**5**	43.6
			Accelerated Superstructure Replacement (50 to 75 years)	**3**	26.2
			Accelerated Bridge Replacement (75 years)	**1**	8.7
			Off-Alignment Immersed Tube Tunnel (75+ years)	**1**	8.7
E	**Forward Compatibility**	8.5	Temporary Shoring to Restore Live Load (3 to 5 years)	**1**	8.5
			Direct Strengthening to Restore Live Load (40 years)	**5**	42.6
			Accelerated Superstructure Replacement (50 to 75 years)	**7**	59.7
			Accelerated Bridge Replacement (75 years)	**9**	76.7
			Off-Alignment Immersed Tube Tunnel (75+ years)	**7**	59.7
F	**Funding Opportunities**	9.3	Temporary Shoring to Restore Live Load (3 to 5 years)	**5**	46.3
			Direct Strengthening to Restore Live Load (40 years)	**5**	46.3
			Accelerated Superstructure Replacement (50 to 75 years)	**3**	27.8
			Accelerated Bridge Replacement (75 years)	**1**	9.3
			Off-Alignment Immersed Tube Tunnel (75+ years)	**1**	9.3

WSHB
Attribute Matrix
Prepared by WSP USA

	VALUE MATRIX - TAP Input				
	West Seattle Bridge Rehabilitation/Replacement Study				
	Evaluation Criteria	**Attribute Weight**	**Alternative**	**TAP**	**Total Performance**
G	**Local Business Impacts**	8.8	Temporary Shoring to Restore Live Load (3 to 5 years)	**3**	26.5
			Direct Strengthening to Restore Live Load (40 years)	**5**	44.2
			Accelerated Superstructure Replacement (50 to 75 years)	**3**	26.5
			Accelerated Bridge Replacement (75 years)	**1**	8.8
			Off-Alignment Immersed Tube Tunnel (75+ years)	**1**	8.8
H	**Mobility Impacts**	13.8	Temporary Shoring to Restore Live Load (3 to 5 years)	**3**	41.3
			Direct Strengthening to Restore Live Load (40 years)	**5**	68.8
			Accelerated Superstructure Replacement (50 to 75 years)	**3**	41.3
			Accelerated Bridge Replacement (75 years)	**3**	41.3
			Off-Alignment Immersed Tube Tunnel (75+ years)	**3**	41.3
I	**Multi-modal Impacts**	9.4	Temporary Shoring to Restore Live Load (3 to 5 years)	**5**	47.0
			Direct Strengthening to Restore Live Load (40 years)	**5**	47.0
			Accelerated Superstructure Replacement (50 to 75 years)	**7**	65.9
			Accelerated Bridge Replacement (75 years)	**7**	65.9
			Off-Alignment Immersed Tube Tunnel (75+ years)	**7**	65.9

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Attribute Matrix
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VALUE MATRIX - TAP Input

West Seattle Bridge Rehabilitation/Replacement Study

	Evaluation Criteria	Attribute Weight	Alternative	TAP	Total Performance
J	**Seismic/Safety**	13.7	Temporary Shoring to Restore Live Load (3 to 5 years)	**3**	41.0
			Direct Strengthening to Restore Live Load (40 years)	**5**	68.3
			Accelerated Superstructure Replacement (50 to 75 years)	**7**	95.6
			Accelerated Bridge Replacement (75 years)	**9**	122.9
			Off-Alignment Immersed Tube Tunnel (75+ years)	**9**	122.9

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Attribute Matrix
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VALUE MATRIX - WSP Input

West Seattle Bridge Rehabilitation/Replacement Study

	Evaluation Criteria	Attribute Weight	Alternative	Performance Rating 1	3	5	7	9	Total Performance	WSP
A	**Bridge Maintenance, Inspection & Operation**	8.8	Temporary Shoring to Restore Live Load (3 to 5 years)			5			43.8	5
			Direct Strengthening to Restore Live Load (40 years)			5			43.8	5
			Accelerated Superstructure Replacement (50 to 75 years)				7		61.3	7
			Accelerated Bridge Replacement (75 years)				7		61.3	7
			Off-Alignment Immersed Tube Tunnel (75+ years)		3				26.3	3
B	**Constructability**	10.6	Temporary Shoring to Restore Live Load (3 to 5 years)		3				31.8	3
			Direct Strengthening to Restore Live Load (40 years)			5			53.0	5
			Accelerated Superstructure Replacement (50 to 75 years)				7		74.1	7
			Accelerated Bridge Replacement (75 years)			5			53.0	5
			Off-Alignment Immersed Tube Tunnel (75+ years)		3				31.8	3
C	**Environmental**	8.5	Temporary Shoring to Restore Live Load (3 to 5 years)			5			42.3	5
			Direct Strengthening to Restore Live Load (40 years)			5			42.3	5
			Accelerated Superstructure Replacement (50 to 75 years)				7		59.3	7
			Accelerated Bridge Replacement (75 years)				7		59.3	7
			Off-Alignment Immersed Tube Tunnel (75+ years)	1					8.5	1
D	**Equity**	8.7	Temporary Shoring to Restore Live Load (3 to 5 years)		3				26.2	3
			Direct Strengthening to Restore Live Load (40 years)			5			43.6	5
			Accelerated Superstructure Replacement (50 to 75 years)			5			43.6	5
			Accelerated Bridge Replacement (75 years)			5			43.6	5
			Off-Alignment Immersed Tube Tunnel (75+ years)	1					8.7	1
E	**Forward Compatibility**	8.5	Temporary Shoring to Restore Live Load (3 to 5 years)		3				25.6	3
			Direct Strengthening to Restore Live Load (40 years)			5			42.6	5
			Accelerated Superstructure Replacement (50 to 75 years)				7		59.7	7
			Accelerated Bridge Replacement (75 years)				7		59.7	7
			Off-Alignment Immersed Tube Tunnel (75+ years)		3				25.6	3

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Attribute Matrix
Prepared by WSP USA

VALUE MATRIX - WSP Input

West Seattle Bridge Rehabilitation/Replacement Study

	Evaluation Criteria	Attribute Weight	Alternative	Performance Rating 1		3		5		7		9		Total Performance	WSP
F	**Funding Opportunities**	9.3	Temporary Shoring to Restore Live Load (3 to 5 years)							7				64.9	7
			Direct Strengthening to Restore Live Load (40 years)					5						46.3	5
			Accelerated Superstructure Replacement (50 to 75 years)							7				64.9	7
			Accelerated Bridge Replacement (75 years)							7				64.9	7
			Off-Alignment Immersed Tube Tunnel (75+ years)					5						46.3	5
G	**Local Business Impacts**	8.8	Temporary Shoring to Restore Live Load (3 to 5 years)					5						44.2	5
			Direct Strengthening to Restore Live Load (40 years)					5						44.2	5
			Accelerated Superstructure Replacement (50 to 75 years)					5						44.2	5
			Accelerated Bridge Replacement (75 years)			3								26.5	3
			Off-Alignment Immersed Tube Tunnel (75+ years)			3								26.5	3
H	**Mobility Impacts**	13.8	Temporary Shoring to Restore Live Load (3 to 5 years)			3								41.3	3
			Direct Strengthening to Restore Live Load (40 years)					5						68.8	5
			Accelerated Superstructure Replacement (50 to 75 years)					5						68.8	5
			Accelerated Bridge Replacement (75 years)					5						68.8	5
			Off-Alignment Immersed Tube Tunnel (75+ years)			3								41.3	3
I	**Multi-modal Impacts**	9.4	Temporary Shoring to Restore Live Load (3 to 5 years)			3								28.2	3
			Direct Strengthening to Restore Live Load (40 years)					5						47.0	5
			Accelerated Superstructure Replacement (50 to 75 years)					5						47.0	5
			Accelerated Bridge Replacement (75 years)					5						47.0	5
			Off-Alignment Immersed Tube Tunnel (75+ years)			3								28.2	3
J	**Seismic/Safety**	13.7	Temporary Shoring to Restore Live Load (3 to 5 years)							7				95.6	7
			Direct Strengthening to Restore Live Load (40 years)					5						68.3	5
			Accelerated Superstructure Replacement (50 to 75 years)									9		122.9	9
			Accelerated Bridge Replacement (75 years)									9		122.9	9
			Off-Alignment Immersed Tube Tunnel (75+ years)									9		122.9	9

COMBINED EVALUATION RESULTS

EVALUATION MATRIX

WSHB
Attribute Matrix
Prepared by WSP USA

VALUE INDEX SUMMARY

West Seattle Bridge Rehabilitation/Replacement Study

		SDOT			TAP			WSP			Combined		
		Performance	**Capital Value Index**	**Life Cycle Value Index**	**Performance**	**Capital Value Index**	**Life Cycle Value Index**	**Performance**	**Capital Value Index**	**Life Cycle Value Index**	**Performance**	**Capital Value Index**	**Life Cycle Value Index**
		Unadjusted (0% Net Rate)											
Temporary Shoring to Restore Live Load (3 to 5 years)	Alt 1	333	2.66	0.21	303	2.42	0.19	444	3.55	0.28	360	2.88	0.23
Direct Strengthening to Restore Live Load (40 years)	Alt 2	500	2.14	0.55	500	2.14	0.55	500	2.14	0.55	500	2.14	0.55
Accelerated Superstructure Replacement (50 to 75 years)	Alt 4	582	0.79	0.58	461	0.63	0.46	646	0.88	0.64	563	0.76	0.56
Accelerated Bridge Replacement (75 years)	Alt 5	538	0.43	0.35	435	0.35	0.28	607	0.49	0.39	527	0.42	0.34
Off-Alignment Immersed Tube Tunnel (75+ years)	Alt 6	358	0.13	0.13	362	0.13	0.13	366	0.13	0.13	362	0.13	0.13
		Discounted (-3% Net Rate)											
Temporary Shoring to Restore Live Load (3 to 5 years)	Alt 1	333	2.66	0.32	303	2.42	0.29	444	3.55	0.42	360	2.88	0.34
Direct Strengthening to Restore Live Load (40 years)	Alt 2	500	2.14	0.97	500	2.14	0.97	500	2.14	0.97	500	2.14	0.97
Accelerated Superstructure Replacement (50 to 75 years)	Alt 4	582	0.79	0.83	461	0.63	0.65	646	0.88	0.92	563	0.76	0.80
Accelerated Bridge Replacement (75 years)	Alt 5	538	0.43	0.46	435	0.35	0.37	607	0.49	0.52	527	0.42	0.45
Off-Alignment Immersed Tube Tunnel (75+ years)	Alt 6	358	0.13	0.15	362	0.13	0.15	366	0.13	0.15	362	0.13	0.15

APPENDIX F

CONSTRUCTION SCHEDULES

SUMMARY

This appendix contains the conceptual construction schedule used in the CBA report for the five alternatives starting from 2021. The schedule provides the main work activities (with durations) leading up to the repair or replacement and continues to show work activities during the new service life of the structure.

WSP USA
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33301 Ninth Avenue South
Federal Way, WA 98003-2600
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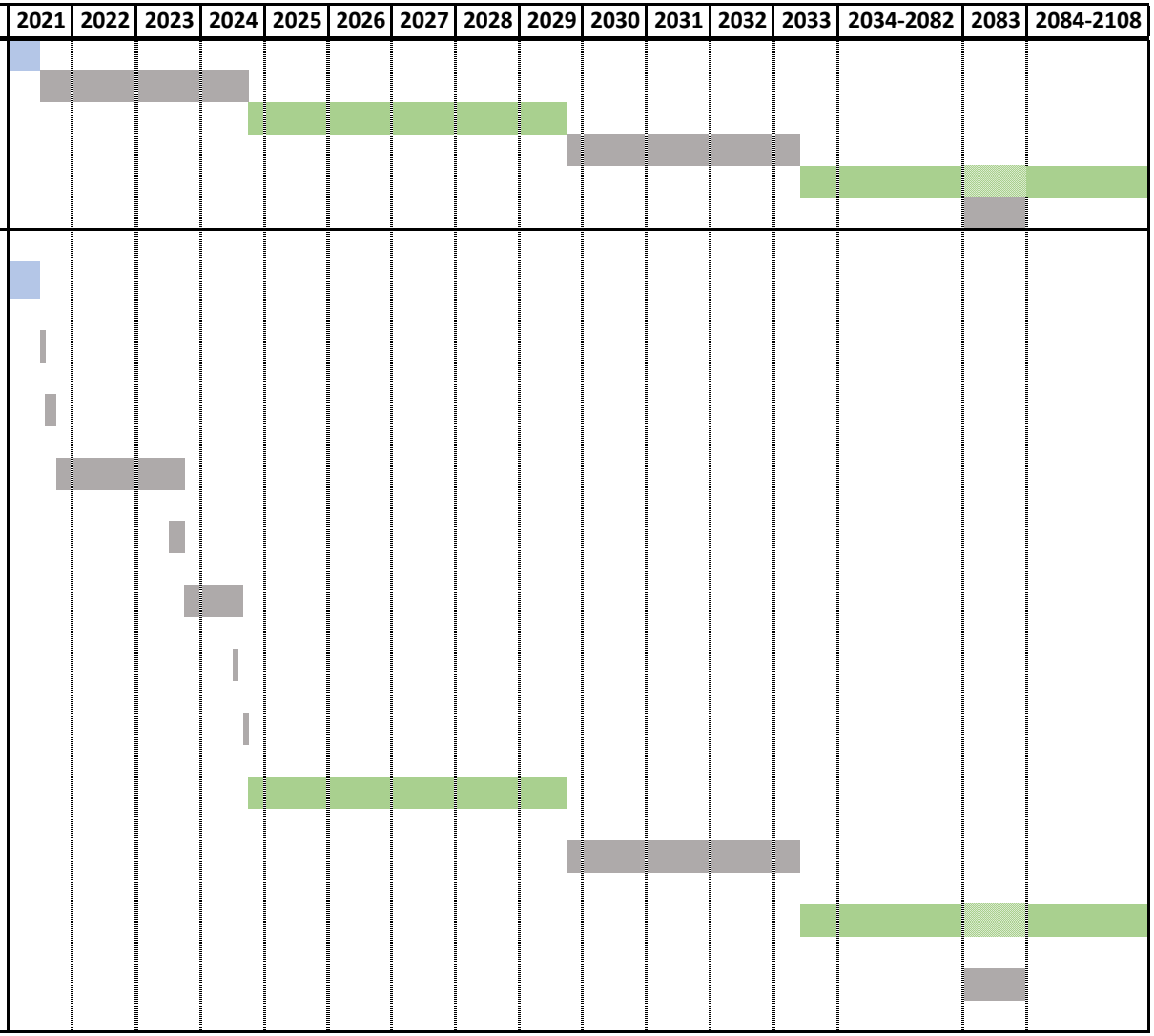
West Seattle Bridge - Alt 1 Shoring to Restore Live Load

Conceptual Schedule

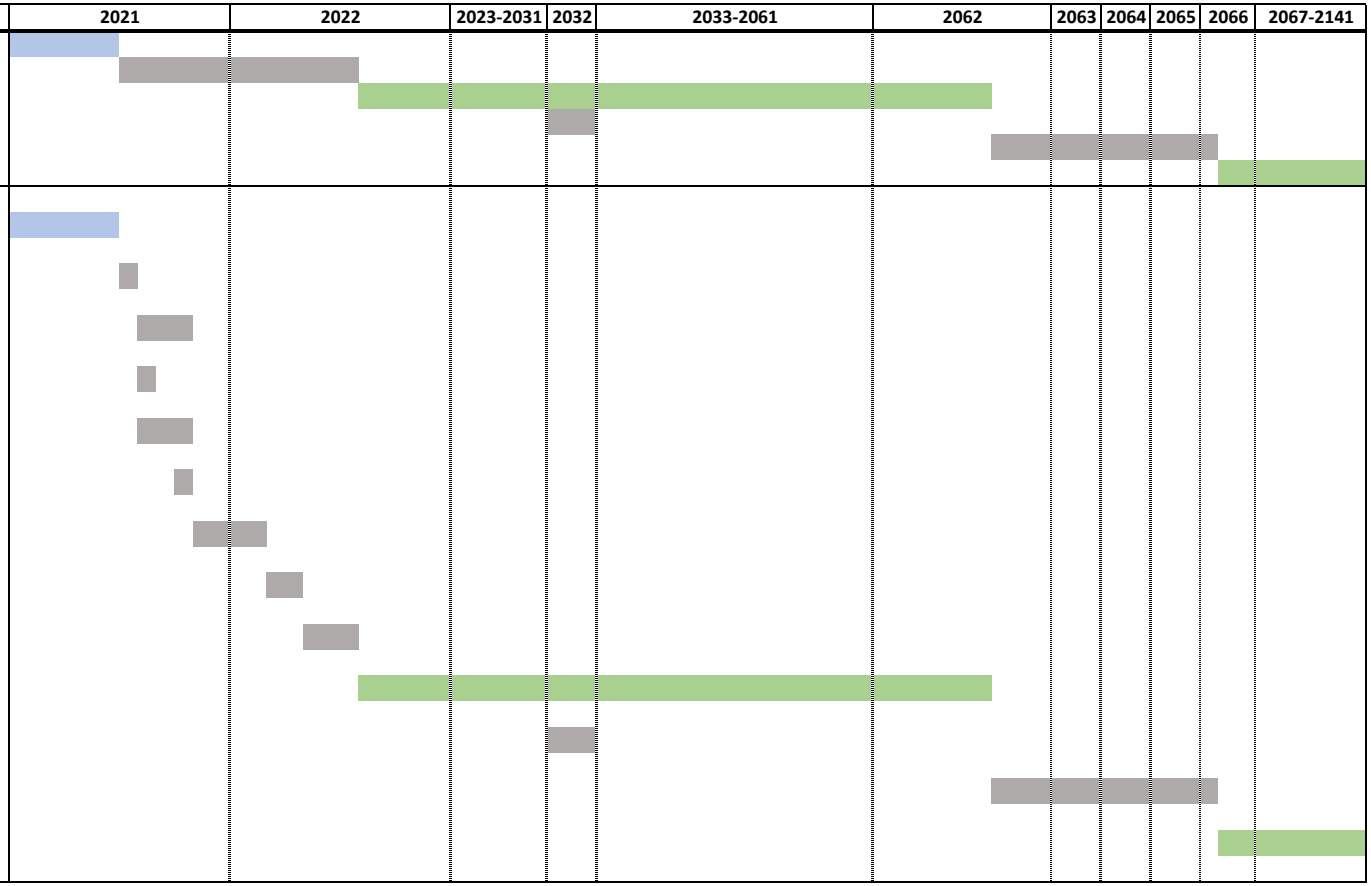
Activity	Duration
Design Shoring	0.5 Years
Construct Shoring	3.25 Years
Traffic on Shored Bridge	5 Years
Construct New Bridge	3.67 Years
New Bridge Service Life	75 Years
Direct Strengthening	1 Year
Design Shoring	6 mo
Mobilization	1 mo
Truss Shops	2 mo
Procurement/Fabrication	24 mo
Shoring - East and West Concurrent	3 mo
Erection	10 mo
Site Civil	4 mo
Project Complete	X
Traffic On Bridge (Service Life)	5 yr
Close Bridge - Construct New Bridge	3.67 yr
Traffic on New Bridge	75 yr
Direct Strengthening	1 yr



West Seattle Bridge - Alt 2 Direct Rehab to Restore Live Load

Conceptual Schedule

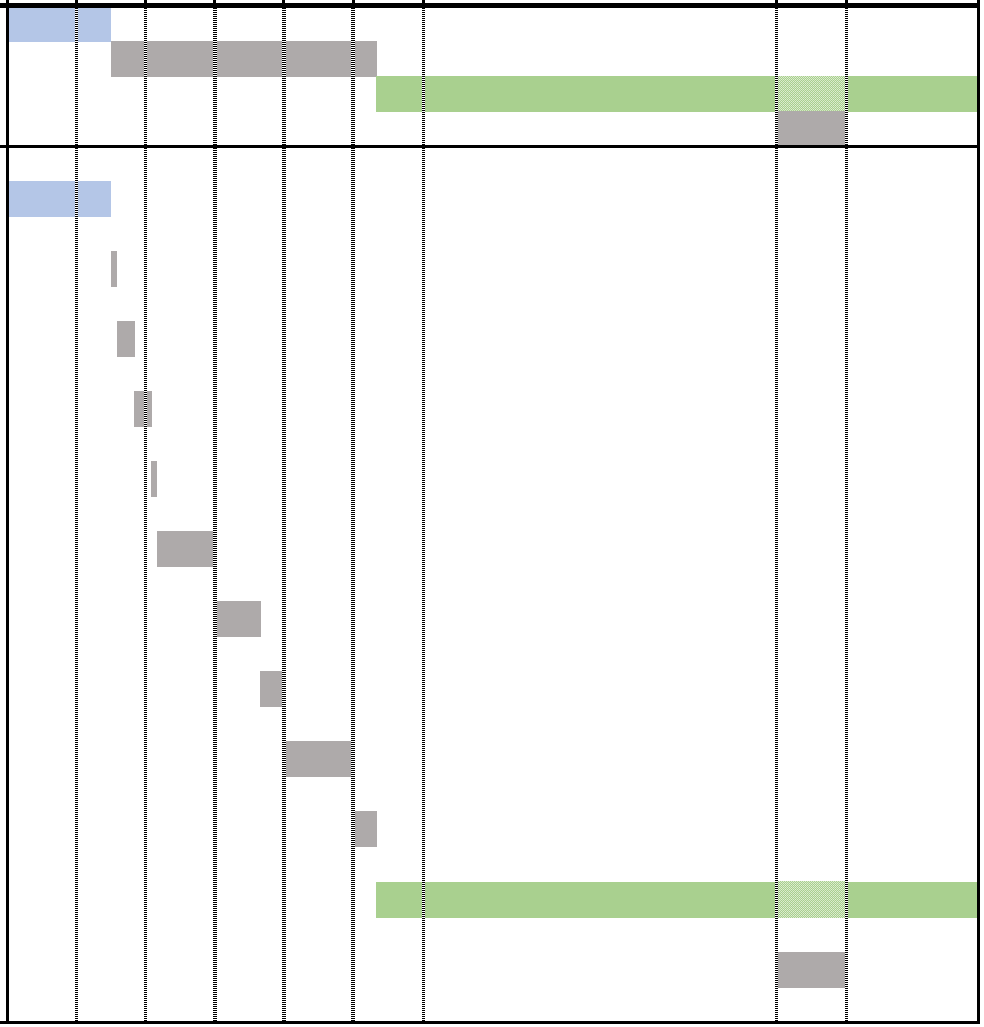
Activity	Duration
Design Rehabilitation	0.5 Years
Construct Rehabilitation	1.08 Years
Traffic on Rehabilitated Bridge	40 Years
Foundation Retrofit (No Closure)	1 year
Construct New Bridge	3.67 Years
New Bridge Service Life	75 Years
Design	6 mo
Mobilization	1 mo
Procure PT Materials	3 mo
Cut Access Holes	1 mo
Phase 1 CFRP Incl Access Platforms	3 mo
Install Anchorge Access Platforms	1 mo
PT Web and Anchorage Construction	4 mo
Install and Tension PT	2 mo
Phase 2 CFRP Incl Access Platforms	3 mo
Traffic On Bridge (Service Life)	40 yr
Foundation Retrofit (No Closure)	1 yr
Close Bridge - Construct New Bridge	3.75 yr
Traffic on New Bridge	75 yr



West Seattle Bridge - Alt 4 Accelerated Superstructure Replacement (Concrete)

Conceptual Schedule

Activity	Duration
Design Superstructure Replacement	1.5 Years
Construct Superstructure Replacement	3.83 Years
New Bridge Service Life	75 Years
Direct Strengthening	1 Year
Design	18 mo
Mobilization	1 mo
Railroad review of Bridge Demo Plan	3 mo
Install Demo Shoring all spans	3 mo
Detention Crack Arrest PT	1 mo
Bridge Demo Incl. cross heads	10 mo
Foundation Retrofit - Pier 16/17	8 mo
Stage 1 Superstructure - Concurrent	4 mo
Stage 2 Superstructure	12 mo
Utilities/Lighting/Barrier/Civil	4 mo
Open New Bridge	
Direct Strengthening	1 yr



West Seattle Bridge - Alt 5 Accelerated Bridge Replacement

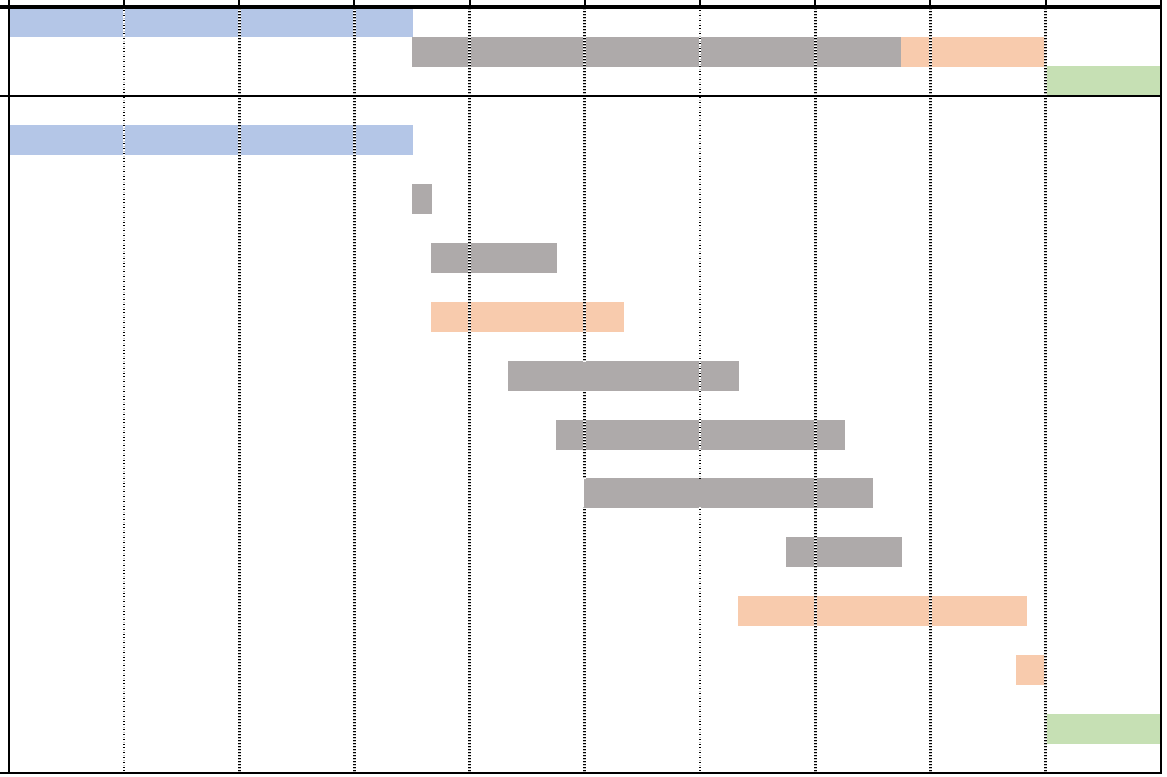
Conceptual Schedule

Activity	Duration	2021	2022	2023	2024	2025	2026	2027-2075	2076	2077-2101
Design Replacement Bridge	1.5 Years									
Construct Replacement Bridge	3.67 Years									
New Bridge Service Life	75 Years									
Direct Strengthening	1 Year									
Design	18 mo									
Mobilization	1 mo									
Railroad review of Bridge Demo Plan	3 mo									
Install Demo Shoring all spans	3 mo									
Detention Crack Arrest PT	1 mo									
Bridge Demo Incl. P13 - 18	12 mo									
Substructure - Concurrent	6 mo									
Superstructure	12 mo									
Utilities/Lighting/Barrier/Civil/Sealer	6 mo									
Open New Bridge										
Direct Strengthening	1 yr									

West Seattle Bridge - Alt 6 Immersed Tunnel

Conceptual Schedule

Activity	Duration
Design and ROW/Easements	3.5 Years
Construct Tunnel and Tie-Ins	5.5 Years
New Tunnel Service Life	75+ Years
Design and ROW/Easements	42 mo
Mobilization	2 mo
Develop Casting Facility	12 mo
Ancillary Construction Activities	18 mo
Excavation Support and Excavations	24 mo
Cast, Deliver, Install Immersed Segments	30 mo
Cut-and-Cover/Open Cut Segments	30 mo
Tunnel Interiors	12 mo
Infrastructure Tie-Ins & Existing Bridge Demo	30 mo
Final Utility Connections	3 mo
New Tunnel Opening/Service Life	75+ yr



APPENDIX G

TEMPORARY SHORING AND DEMOLITION CONCEPTS MEMORANDUM

WEST SEATTLE HIGH-RISE BRIDGE
SHORING TO RESTORE TRAFFIC & DEMOLITION CONCEPTS

This appendix is intended to describe shoring to temporarily restore live load on the West Seattle High-Rise Bridge (WSHB) as well as demolition concepts. The shoring will support some or all of the superstructure dead load and the restored live load as well as the shoring weight. After the time period for live load traffic restoration is over, the shoring can be used for the controlled demolition of the existing superstructure if necessary.

Main Span Shoring Concepts

The Spokane Street Swing Bridge is located approximately 40 feet north of WSHB. The two moving segments of the bridge rotate under WSHB as depicted in the plan view in Figure 1. The shoring system must consider the required operation of the swing bridge. Approximate shoring frame lines under WSHB are also shown in the figure. The envelopes of the swing bridge movement along each of these frame lines are depicted in Figure 2. The tops of the envelopes are at an approximate elevation of 67.86 feet. This elevation is for the top of the swing bridge rail based on the as-built plans. An elevation of 73 feet is recommended for shoring design.

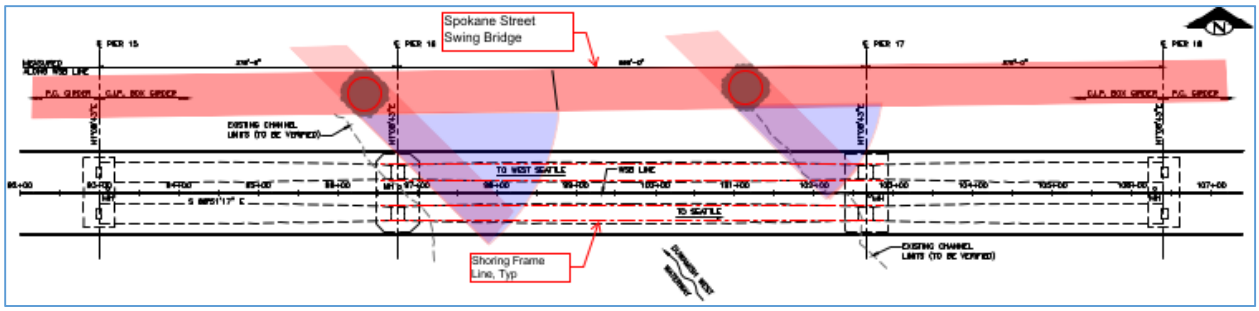


Figure 1: Envelope of Spokane Street Swing Bridge Under WSHB (Plan View)

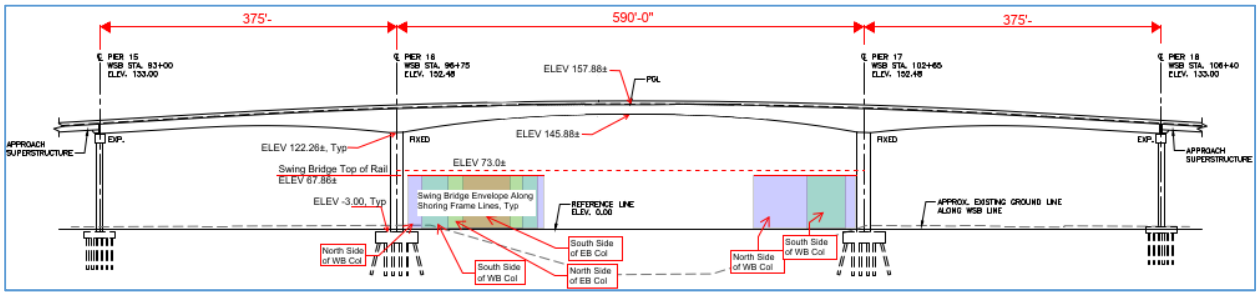


Figure 2: Envelope of Spokane Street Swing Bridge (Elevation)

WSP USA
Suite 300
33301 Ninth Avenue South
Federal Way, WA 98003-2600
wsp.com

Option 1: Fully Operational Swing Bridge

Shoring will need to span the entire channel for the swing bridge to remain fully operational. A truss is recommended as depicted in Figure 3. The truss will be approximately 59 feet deep and can extend above the bottom of the existing superstructure as shown. The top of shoring should be kept approximately 1.5 feet below the overhang so support is provided from the bottom of the superstructure. The bottom of truss elevation will be at approximately 93 feet based on the City of Seattle datum used in the as-built plans. This elevation is 47 feet lower than the top of the permanent clearance envelope and 93 feet above the mean high water elevation based on the as-built plans. Approval from the Coast Guard and other parties with jurisdiction over the channel will be required.

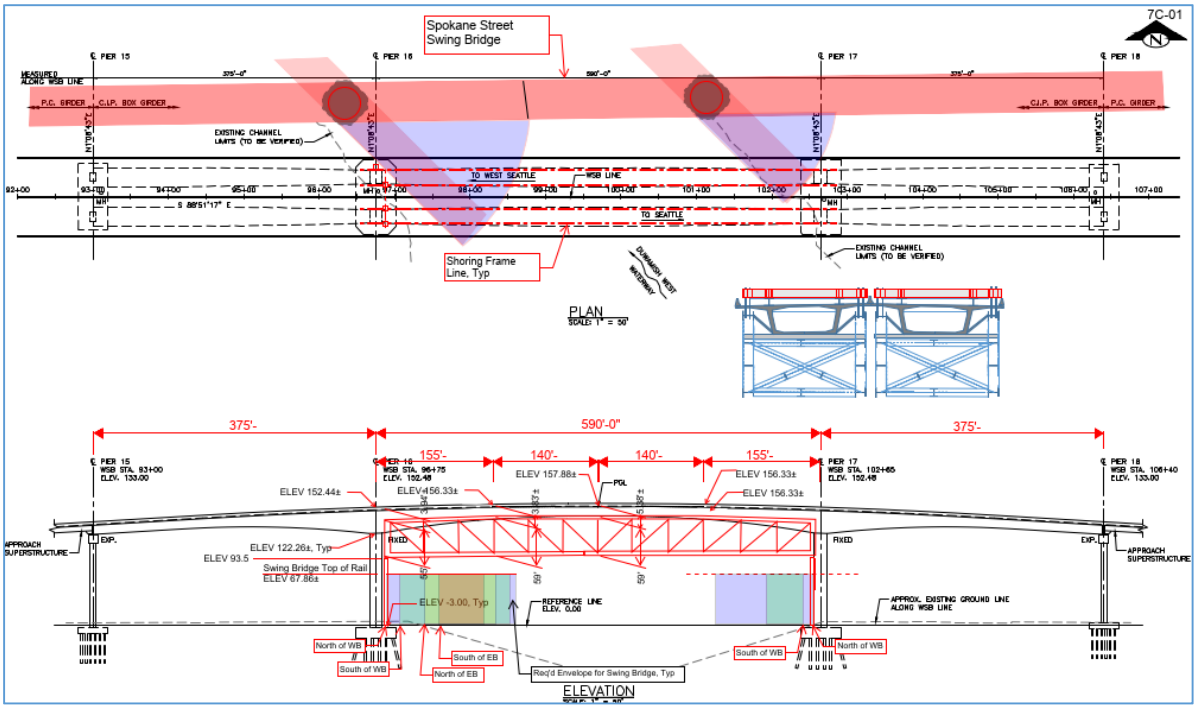


Figure 3: Main-Span Shoring for Full Operation of Swing Bridge

Option 2: Partially Operational Swing Bridge

If the swing bridge operation is limited to the western portion, the WSHB shoring can be supported in the eastern half as depicted in Figure 4. The support will allow the truss depth to be reduced by approximately 16 feet. The bottom of truss elevation could then be lifted to 109 feet. This elevation is 31 feet lower than the top of the permanent clearance envelope and 111 feet above the mean high water elevation. Approval from the Coast Guard and other parties with jurisdiction over the channel will be required.

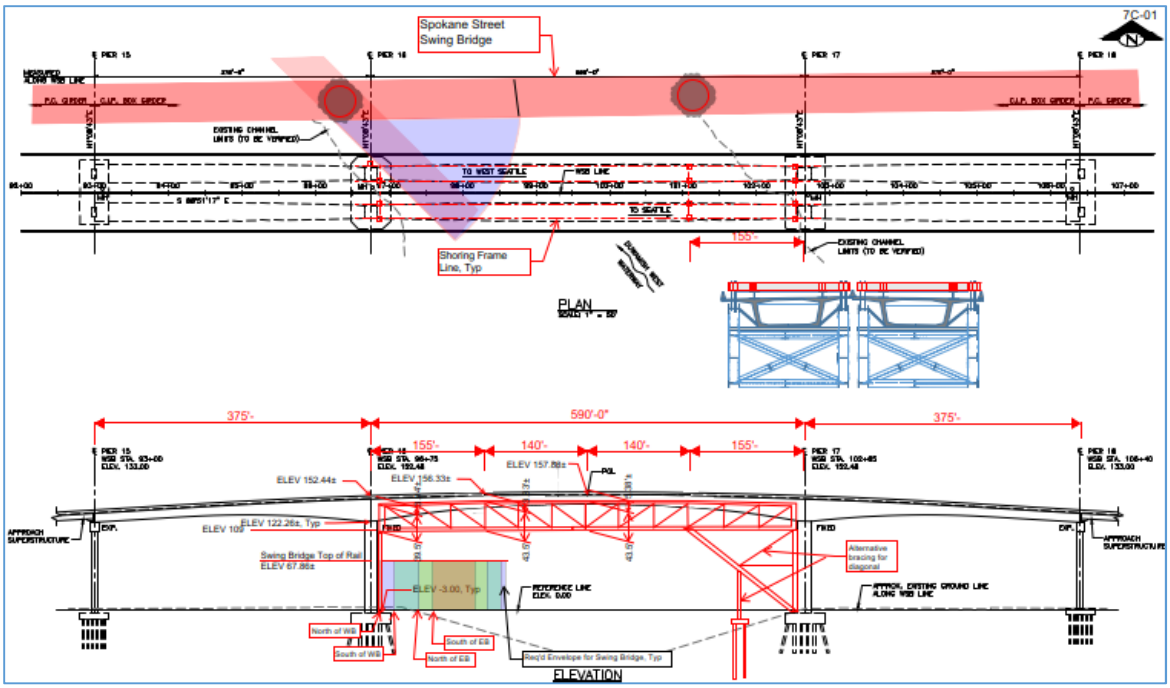


Figure 4: Main-Span Shoring for Partial Operation of Swing Bridge

Option 3: Non-Operation of Swing Bridge

For the highly unlikely case of non-operation of the swing bridge, a K-Frame shoring system under the main span is proposed as shown in Figure 5. It is proposed that the main longitudinal shoring members be shored near the third points. The resulting spans for the shoring members are approximately 190 feet. Plate girders are recommended for these members. The required depth of the plate girders is 10 feet.

It is recommended that the longitudinal shoring elements be placed below the superstructure since channel clearance is controlled by the swing bridge. Placing the girders fully below the superstructure soffit allows the girders to brace each other and to support the superstructure from the top of the girders instead of at a varying intermediate height.

The existing main-span pier caps will be utilized as a foundation for the shoring. Diagonal elements will extend from the existing pile caps to the plate girder support points, see Figure 5.

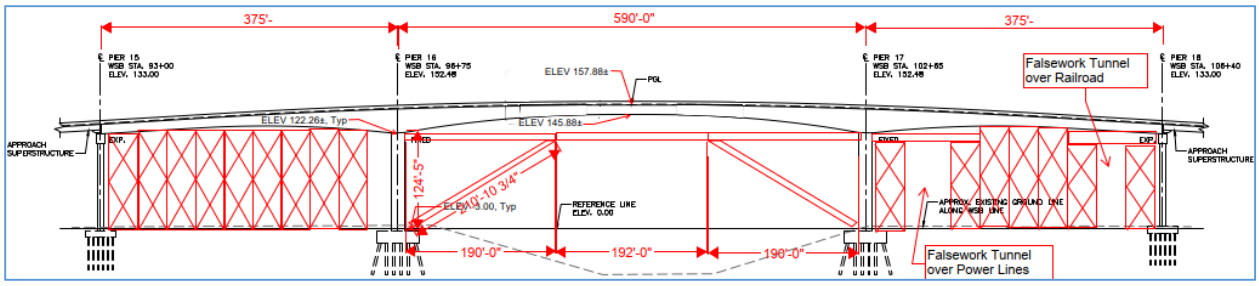


Figure 5: Main Span Shoring Assuming Non Operational Low Bridge

Allowing for 2.5 feet for support beams over the longitudinal girders and an additional 1.5' for blocking, the bottom of the truss elevation will at approximately 108 feet based on the City of Seattle datum used in the as-built plans. This elevation is 32 feet lower than the top of the permanent clearance envelope and 110 feet above the mean high water elevation based on the as-built plans. Approval from the Coast Guard and other parties with jurisdiction over the channel will be required.

Option 4: No Intrusion of Navigable Channel

The previous shoring options support the bridge from underneath with varying levels of intrusion into the Duwamish Waterway clearance envelope, some by narrowing the usable width due to limitations on opening operations of the WSLB, and all by reducing the allowable vertical clearance. Potential impacts on Port of Seattle and upstream industry operations were deemed unacceptable, therefore an above bridge shoring option was considered the most practical. This shoring option is referred to as "Alternative 1" in this report.

Error! Reference source not found. below depicts the temporary shoring scheme. Trusses are cantilevered over the main piers to support the compromised segments in the main span. Each side of the bridge would have a series of four trusses, one over each box girder web. Because these trusses and the hardware to support the existing structure take up some of the roadway width, only between three and five lanes would be available for live load. This alternative would not be future compatible with light rail.

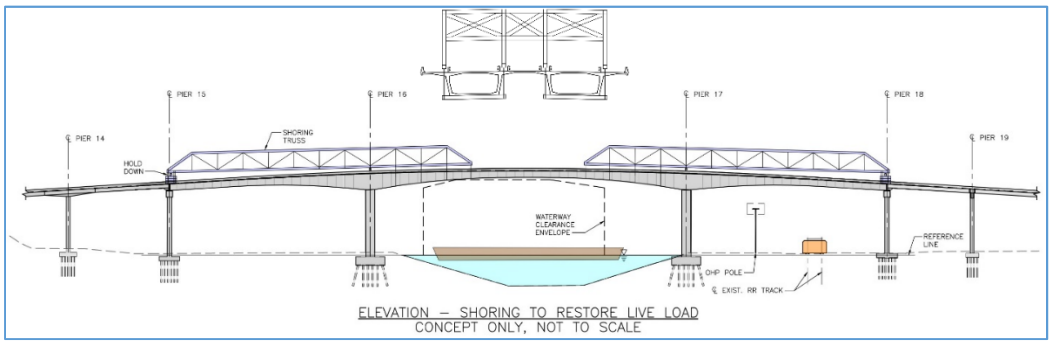


Figure 6: Above Bridge Temporary Shoring Scheme

Approach Span Shoring Concept

Shoring of the approach spans (aka side spans or end spans) can be provided from the ground as depicted in Figure 5. This shoring will need to be carefully designed considering the height of approximately 120 feet and very poor soil conditions. The area under the west approach span is relatively free of obstructions. However, the east span crosses overhead power lines, two railroad tracks and local streets. The streets provide the western access to several businesses, see Figure 7. Coordination with the impacted agencies will be required.

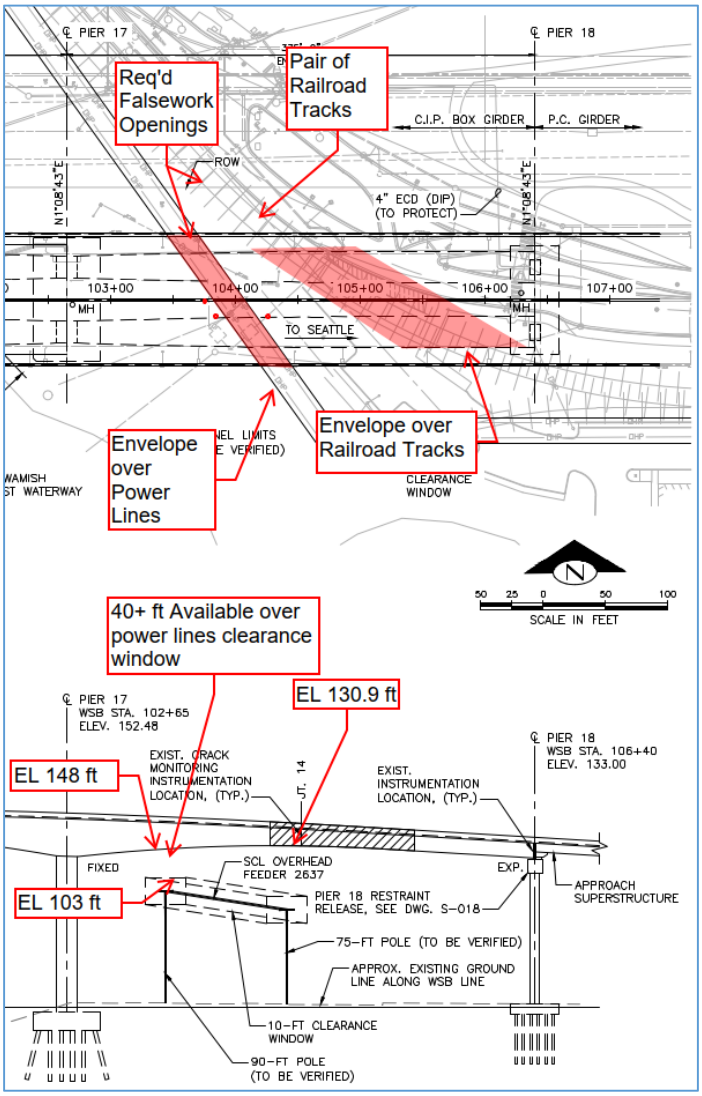


Figure 7: Partial Site Base Map for East Approach Span

DEMOLITION SEQUENCE CONCEPTS

Common Concerns Regarding Demolition

With varying timelines, all alternatives presented in this cost-benefit analysis report eventually consider replacement of the main span. Post tensioned segmental box girder bridges are still a relatively new bridge type and as such documented cases of controlled demolition are few, particularly at the height of the WSHB. Site constraints, internal structural composition and dimensional properties of the segments represent unknowns that increase the complexity involved in the deconstruction sequence and should be studied in greater detail in the future.

Demolition Concept for Alternative 1

The above bridge shoring concept is meant to serve as an interim solution to return the live load to the bridge while a permanent replacement structure is planned and designed. After 5 years, demolition of the shored bridge is expected to begin. At this point, the shoring trusses would be used to assist in the disassembly of the box girder, as show in Figure 8.

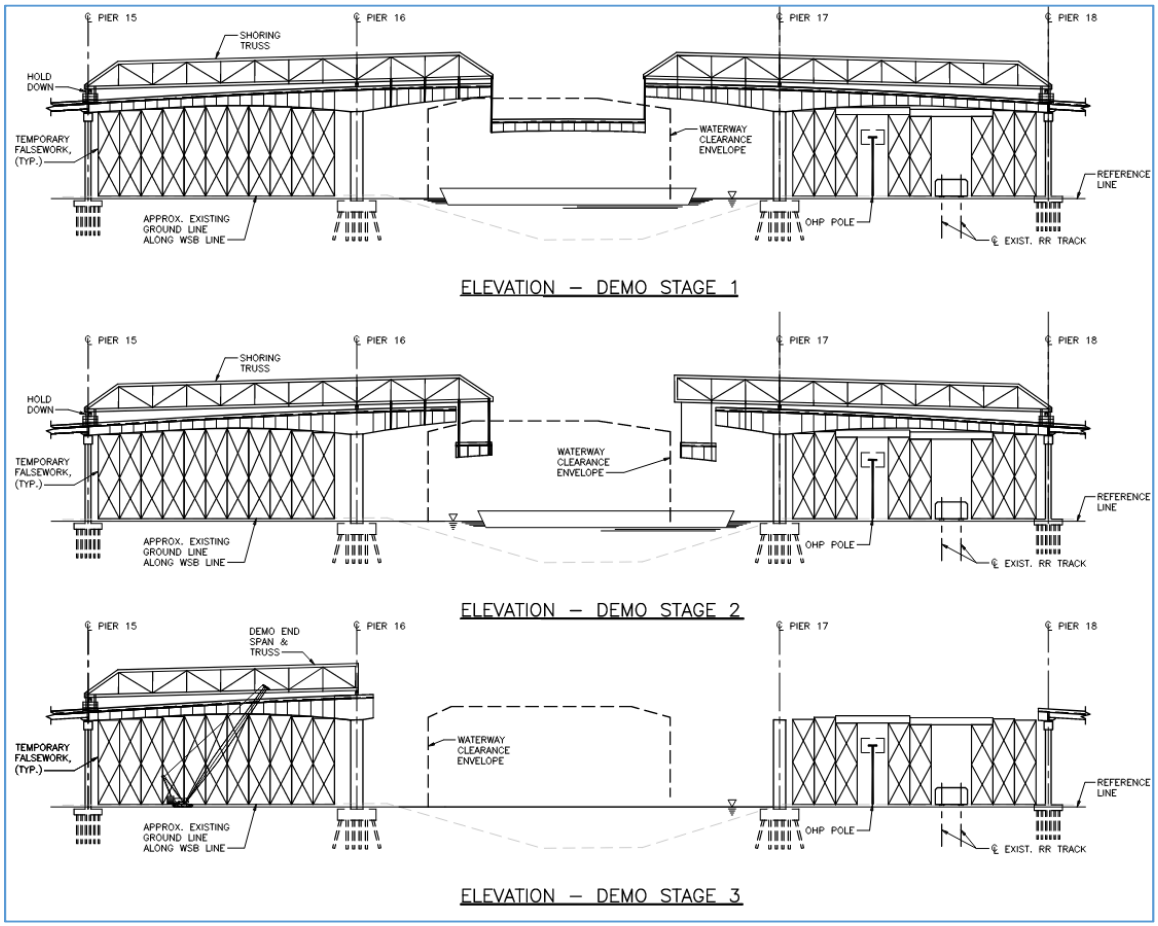
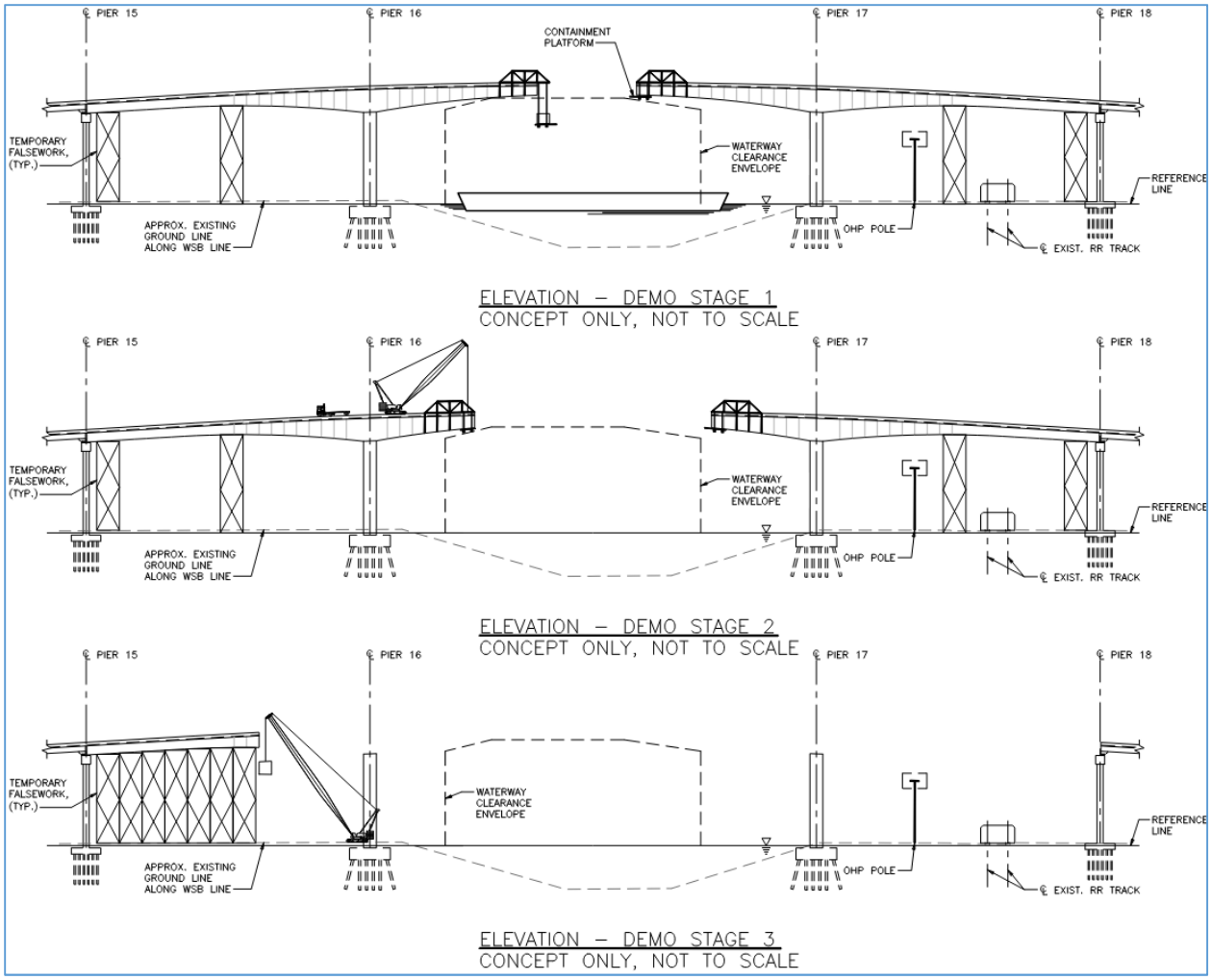


Figure 8: Above Bridge Shoring Concept During Demolition

Demolition would be carried out in stages. The unsupported portion of the midspan between the ends of the cantilevers would be separated and lowered, with the aid of the shoring, as a monolithic assembly during stage 1. Stage 2 would see sequential removal of the box girder segments, working back towards the piers, followed by the disassembly of the shoring shortly after. During stage 3 what was the back span portion of the shoring truss would be used to stabilize and lower the remaining segments of the box girder of the approach span.

Demolition Concept for Alternatives 2 through 6

Alternatives 2 through 6 will require eventual demolition and replacement without the assistance of temporary shoring. For these alternatives, a "conventional" deconstruction sequence is envisioned.



APPENDIX H

PRELIMINARY STRUCTURAL REHABILITATION MEMORANDUM

WSHB Preliminary Work-In-Progress Phase II Calculations (Oct. 5, 2020)

This document presents results from an evaluation of the preliminary work-in-progress Phase II external tendon strengthening layout for the West Seattle High-Rise Bridge (WSHB) in support of the Cost-Benefit Analysis (CBA). The calculations were originally provided to support WSP's response to the TAP's memorandum titled "Alternate PT Profile Recommendation" dated August 2020. The calculations build upon analysis methods, assumptions, and results discussed in the Temporary Crack Arrest – Calculation Package dated July 02, 2020. An attempt was made to resolve all open comments by including the noted changes in the preliminary Phase II evaluation.

Highlights are used in this document to draw attention to items requiring additional evaluation or exceedances relative to initial conservative assumptions regarding the effect of softening at the distressed joints on the overall behavior of the structure. The Phase I and Phase II calculations considered demands from both a typical Uncracked Model segmental construction analysis and a more conservative Hinged Model analysis that included the effects of softening at the distressed Joints 38 on redistribution of forces in the existing structure that are locked in when stressing the retrofit PT. Calculations discussed herein use the conservative effective hinged section geometry at Joints 38.

More recently, the structural health monitoring instrumentation was used to evaluate model assumptions for stiffness at the distressed Joints 14 and 38. Predicted bridge deformations under thermal variations and discrete loading from the platform hoist operations were compared with measured data. In summary, the analysis indicated additional softening at Joints 14 and 38 had occurred relative to analytical predictions using uncracked behavior. Analysis of longitudinal floor gauge instrumentation at Joints 14 and 38 and vertical displacements along the bridge between Piers 16 and 17 indicated that use of the effective hinged section geometry was unnecessarily conservative. Instead, use of section property modifiers at Joints 14 and 38 were found to improve the accuracy of predictions for floor gauge displacements under thermal loading, with additional softening at Joints 38 consistent with visual observations for the extent of cracking.

The reduced effects of redistribution are expected to allow for optimization of the Phase II external post-tensioning layout relative to the preliminary conservative assessment discussed herein. The reduced effects of redistribution would be expected to decrease the magnitudes of positive moments and soffit tension that govern at Joints 38 and 40, allowing for optimization of the external tendons that in turn decreases compressive stresses at Joints 40 that had approached applicable limits in the preliminary evaluation. The side span results had margins to include any redistribution effects associated with the use of cracked section property modifiers at Joints 14.

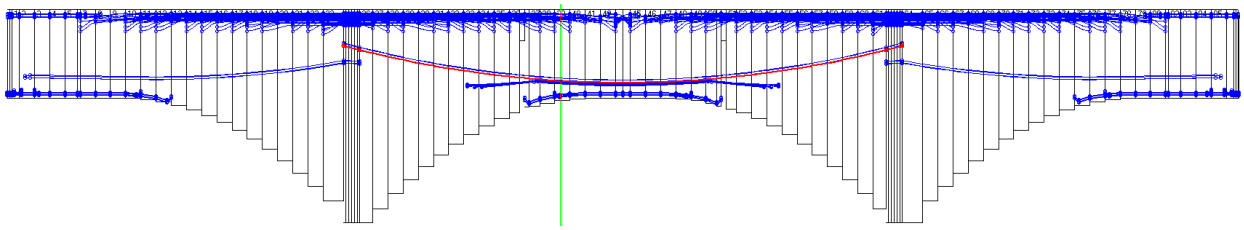
Continued evaluation of the structural health monitoring instrumentation both leading up to, during, and following stressing of the Phase I external post-tensioning will help to confirm assumptions regarding bridge behavior and stiffness at the distressed Joints 14 and 38. Potential future Phase II calculations will include the effects of softening consistent with the measured behavior. Again, this offers a refinement relative to the conservative effective hinged model section approach included in these preliminary calculations.

Preliminary Work-In-Progress Phase II PT Layout

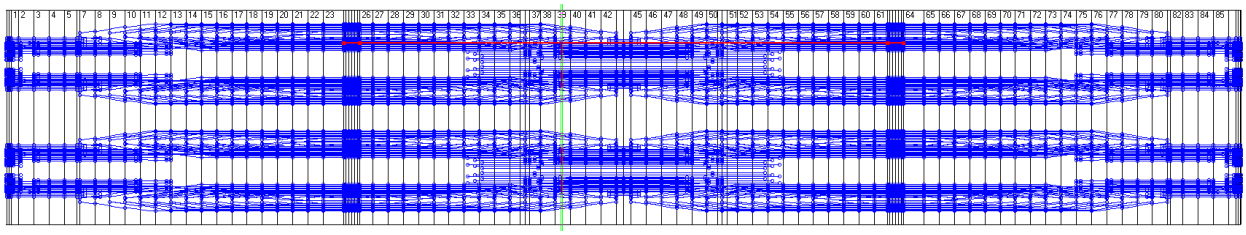
The following describes the layout of external retrofit tendons. No deviators were applied; however, their addition would likely not have a big impact to the considered profile and results due to the small angle change. Deviators are added to provide additional control during tendon placement.

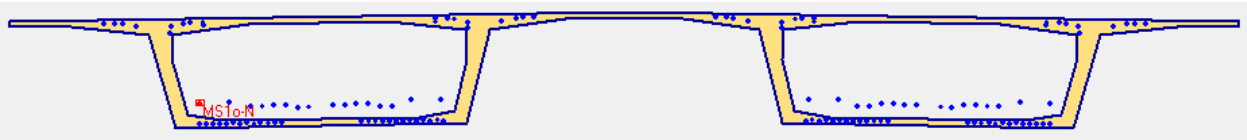
Side Span PT = 1 (19-0.6) and 1 (12-0.6) per web (results indicate reduction is possible)

Main Span PT = 1 (31-0.6) and 1 (19-0.6) per web in addition to the Crack Arrest PT

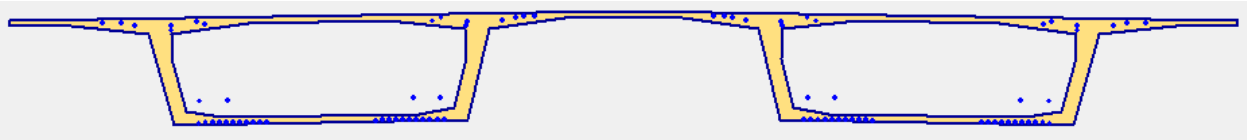


This view in Larsa 4D flattens out the vertical profile of the bridge, the external tendons are straight.

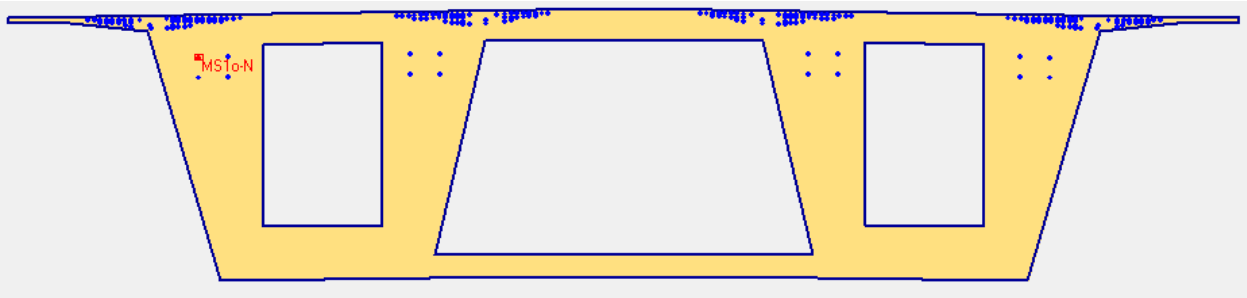




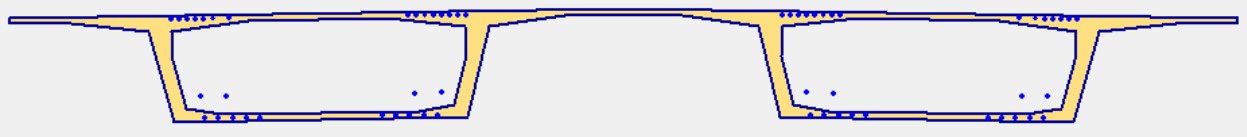
Section at Joint 40 (above)



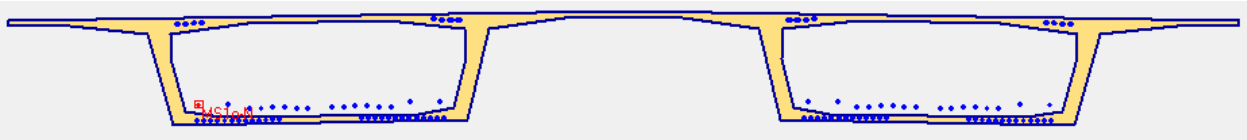
Section at Joint 12 (above)



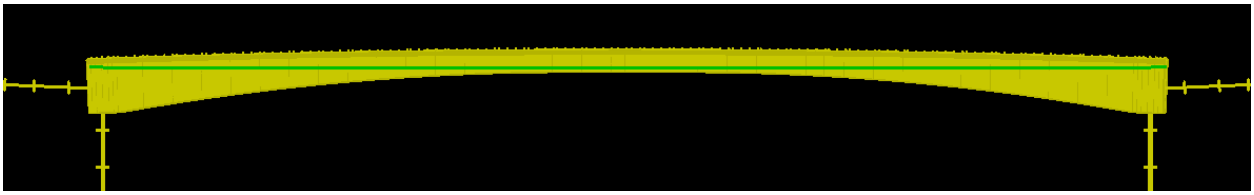
Section at Pier Table (above) – Minor adjustments to final placement anticipated to aid in construction.



Section at Side Span Anchorage (above) – Note that the provided sketch has a staggered anchorage for the side span tendons. This will be included in subsequent iterations for the tendon profile.



Section at Midspan (above)



Mainspan external tendon with vertical profile curvature of the bridge shown for reference.

Discussion of Results for the Current Work-In-Progress Phase II PT Layout

Side Span PT = 1 (19-0.6) and 1 (12-0.6) per web (results indicate reduction is possible)

Main Span PT = 1 (31-0.6) and 1 (19-0.6) per web in addition to the Crack Arrest PT

The goal for this analysis run was to evaluate Strength and Service load combinations that include wind and to attempt to satisfy tension force resultant requirements without considering the soffit CFRP. A two-box model was created for wind, seismic, and eccentric gravity loads. An effort was made to address all remaining Phase I comments. Locked-in forces from the form travelers were included during the main span closure, which helped as the elastic rebound reduced the main span positive moments. The geometry for the hinged cross section was adjusted to match the first-slope stiffness of the moment-curvature response applicable within the range of Service III moments, which helped to reduce the effects of redistribution. The following change increased the demands. Uniform Thermal (TU) effects now consider friction developed at the Pier 15 and 18 supports. These forces were defined based on an assumed 10% coefficient of friction. The following provides a brief discussion of results, with stress limits defined by the original 5 ksi design concrete strength.

The hinged model demands include the effects of redistribution due to softening at Joints 38, which are locked in by the crack arrest retrofit PT. All subsequent loading is applied to the continuous structure. Hinged Model Service III soffit tensile stresses at Joints 38 and 40 are below $6\sqrt{f'c}$. Uncracked Model Service III soffit tensile stresses in both the main and side spans are below $3\sqrt{f'c}$. Under the Hinged model demands, the provided tension force resultant is still about 300 kips short of that required for Strength I demands without consideration for the soffit CFRP. This gap in tension force is well within the

capacity of the soffit CFRP. Use of an external PT stress greater than fpe accounting for deformation compatibility will also decrease and potentially eliminate the gap in the provided tension force resultant. The tension force resultant requirements at Joints 38 and 40 are similar, indicating that no reduction in quantity of main span retrofit tendons is possible given the Hinged Demands.

There is no room for additional main span PT beyond that considered, since the DC+DW+PS soffit compressive stress at Joint 40 evaluate using the Uncracked Demands is just below the 0.45f'c limit. Due to the increased main span positive moments, deck compression is maximized under the Hinged Model demands while soffit compression is governed by the Uncracked Model. Currently, both the deck and soffit Service I compressive stresses at Joint 40 exceed applicable limits under the respective demands when considering ϕw reduction factors for slenderness and the original 5 ksi design concrete strength.

There are similar Service I soffit compression exceedances at Joints 12 in the side spans, but since the Service III tensile stresses are further below 3√f'c, there is some room to decrease the provided PT in a revised layout. I still need to evaluate potential increases in positive moment demands due to softening at Joints 14.

Strength I, III, IV, and V moments were satisfied by the provided strength when considering both the maximum and minimum DC load factors.

I still need to fully evaluate shear and torsion for Strength III. This will include a comparison to the threshold torsion and any modifications to replace Vu by Veff considering torsion in the simplified modified compression field theory per AASHTO LRFD. Lower torsional moments were obtained for Strength V, due to lack of a vertical uplift WS component. A spot check for the threshold torsion at Joint 38 considering K=1.0 due to cracking indicated that values for Str V torsion are below the threshold and should not influence the shear capacity calculations per AASHTO LRFD. After setting up the torsional calculations for Strength III, this check will be expanded to verify.

Current evaluations for wind load only consider 0deg transverse wind. Results for additional angles are needed, starting with 90deg; however, these are likely less important.

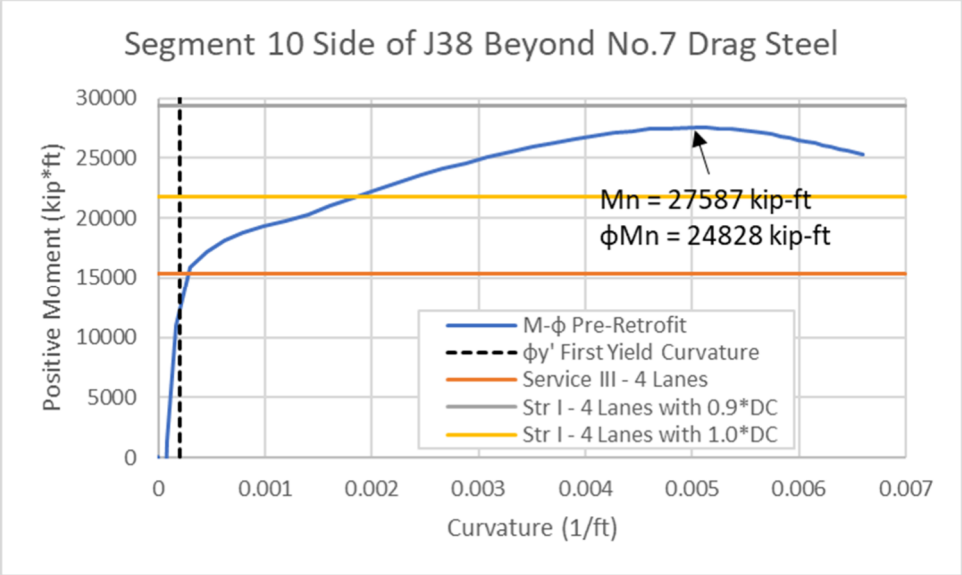
Shear demands from Strength V were found not to govern over those from Strength I. The provided shear strength and tension force resultant were found to satisfy the Strength V demands.

Since no additional PT can be provided without creating more problems for soffit and deck compression, the goal would be to determine if the tendon layout can satisfy the seismic demands. A longitudinal pushover at flexural overstrength of the column plastic hinges has already been setup to determine the superstructure demands. Next, an RSA demand analysis will be setup including a vertical spectra, but first, demands on the foundations will be assessed.

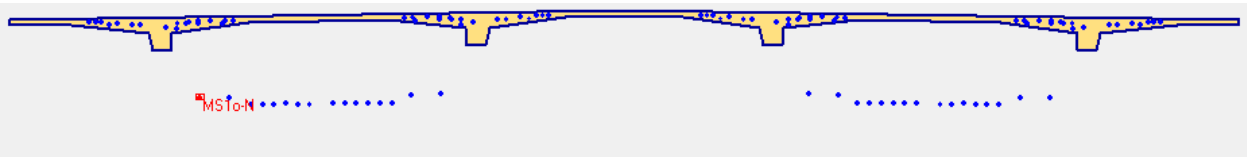
Revised Hinge Section

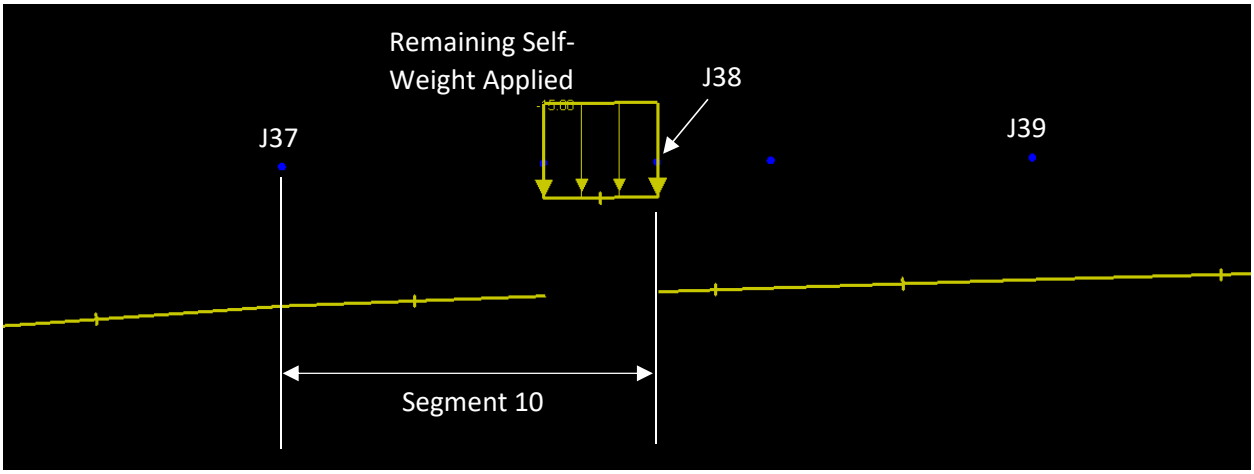
Service III demands at Joint 38 with four design HL93 lanes loaded are enough to yield the soffit reinforcement as shown in the following table and figure. The horizontal component of the diagonal compressive strut developed to resist the applied shear also increases the soffit tension, thus the soffit reinforcement is assumed to have yielded under the Service III demands.

Service III Moments at Joint 38 - Existing Bridge		
DC	-75422	kip-ft
DW	1105	kip-ft
PT Secondary	59070	kip-ft
TU -40deg	3229	kip-ft
TG Deck Increase	14950	kip-ft
HL93 Three Lanes	19545	kip-ft
HL93 Four Lanes	19929	kip-ft
Service III - 4 Lane	15386	kip-ft
Str I - 4 Lane with 1.0 DC	21795	kip-ft
Str I - 4 Lane with 0.9 DC	29337	kip-ft

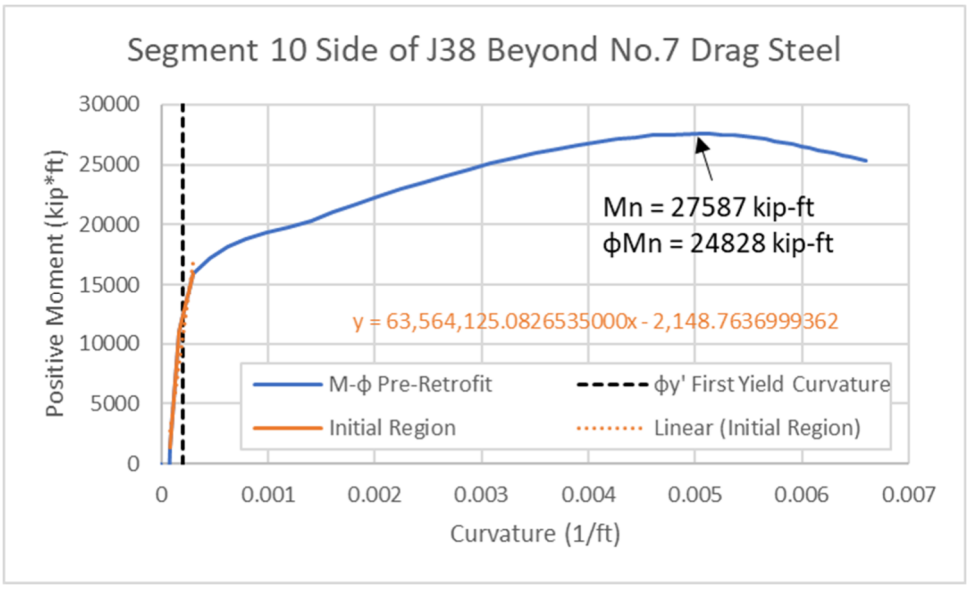


A 5ft long hinged section was introduced on the Segment 10 side of Joint 38. The moment of inertia for the hinged section was revised to reflect the post-cracking first-slope stiffness of the moment-curvature response, applicable within the range of Service III moments. The hinged section extended 3.75ft below the deck surface, providing Ig = 93.07 ft^4, which compares well with values obtained from bilinearization of the moment-curvature response. The following provides a diagram of the section and supporting calculations. Note that the similar results for the moment-curvature response at Joint 38 were obtained between Xtract analysis and use of the Mathcad moment-curvature script.





Cracked Section Stiffnes via Moment-Curvature		
Ec from LRFD	4592	ksi
Icr = My'/(Ec*ϕy')	91.78	ft^4
Ig	3057	ft^4
Ratio Icr/Ig	0.03	at J38
Graphical Comparison using Initial Slope		
Ec*Icr	63564125	kip*ft^2
Ec from LRFD	661248	kip/ft^2
Icr	96.13	ft^4



Midspan Closure Locked-In Force Consideration

Results include locked-in forces from form travelers (120 ton each) placed on the Pier 16 side of the midspan closure as shown in the figure below from Ching (1984). Once removed, the elastic rebound decreases the midspan positive moments, but leads to slight asymmetric behavior. The Pier 17 side Joint 38 and 40 become the controlling locations for positive moments and thus shear.

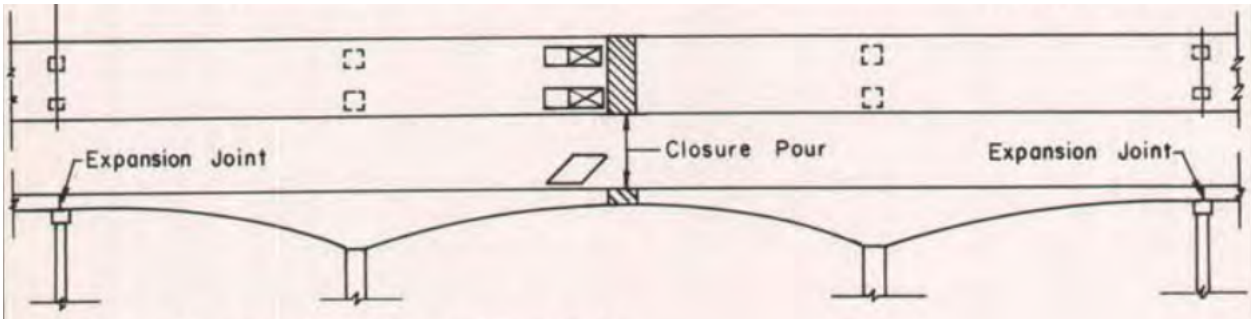
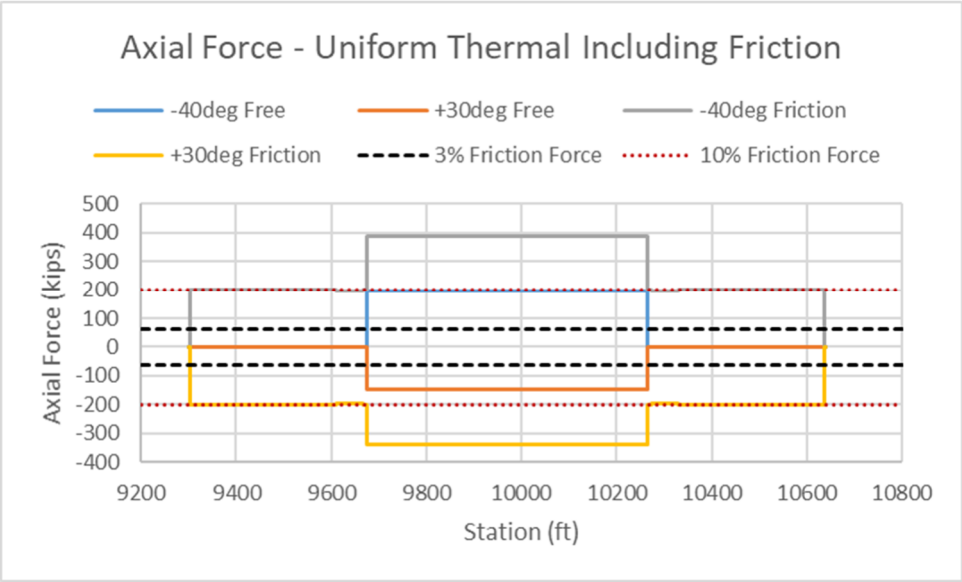
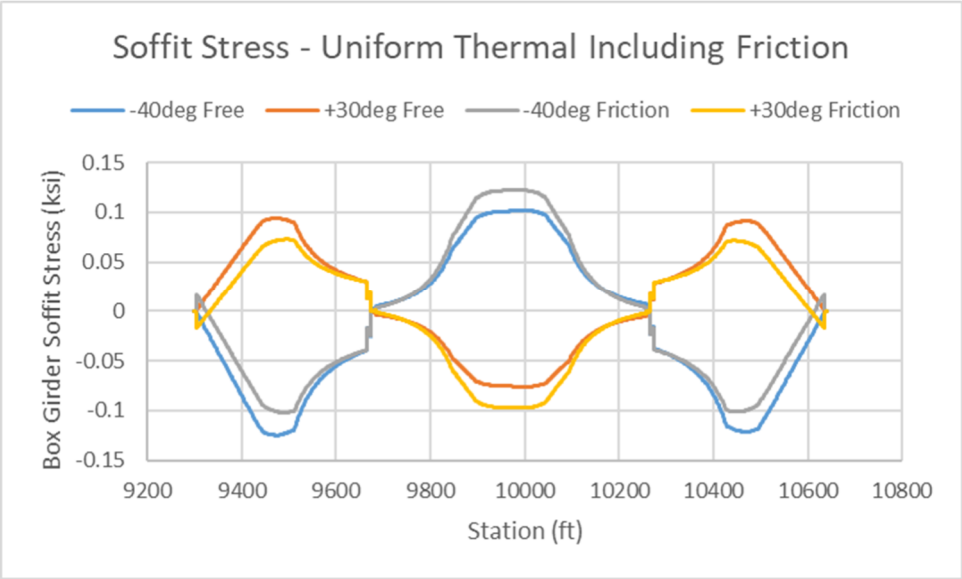


Fig. 10. Actual construction sequence.

Adjusted TU Effects for Friction Developed at Piers 15 and 18 (per internal comment)

Friction developed at the Piers 15 and 18 vertical bearings was accounted for in revised estimates for Uniform Thermal (TU) effects. The friction force was calculated with the DC+DW+PS reaction and an assumed 10% coefficient of friction. These forces were applied in directions that oppose the thermal contraction or expansion of the box girder ends near Piers 15 and 18 in respective load cases. The applied forces influence the axial, shear, moment, and stress results for TU. The following figures show a comparison of the axial forces and soffit stresses under TU effects with and without friction considered.

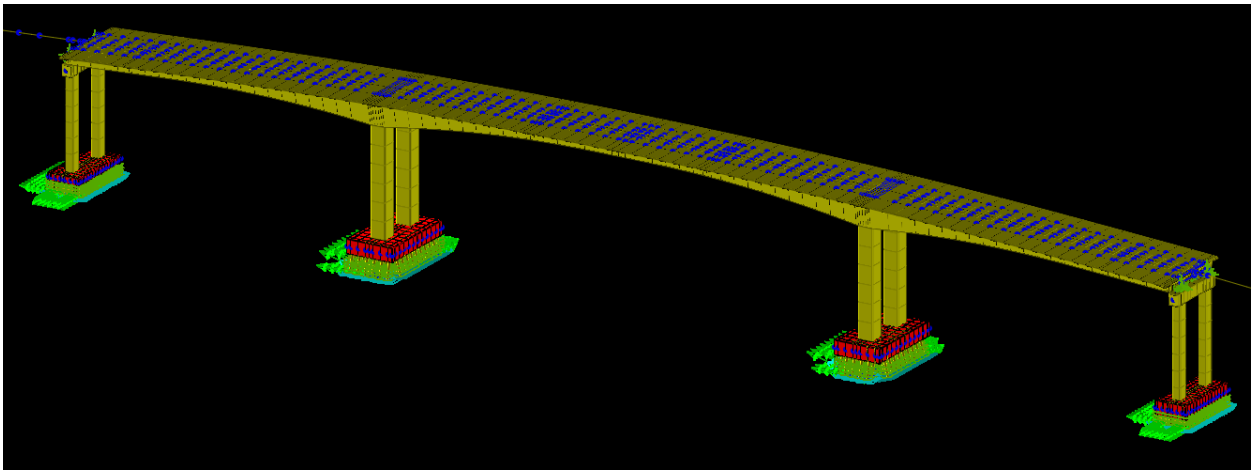




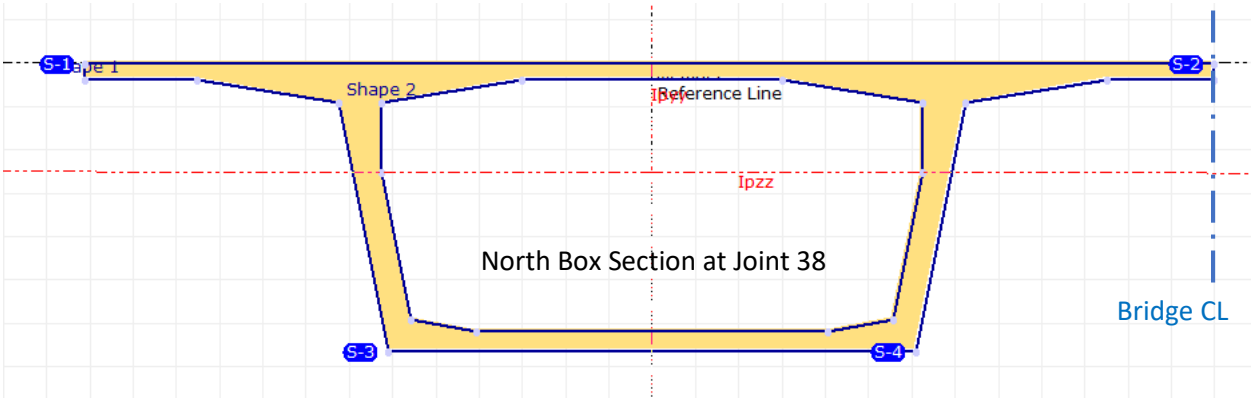
Increase in main span demands when accounting for friction developed at Piers 15 and 18 supports.

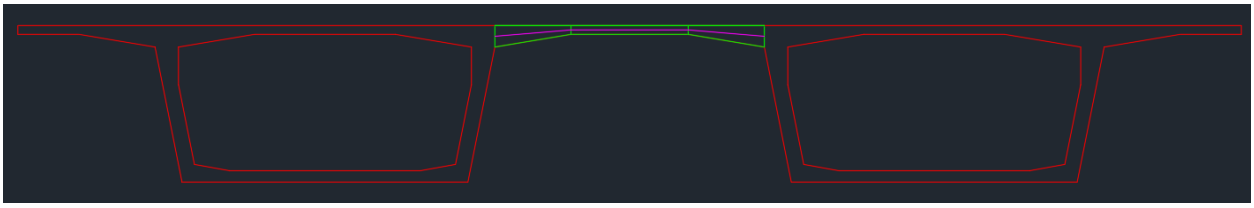
Two-Box Model for Wind, Seismic and Eccentric Gravity Loading

The two-box model has individual frame members for the north and south boxes linked by members that recreate the transverse stiffness of the deck between the interior box webs. Model results are superimposed over the time-dependent DC+DW+PS effects from the single frame model.

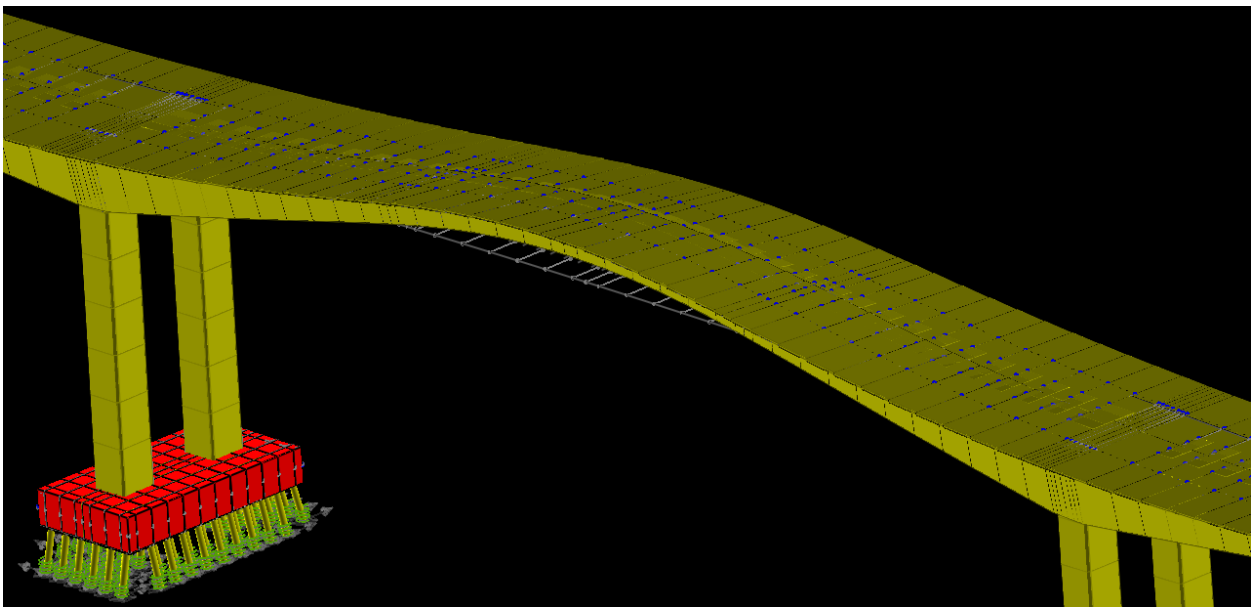


A sample north box cross section at Joint 38 is provided below. Sections for the South box were mirrored about the profile line (located at stress point S-2 below). All sources of DC and DW dead loads were added to the two-box model such that the correct distribution of weight and stiffness is achieved for seismic analysis. The additional transverse members were massless. These effects were verified in a one-to-one comparison between the single frame and two-box models using a static solution for DC+DW loading (fictitious load case that doesn't account for cantilevered construction effects).

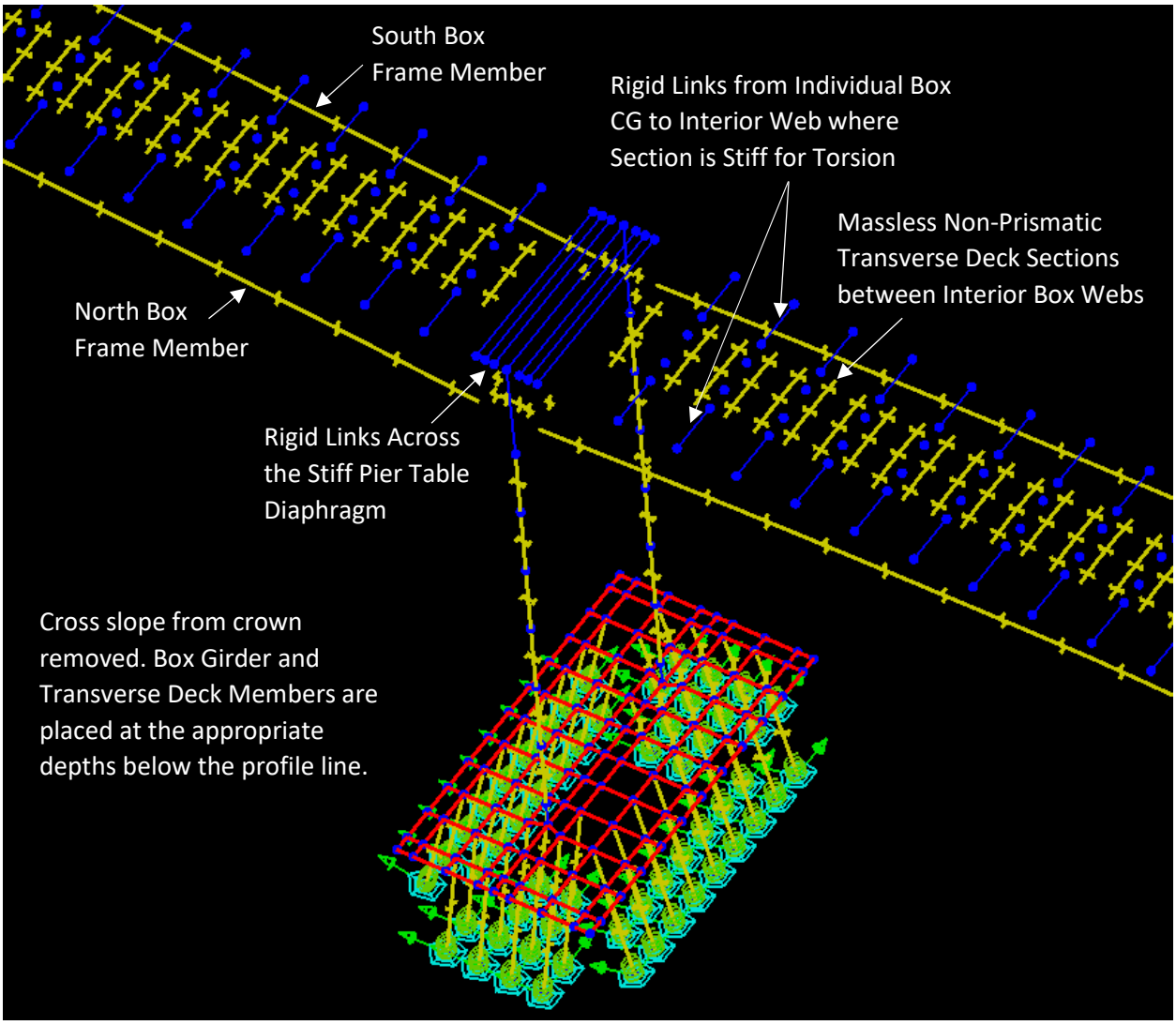




The transverse non-prismatic members recreate the region shown in green above.



Example exaggerated deflection showing behavior of the two boxes for Str III Uplift WS Forces only.



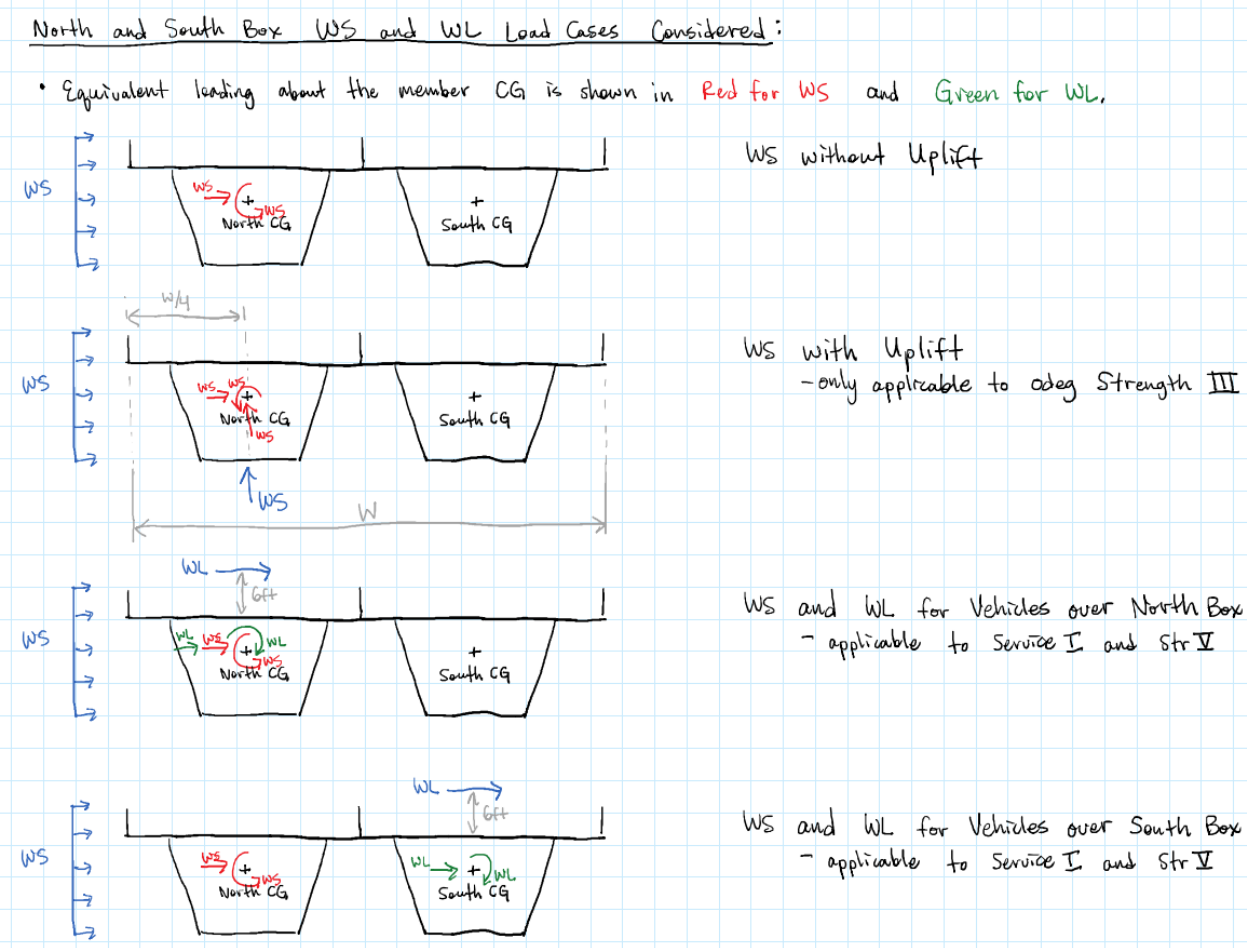
Wind Loads (WS and WL) for Service I, Strength III and Strength V Load Combinations

Calculations for the design wind pressures following AASHTO LRFD 9th Ed. Section 3.8 appear on the following page. Wind Exposure Category D was conservatively used. Wind on Structure (WS) forces were obtained based on the average height of the superstructure in each segment, including both the barrier and cross slope contributions. WS forces were applied as a uniformly distributed lateral load and torque in each segment, accounting for the eccentricity between the resultant force and the average member CG within the segment. The quarter-point of the total roadway width aligns with the CG of one of the box girders; therefore, the vertical WS uplift force was applied to the windward box without eccentricity. The vertical WS uplift force generates torsion in both boxes. Wind on Live Load (WL) was applied 6ft above the roadway surface and eccentricity effects were accounted for using a uniform lateral force and torque within each segment. Separate cases that consider WL applied to vehicles on the windward and leeward boxes were considered. Due to symmetry, only wind from north to south was considered. Pictures depicting the WS and WL loading and equivalent effects about the CG are provided for clarification. Directionality for the torsional effects depend on the sign of the eccentricity.

Calculate the Design Wind Pressures, Pz, per AAHSOT LRFD 9th Ed. Section 3.8		
Load Combination	**Design 3-Second Gust, V (mph)**	from Table 3.8.1.1.2-1
Strength III	110	from Fig. 3.8.1.1.2-1
Strength V	80	
Service I	70	
Ground Surface Roughness Category = C from Section 3.8.1.1.4		
Wind Exposure Category = D (conservatively) from Section 3.8.1.1.5		
G	1.0	from Table 3.8.1.2.1-1
Z (ft)	160	at midspan with barrier
Pressure Exposure Coefficient Applicable for Srength III		
Kz(D)	1.52	from Eq. 3.8.1.2.1-4
Kz(D) is taken as 1.0 for other load combinations		
Cd superstructure	1.30	windward, from Table 3.8.1.2.1-1
Cd substructure	1.60	windward
	Design Wind Pressure, Pz (ksf)	from Eq. 3.8.1.2.1-1
Load Combination	**Superstructure**	**Substructure at Same Elevation**
Strength III	0.0613	0.0754
Strength V	0.0213	0.0262
Service I	0.0163	0.0201

Results shown only consider 0deg wind from the north to the south, due to symmetry. Wind forces at 90deg are still under consideration but have not been evaluated due to the following question regarding the approach. These effects can be easily added. Note that this document was originally written for internal coordination.

- For longitudinal (90deg) wind, should the vertical curvature of the bridge be considered for determining lateral wind forces? The difference in elevation is (257.45 - 133) = 24.45 ft. The roadway width is 105.92 ft. For the 90deg Strength III case, the resulting longitudinal superstructure WS force would be (105.92 ft * 24.45 ft *0.06125 ksf) = 158.6 kips. Presumably, this load would be applied as a uniformly distributed longitudinal load to the projected area based on the change in height between ends of each segment. As a purely longitudinal load, both axial forces and moments would be generated due to the inclined member CG. The magnitude of loading would appear to be small compared to the transverse (0deg) case. I'll look into this after setting up the pushover models and RSA analysis with a vertical spectra.



Wind Load Cases and Calculation of Envelope Effects

The following load cases were considered. Case #N or #S refers to load effects for the north windward or south leeward boxes, respectively.

Strength III Envelope of Wind Cases 1N-6N and separately Cases 1S-6S

- Cases 1N and 1S – Str III Wind without Uplift, including the following:
  - WS for Str III
- Cases 2N and 2S – Str III Wind with Uplift
  - WS for Str III
  - Uplift WS for Str III
- Cases 3N and 3S – Str III Wind without Uplift (main span only)
  - WS for Str III – MS
- Cases 4N and 4S – Str III Wind with Uplift (main span only)
  - WS for Str III – MS
  - Uplift WS for Str III – MS
- Cases 5N and 5S – Str III Wind without Uplift (side spans only)
  - WS for Str III – SS

- Cases 6N and 6S – Str III Wind with Uplift (side spans only)
    - WS for Str III – SS
    - Uplift WS for Str III – SS

Strength V Envelope of Wind Cases 7N-9N and separately Cases 7S-9S

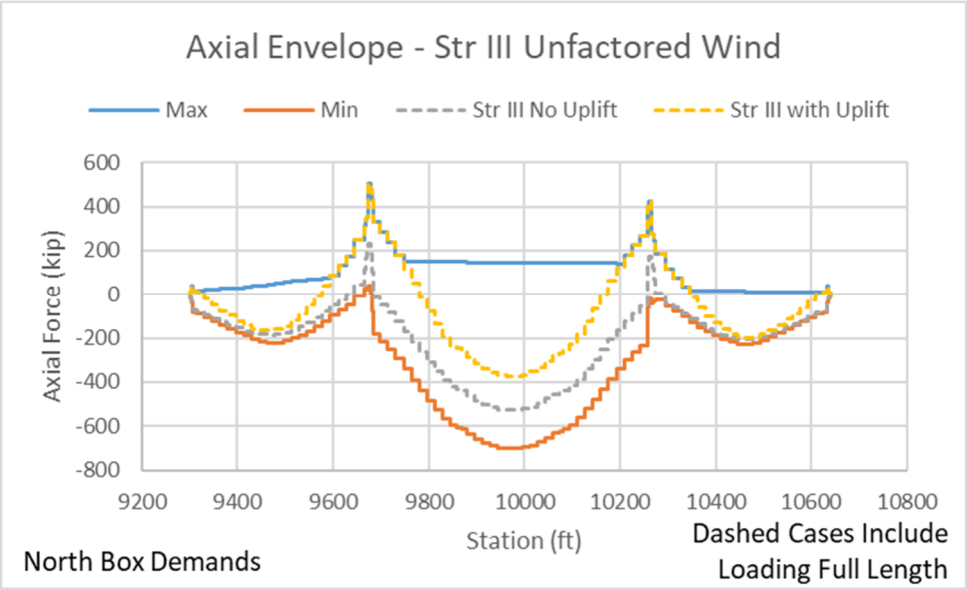
- Case 7 – Str V Wind, including the following for individual North/South Cases
    - Case 7N: WS for Str V and WL North Box
    - Case 7S: WS for Str V and WL South Box
- Case 8 – Str V Wind (main span only)
    - Case 8N: WS for Str V-MS and WL North Box-MS
    - Case 8S: WS for Str V-MS and WL South Box-MS
- Case 9 – Str V Wind (side spans only)
    - Case 9N: WS for Str V-SS and WL North Box-SS
    - Case 9S: WS for Str V-SS and WL South Box-SS

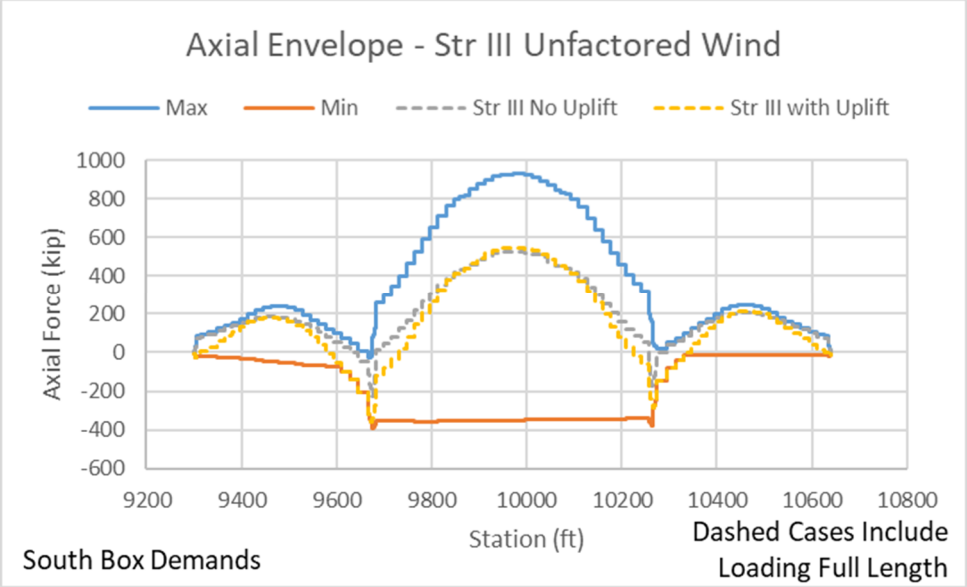
Service I Envelope of Wind Cases 10N-12N and separately Cases 10S-12S

- Case 10N – Service I Wind, including the following:
    - WS for Service I and North box loading for either WL North Box or WL South Box
- Case 10S – Service I Wind
    - WS for Service I and South box loading for either WL North Box or WL South Box
- Case 11N – Service I Wind (main span only)
    - WS for Service I-MS and North box loading for either WL North Box-MS or WL South Box-MS
- Case 11S – Service I Wind (main span only)
    - WS for Service I-MS and South box loading for either WL North Box-MS or WL South Box-MS
- Case 12N – Service I Wind (side spans only)
    - WS for Service I-SS and North box loading for either WL North Box-SS or WL South Box-SS
- Case 12S – Service I Wind (side spans only)
    - WS for Service I-SS and South box loading for either WL North Box-SS or WL South Box-SS

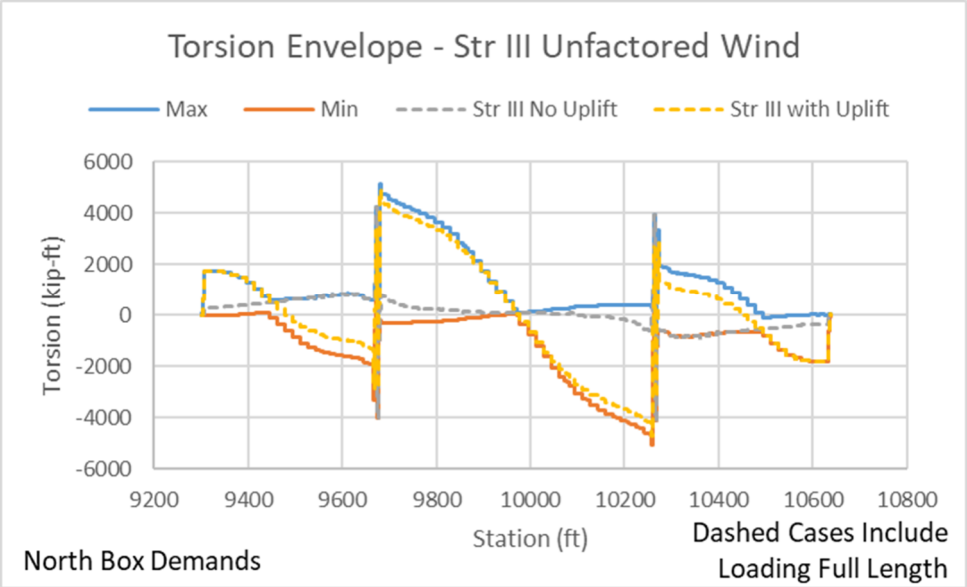
Envelope Wind Effects for Strength III (only WS forces shown)

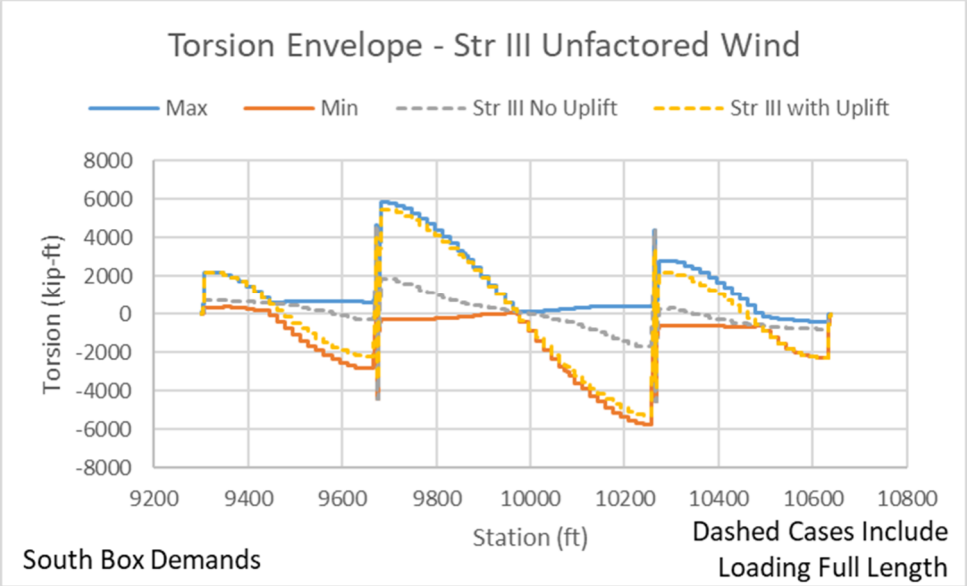
Results are provided for the North windward box and the South leeward box under transverse wind.



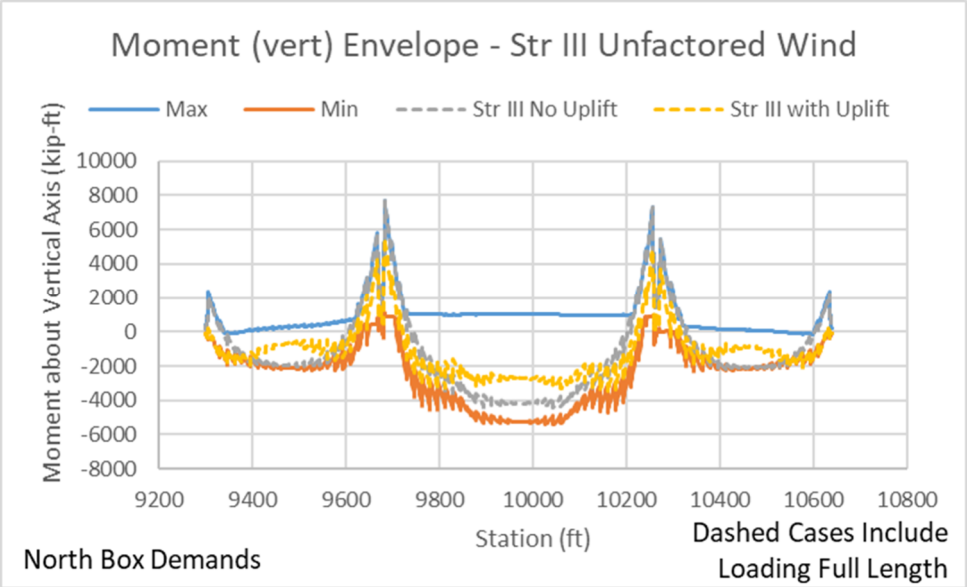


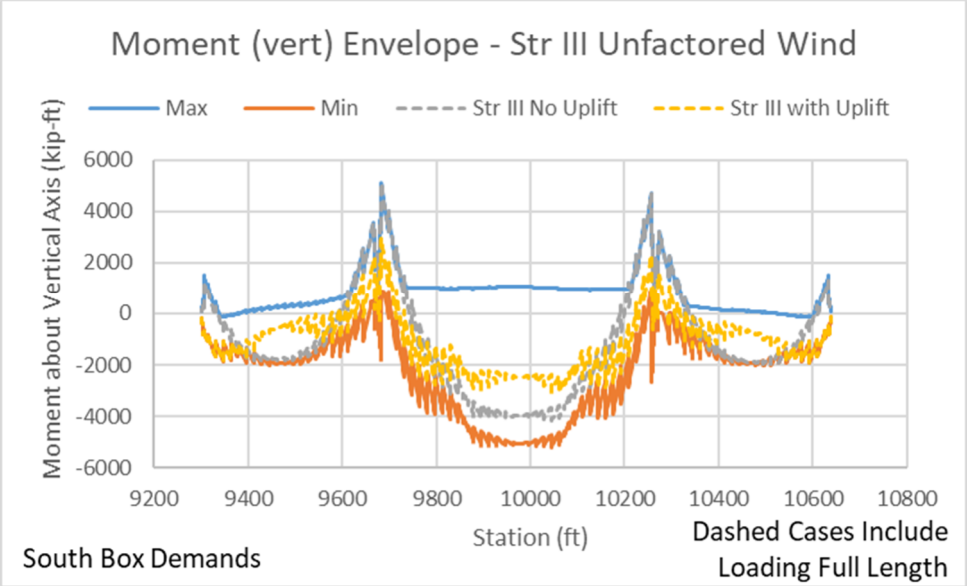
As shown in the figures above, the north (compression) and south (tension) boxes form an axial force couple to resist the applied lateral loads. The dashed lines show results for wind forces applied over the full bridge length, while the envelope results consider the cases previously discussed. The stepped nature of the response is due to the discrete transverse members applied at segment ends transferring load between the two boxes.



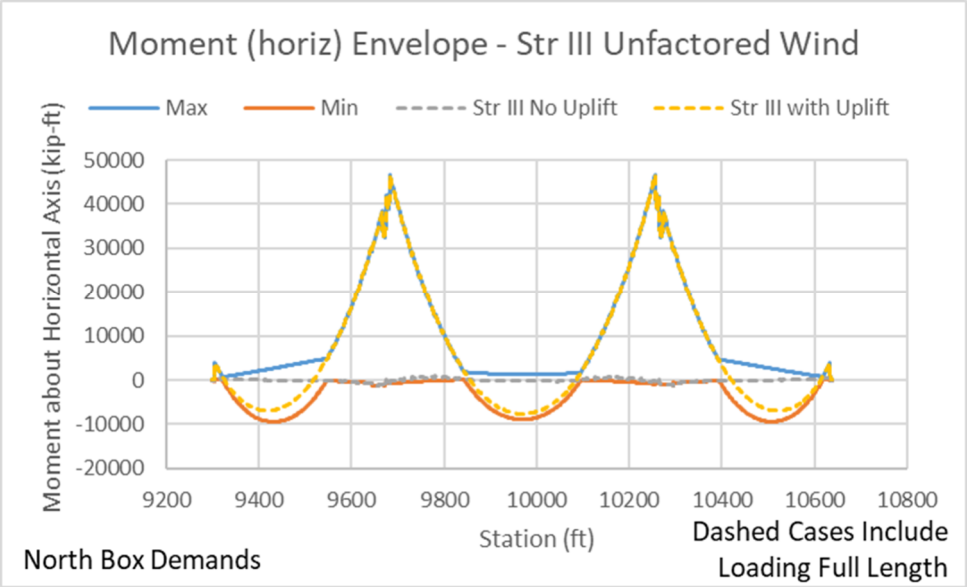


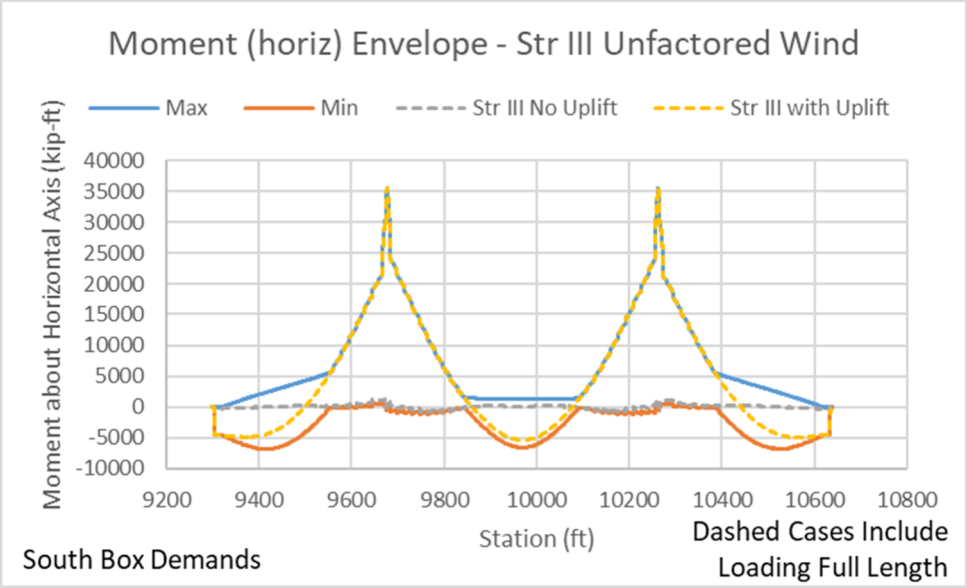
As shown above, the leeward South box develops a higher torsion under the WS uplift forces applied to the windward North box. I still need to fully evaluate shear and torsion for Strength III. This will include a comparison to the threshold torsion and any modifications to replace Vu by Veff considering torsion in the simplified modified compression field theory per AASHTO LRFD.





Moments due to the lateral component of the wind force are shown above.

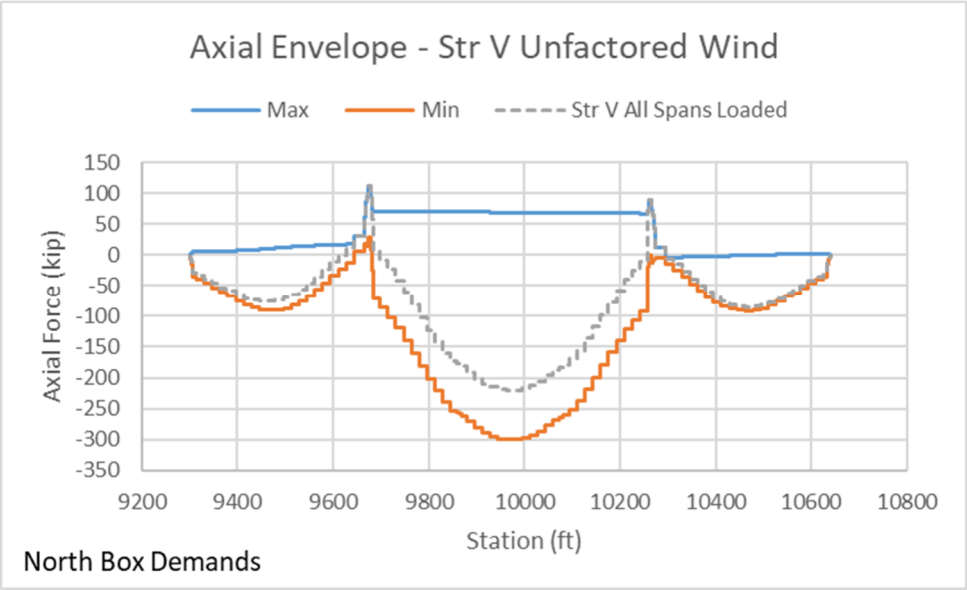




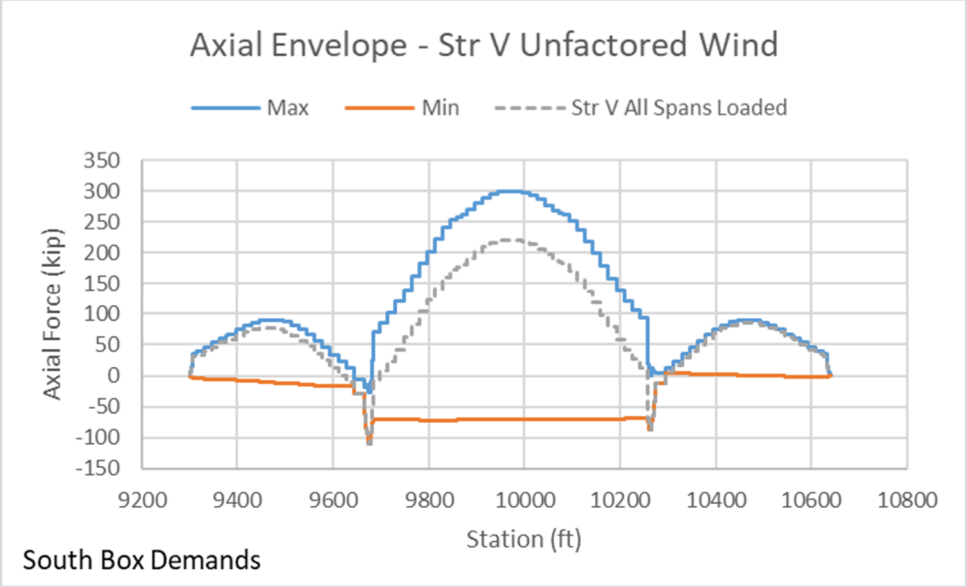
Moments about the horizontal axis (same as for gravity loading) are primarily due to the WS uplift force. Results show the portion of the load shared by the adjacent leeward South box.

Envelope Wind Effects for Strength V (only WS and WL forces shown)

Results are provided for the North windward box and the South leeward boxes under transverse wind.

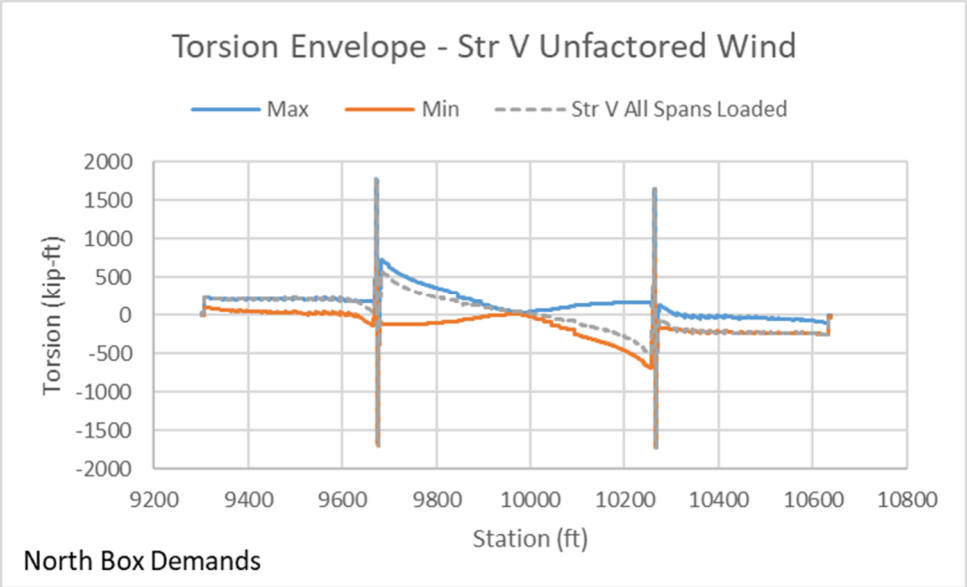


North Box Demands

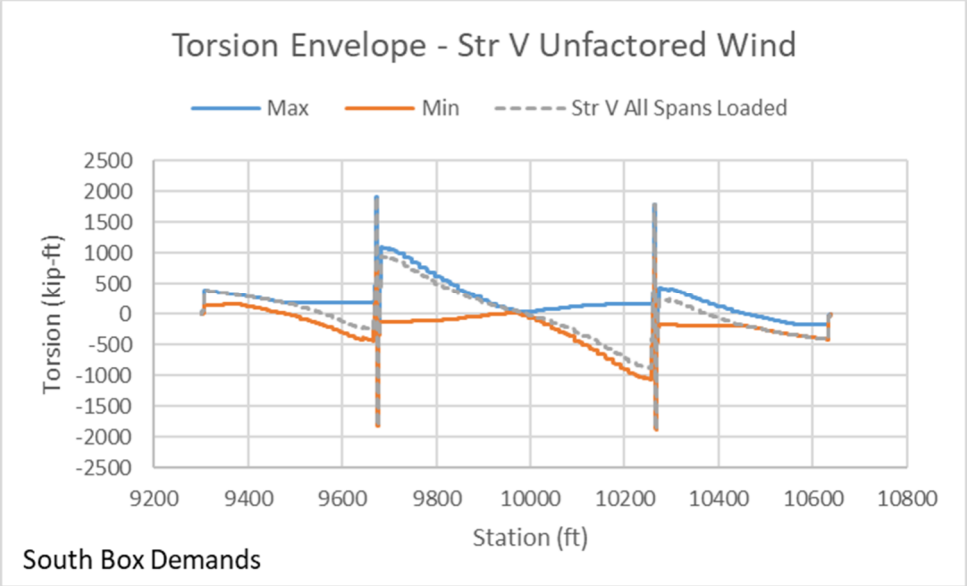


South Box Demands

The same axial force couple between the windward and leeward boxes exists.

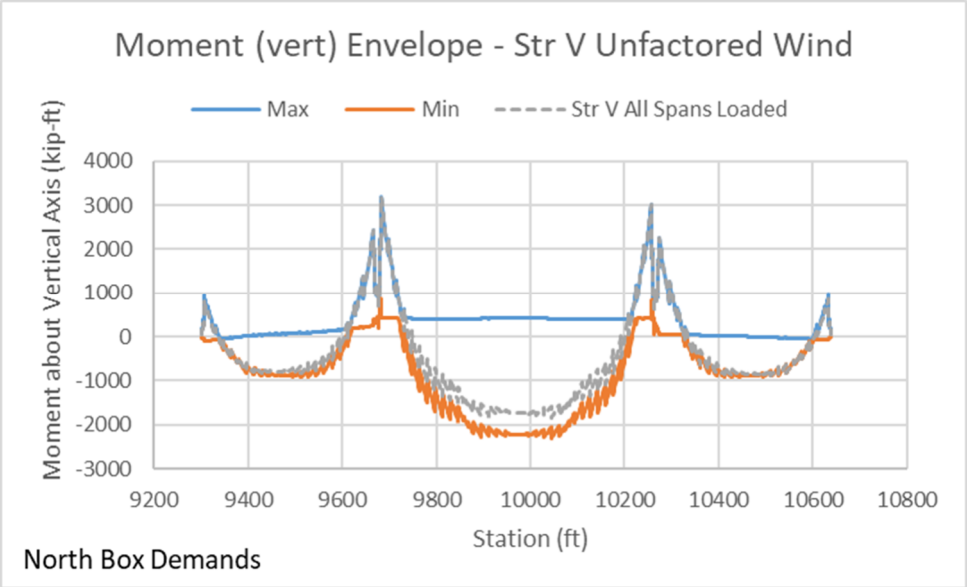


North Box Demands

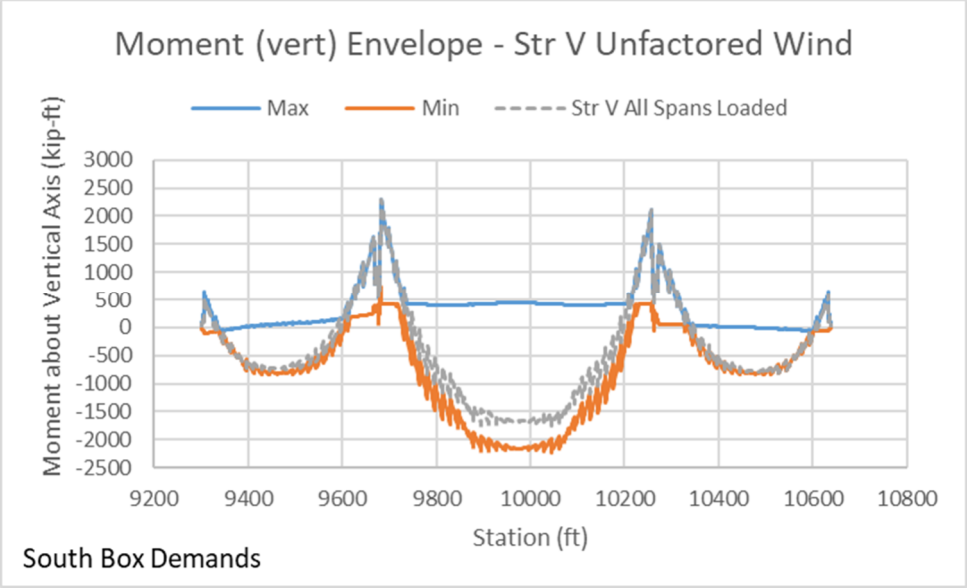


South Box Demands

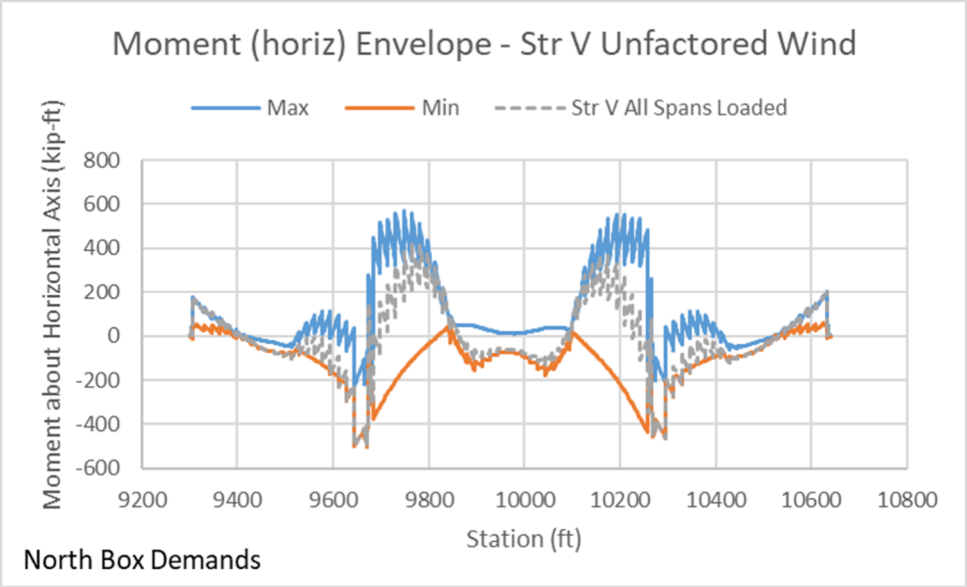
Lower torsional moments were obtained for Str V, due to lack of a vertical uplift WS component. A spot check for the threshold torsion at Joint 38 considering K=1.0 due to cracking indicated that values for Str V torsion are below the threshold and should not influence the shear capacity calculations per AASHTO LRFD. After setting up the torsional calcs for Str III, this check will be expanded to verify.



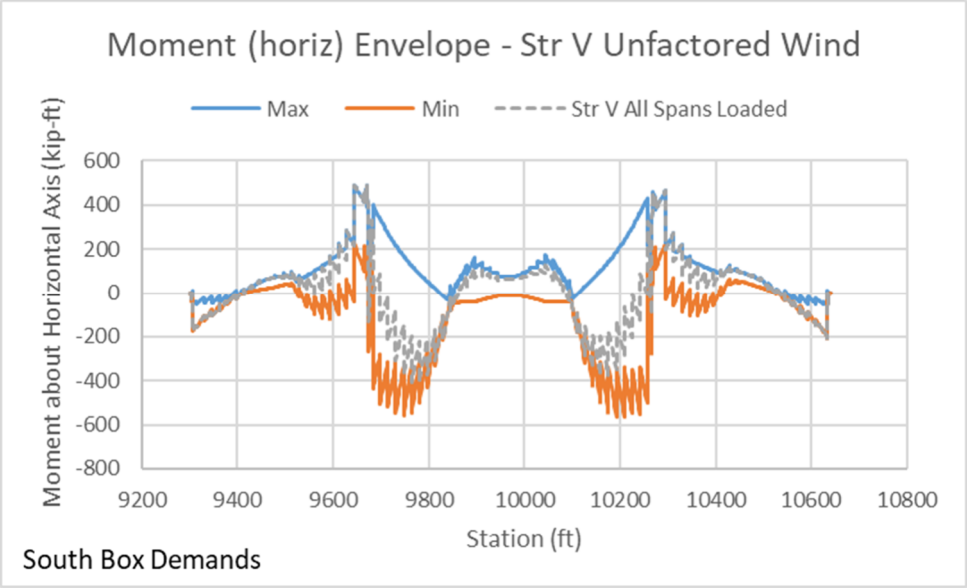
North Box Demands



South Box Demands



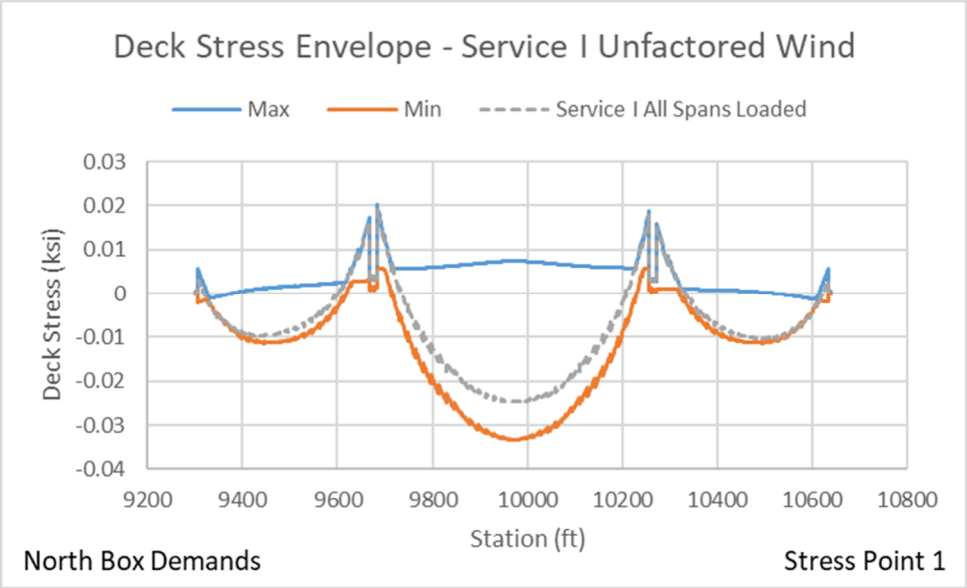
North Box Demands

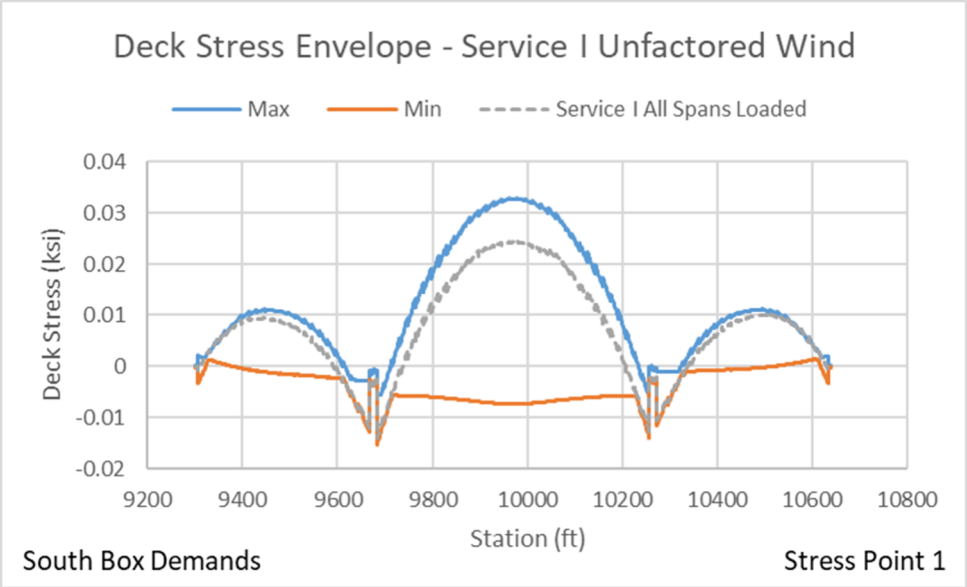


South Box Demands

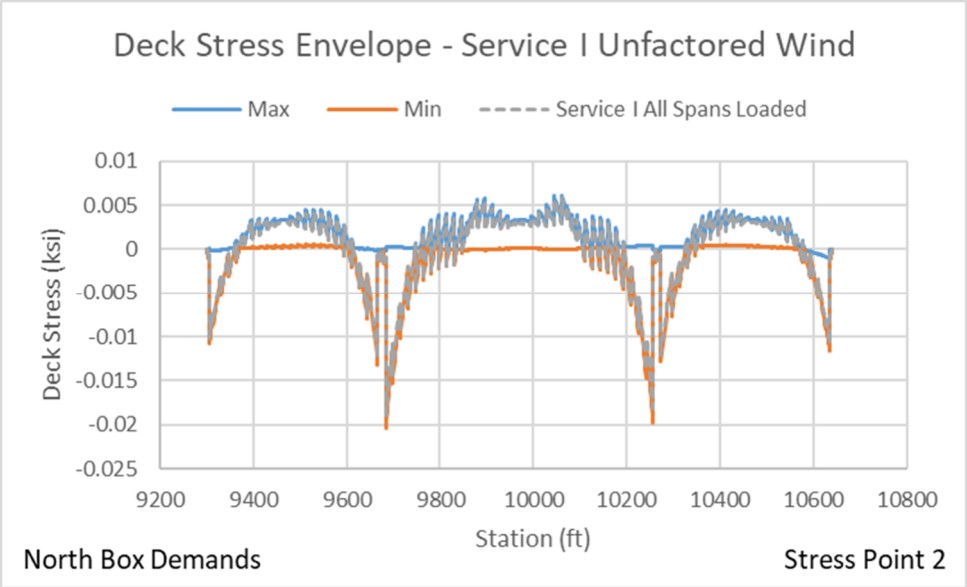
Envelope Wind Effects for Service I (only WS and WL effects shown)

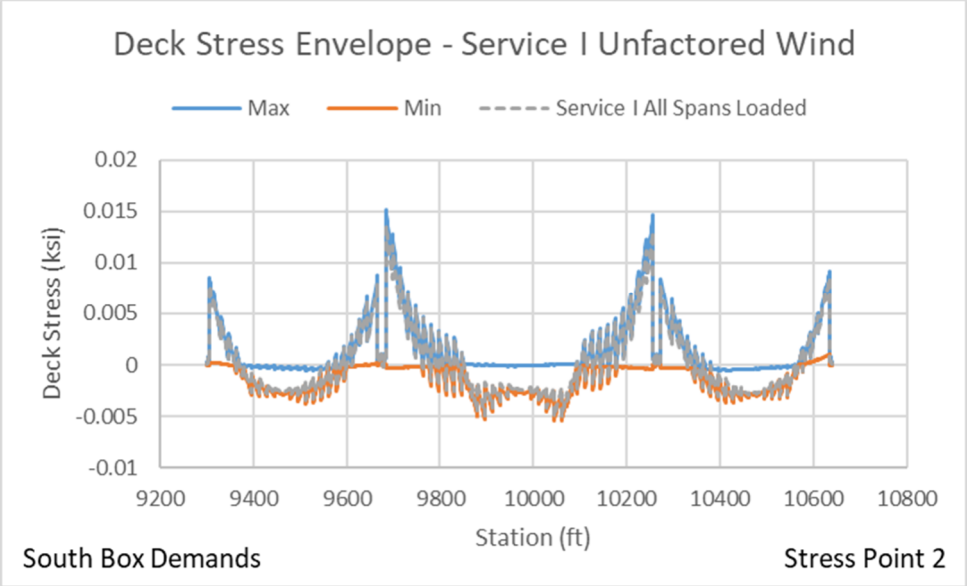
Results are provided for the North windward box and the South leeward box under transverse wind. Stress points 1 and 2 for the deck and 3 and 4 for the soffit were shown previously. Stress point 1 is on the tip of the deck overhang while stress point 2 is at the bridge centerline. Stress point 3 is on the soffit at the corner of the exterior web while point 4 is at the interior web. All four locations were checked, due to the complexity in the loading.



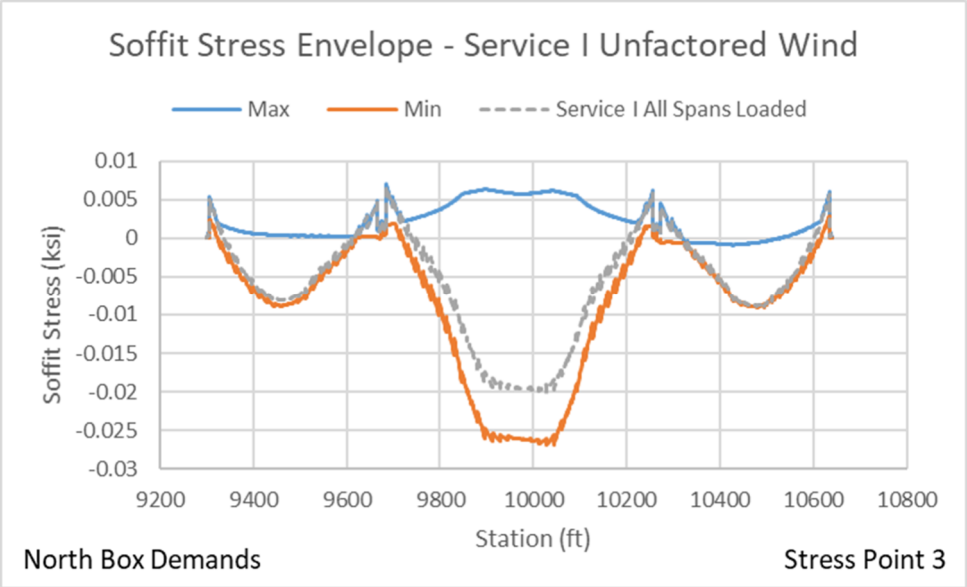


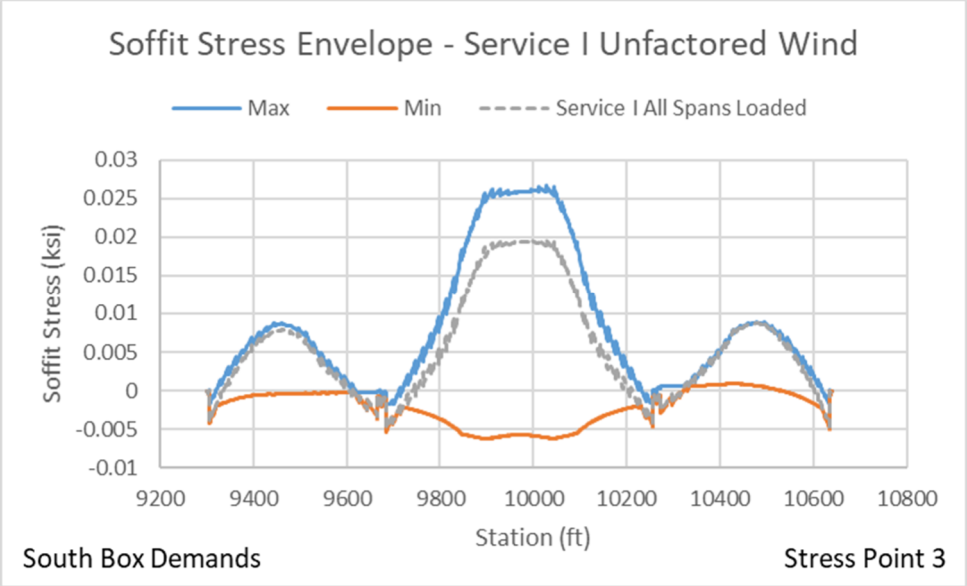
Results for stress point 1 at the tip of the deck overhang are presented above. The north windward box is under compression while the south leeward box is in tension.



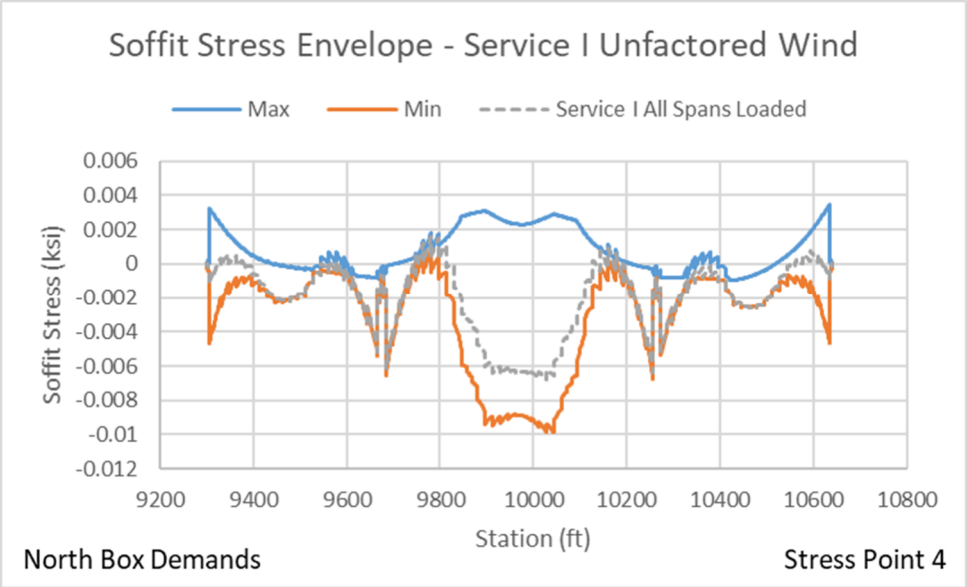


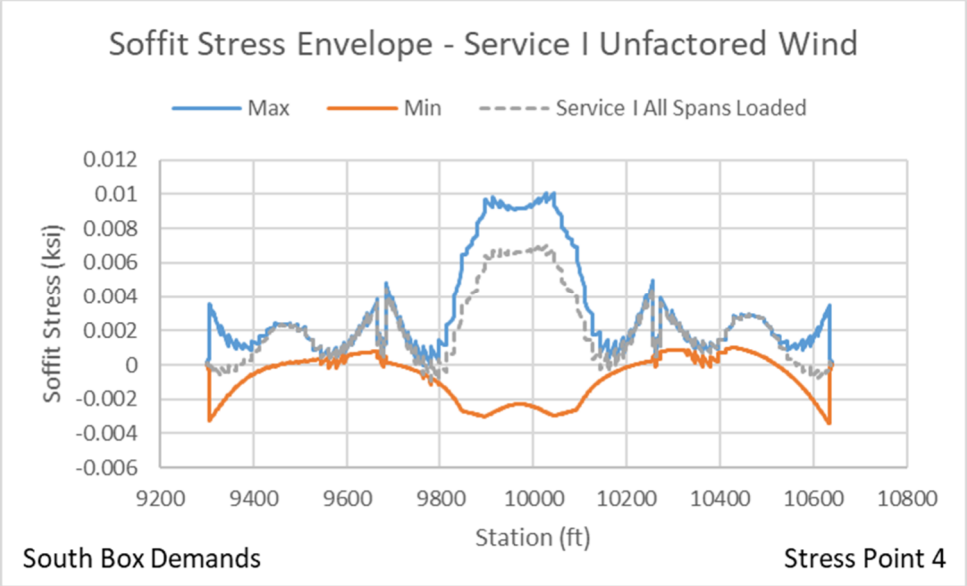
Stress point 2 is at the profile line. Although the north and south boxes share this common location, the individual frame member results must consider the axial force couple under transverse loading, which contributes to the difference in stresses.





Stress point 3 is located at the soffit at the corner of the exterior web. The highest soffit compressive stresses are observed on the windward North box at stress point 3.





Stress point 4 is located on the soffit at the corner of the interior web.

Effective Prestresses at 75yrs after All Losses

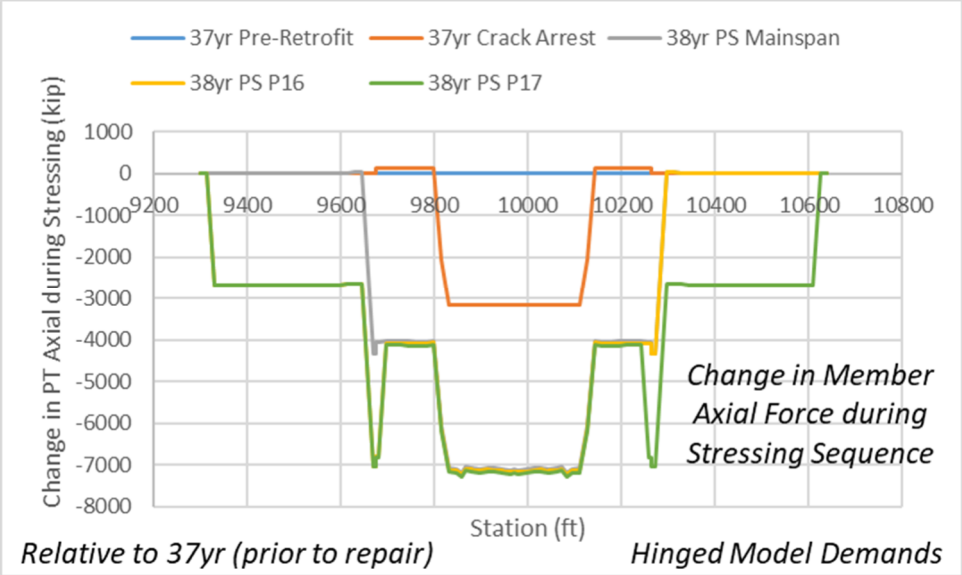
The following describes the effective prestress forces at 75yrs after construction including all losses. MS1 and MS2 are the main span permanent strengthening tendons, with "I" and "o" representing inside or outside locations within a given box girder relative to the profile line. MS1 (31-0.6) is placed closest to the webs, followed by MS2 (19-0.6). For the side spans, the same naming sequence applies or SS1 and SS2, with P16 and P17 representing the given side spans based on the anchorage locations. The SS1 (19-0.6) tendon is closest to the web, followed by SS2 (12-0.6).

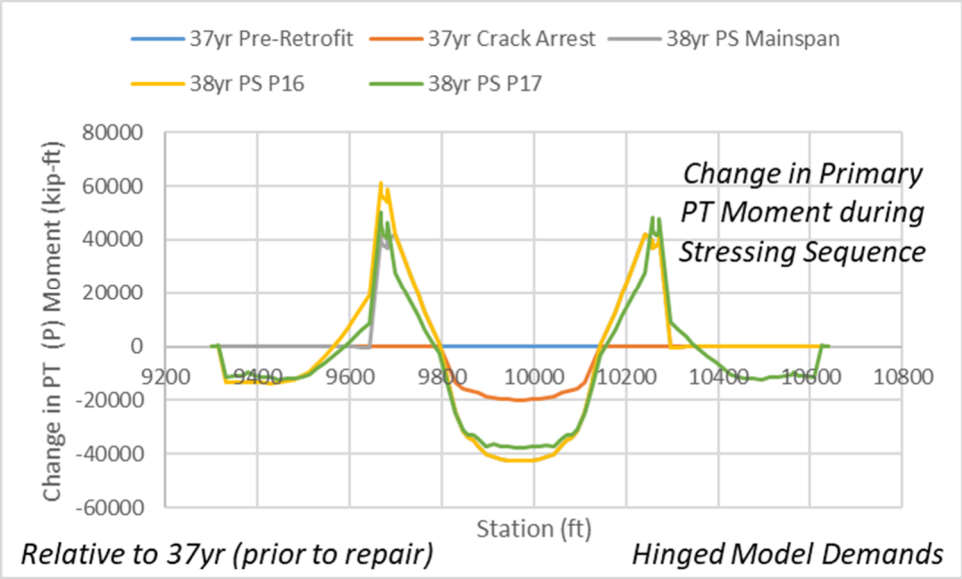
Crack Arrest Tendons 75yrs (Fpj = 280 kip)			
Tendon	**Fpe (kip)**	**Aps (in^2)**	**fpe (ksi)**
MT1	254.1	1.519	167.3
MT4	254.3	1.519	167.4
MT7	254.7	1.519	167.7
MT5	254.9	1.519	167.8
MT2	254.8	1.519	167.7
MT3	256.8	1.519	169.1
MT12	255.7	1.519	168.3
MT9	255.8	1.519	168.4
MT6	255.7	1.519	168.3
MT8	255.9	1.519	168.4
MT11	256.2	1.519	168.7
MT10	256.4	1.519	168.8
MT AVG	**255.4**	**1.519**	**168.2**

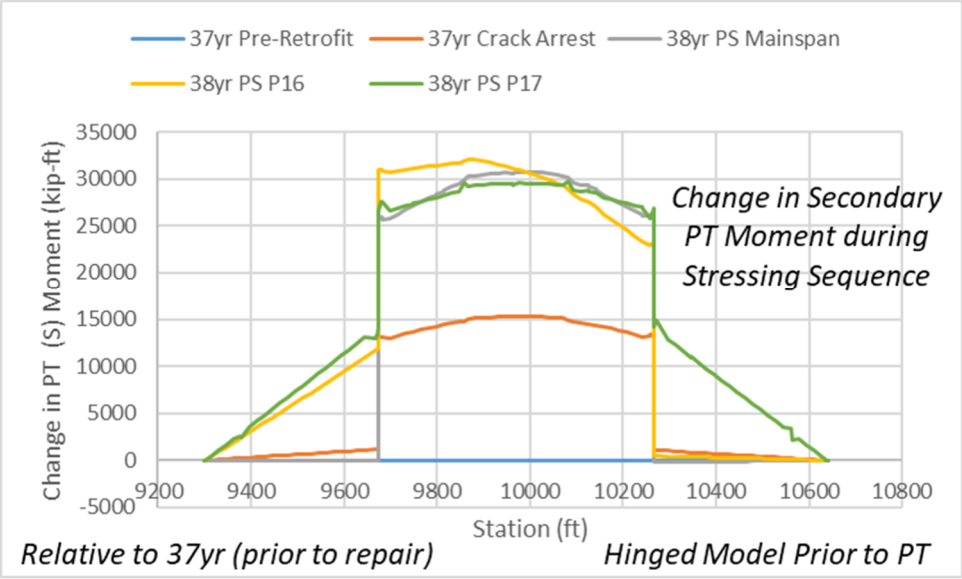
Perm Str Tendons 75yrs (fpj = 0.75fpu)			
Tendon	**Fpe (kip)**	**Aps (in^2)**	**fpe (ksi)**
MS2i	745.2	4.123	180.7
MS2o	747.0	4.123	181.2
MS1i	1220.2	6.727	181.4
MS1o	1224.1	6.727	182.0
SS1i-P16	747.3	4.123	181.2
SS1o-P16	748.5	4.123	181.6
SS2i-P16	471.9	2.604	181.2
SS2o-P16	472.4	2.604	181.4
SS1i-P17	752.8	4.123	182.6
SS1o-P17	753.0	4.123	182.6
SS2i-P17	471.5	2.604	181.1
SS2o-P17	471.6	2.604	181.1
MS AVG	**984.1**	**5.425**	**181.4**
SS AVG	**611.1**	**3.3635**	**181.7**

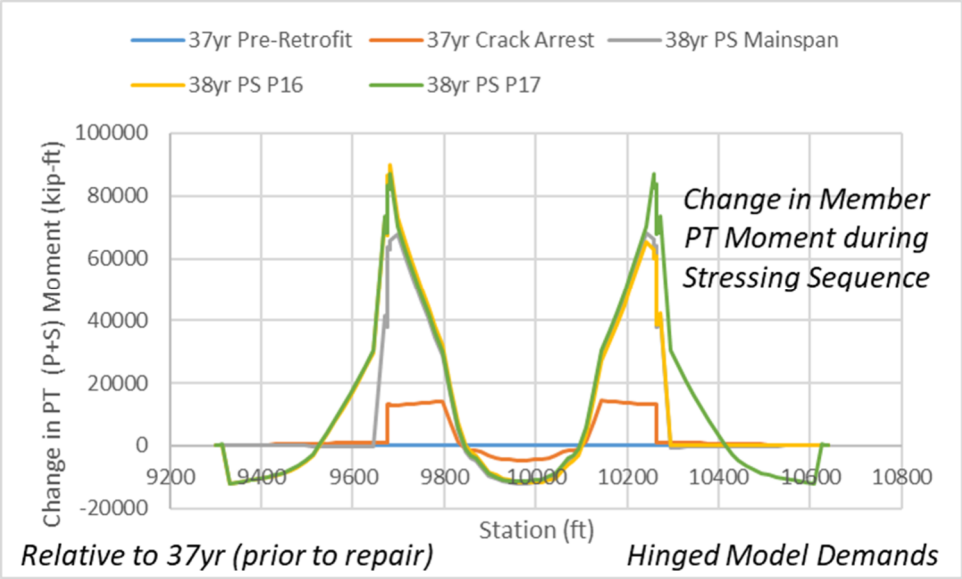
Member PT Load Effects Relative to 37yrs After Construction, but before any Retrofit PT

The following provides the change in member PT load effects during the stressing sequence.

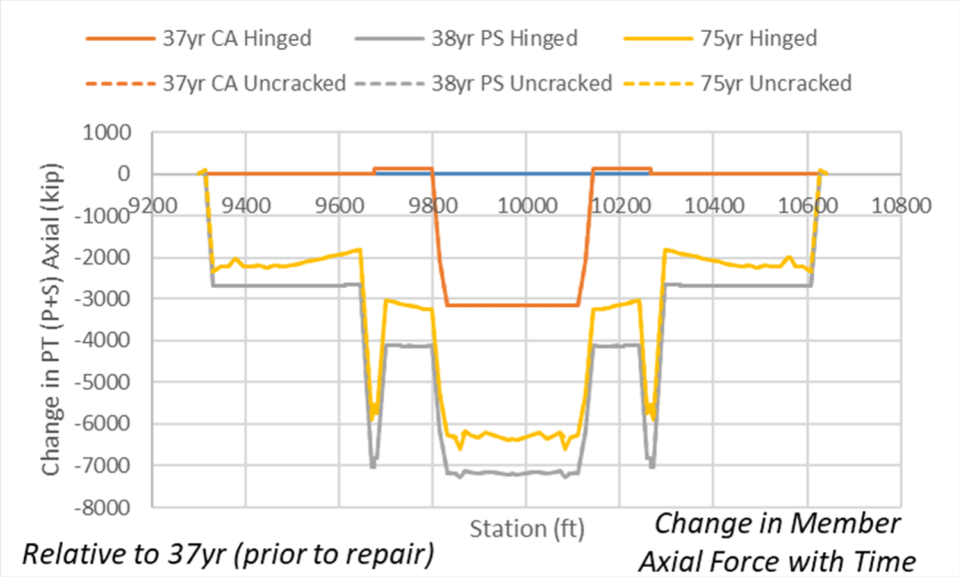


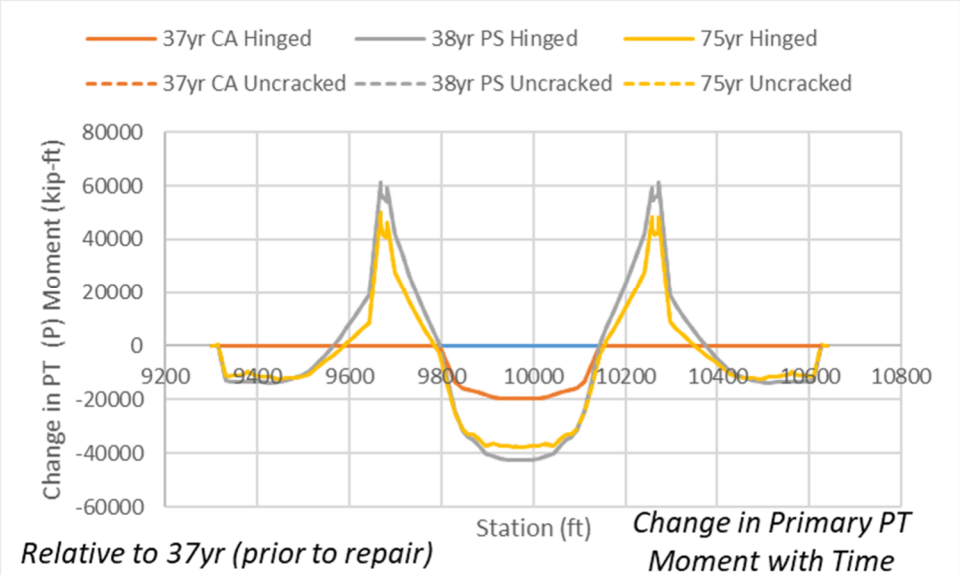

37yr Pre-Retrofit
37yr Crack Arrest
38yr PS Mainspan
38yr PS P16
38yr PS P17
Change in PT (P) Moment (kip-ft)
80000
60000
40000
20000
0
-20000
-40000
-60000
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
Change in Primary PT Moment during Stressing Sequence
Relative to 37yr (prior to repair)
Hinged Model Demands

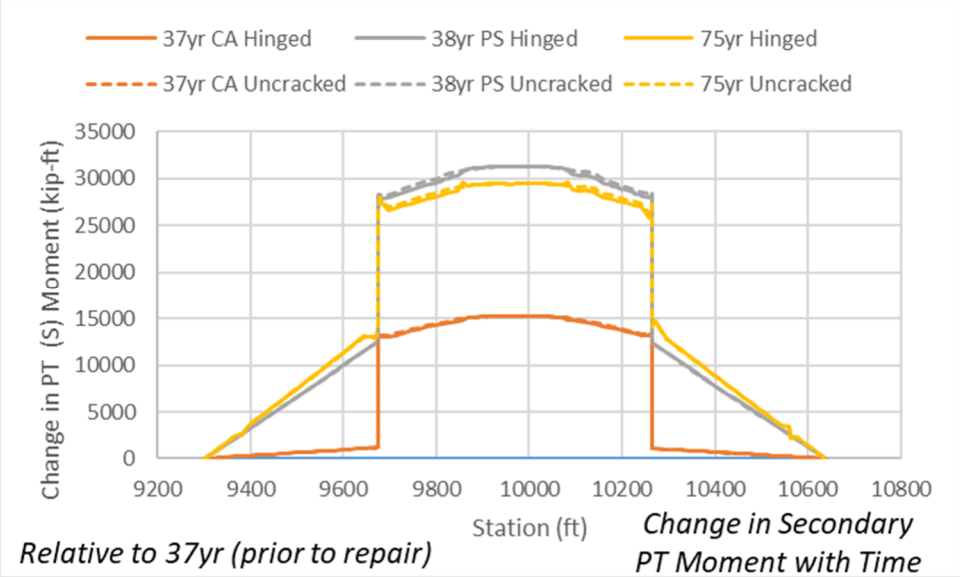

37yr Pre-Retrofit
37yr Crack Arrest
38yr PS Mainspan
38yr PS P16
38yr PS P17
Change in PT (S) Moment (kip-ft)
35000
30000
25000
20000
15000
10000
5000
0
-5000
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
Change in Secondary PT Moment during Stressing Sequence
Relative to 37yr (prior to repair)
Hinged Model Prior to PT

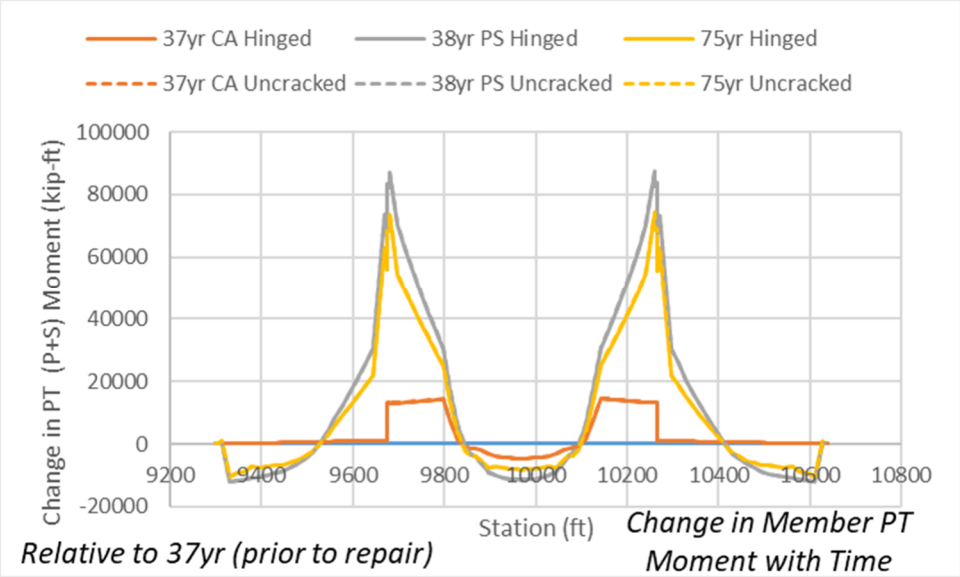


The following compares the member PT load effects in the Hinged and Uncracked Models.




37yr CA Hinged
38yr PS Hinged
75yr Hinged
37yr CA Uncracked
38yr PS Uncracked
75yr Uncracked
Change in PT (P) Moment (kip-ft)
80000
60000
40000
20000
0
-20000
-40000
-60000
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
Relative to 37yr (prior to repair)
Change in Primary PT Moment with Time

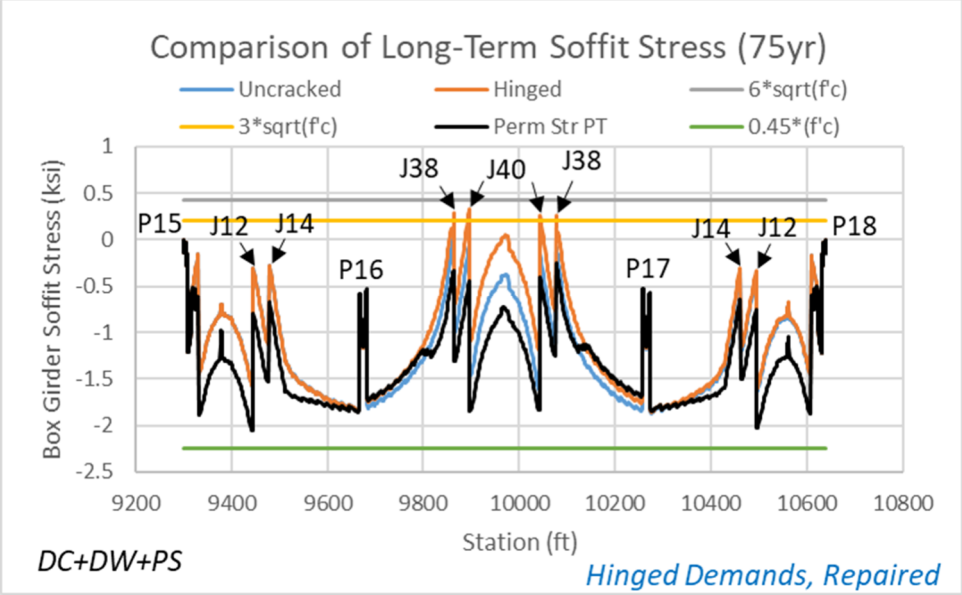

37yr CA Hinged
38yr PS Hinged
75yr Hinged
37yr CA Uncracked
38yr PS Uncracked
75yr Uncracked
Change in PT (S) Moment (kip-ft)
35000
30000
25000
20000
15000
10000
5000
0
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
Relative to 37yr (prior to repair)
Change in Secondary PT Moment with Time



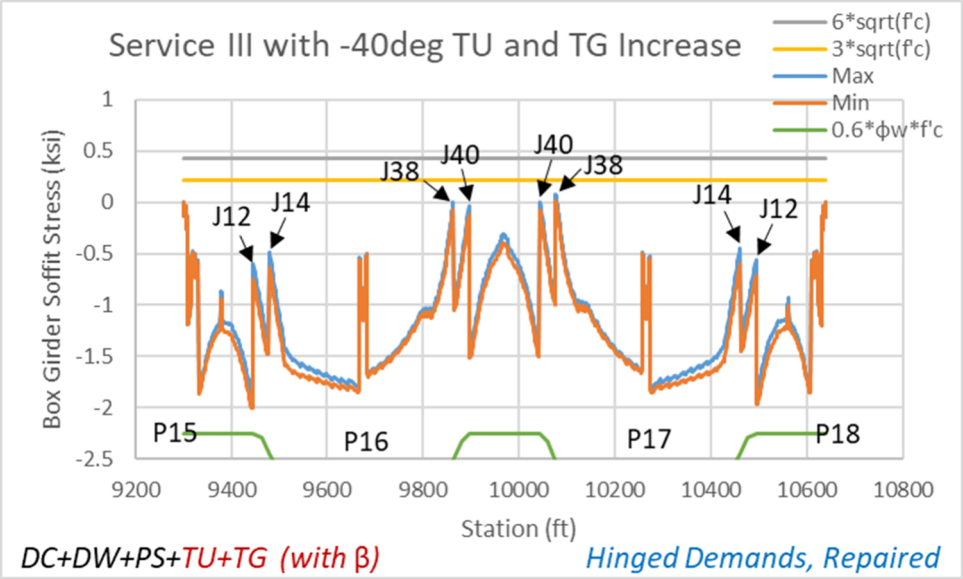
The same results were obtained.

Service III – Analysis with the Hinged Model Demands Locked in by the Retrofit

In the following figures, stress limits are calculated using the original 5ksi design concrete strength.

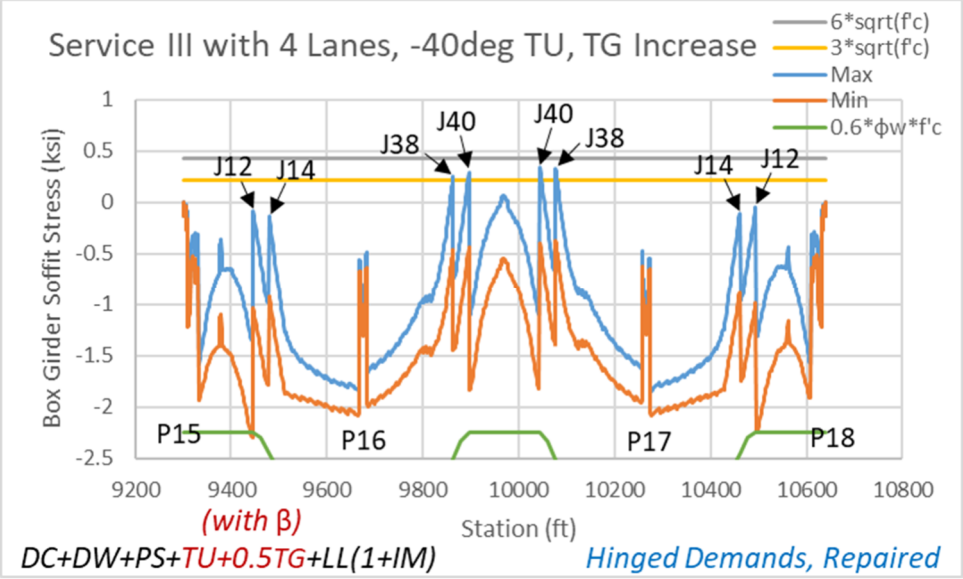


Applicable long-term compression limit of 0.45f'c = 2.25ksi satisfied for the soffit. Zero tension observed. Hinged and Uncracked model results prior to any retrofits are shown for comparison only.



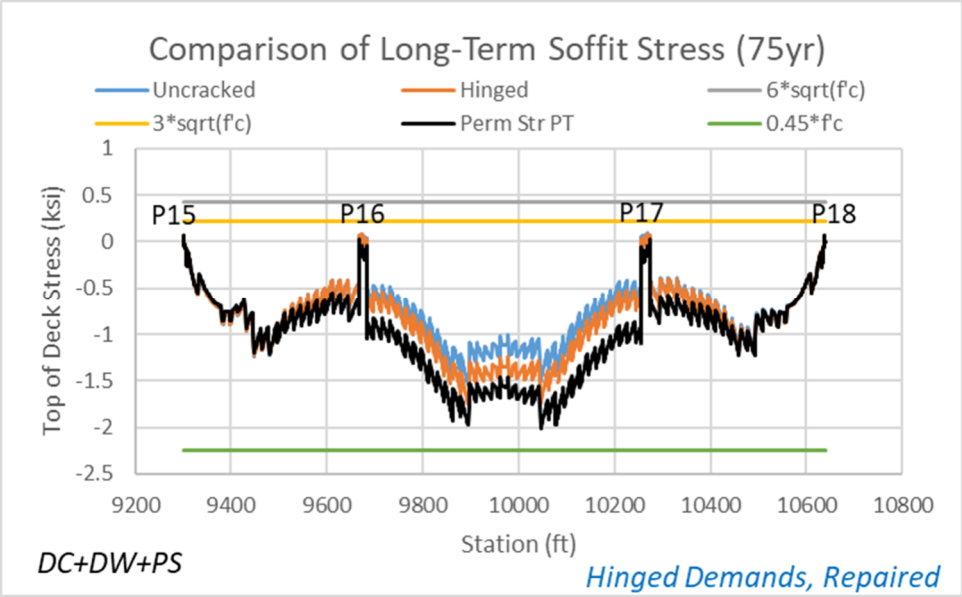
Service III, no LL

Minimal soffit tension stress at Joints 38 and 40. Results for -40deg TU and TG Deck Increase use the β-factor reductions in load factors per the thermal study. The load factors conservatively haven't yet been revised to reflect the most recent findings based on hourly thermal data. This thermal case maximizes mainspan soffit tension and deck compression. Note that uniform thermal (TU) effects have increased due to consideration of friction developed at the supports of Piers 15 and 18.

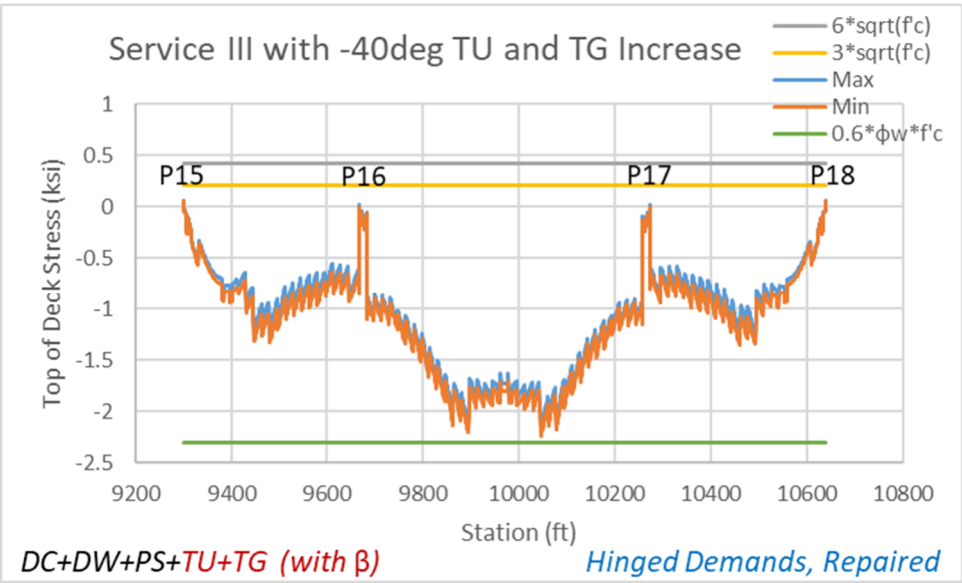


Service III, 4 Lanes

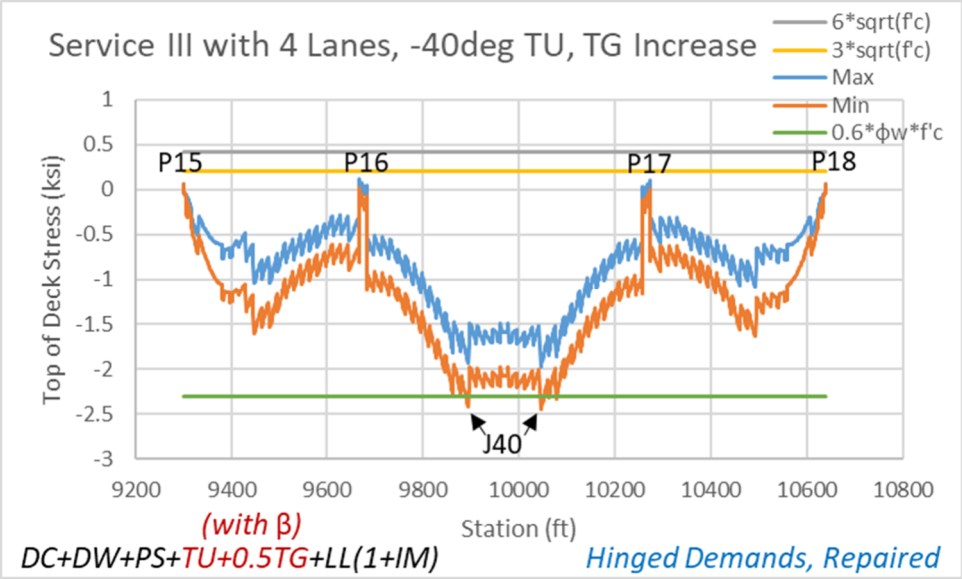
Soffit tension stresses below 6√f'c occur at Joints 38 and 40 in an analysis with the redistributed hinged demands locked in the by the retrofit PT. Main span permanent strengthening PT includes one (31-0.6) and one (19-0.6) per web. Side span compression will be investigated subsequently, since the Service I demands govern.



Applicable long-term compression limit of 0.45f'c = 2.25ksi satisfied for the deck. Hinged and Uncracked model results prior to any retrofits are shown for comparison only.

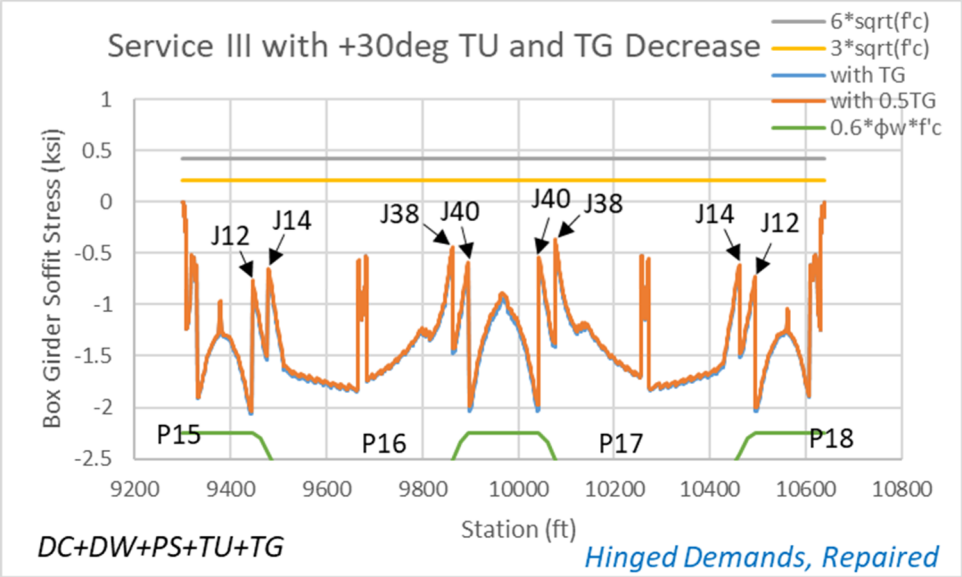


Service III, no LL



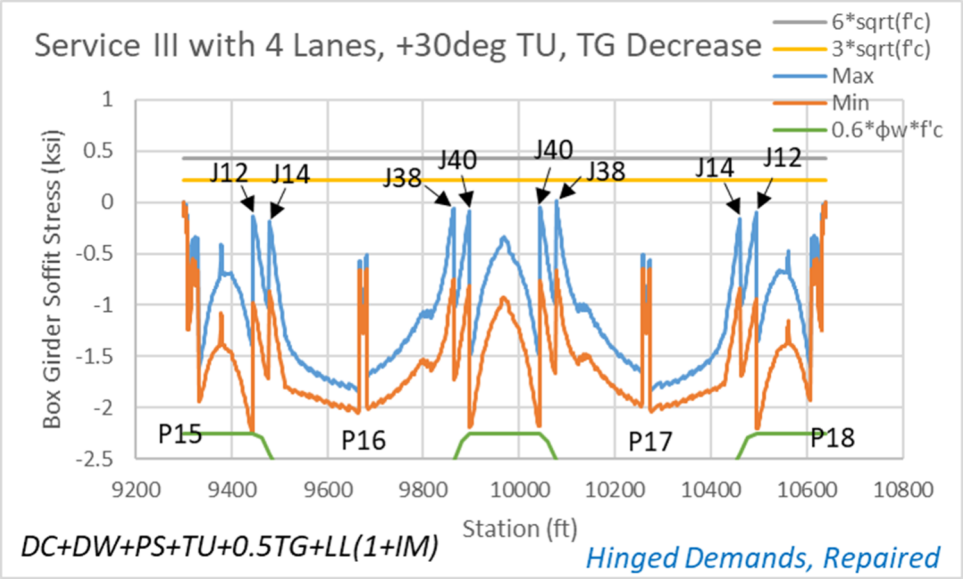
Service III, 4 Lanes

The applicable compressive limit is 0.6(ϕw)f'c, but note the higher compressive stresses of Service I govern. For the Deck, the slenderness based on average thickness λw = 24.23 governs over the slenderness for the constant thickness region λw = 16 (see C.5.9.2.3.2a-1). Since λw > 15, ϕw calculated as 0.769. Therefore, the applicable compressive limit is 0.6(0.769)5 = 2.31ksi. The deck is overstressed at Joints 40 with 5ksi concrete, prior to consideration of the higher Service I stresses.



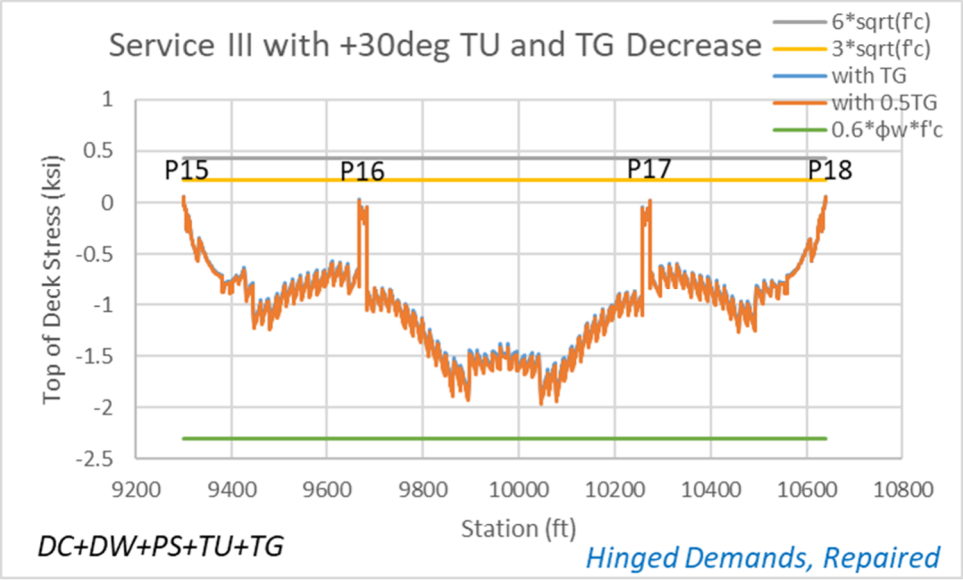
Service III, no LL

Thermal case +30deg TU and TG Deck Decrease maximizes soffit compression in the mainspan region.

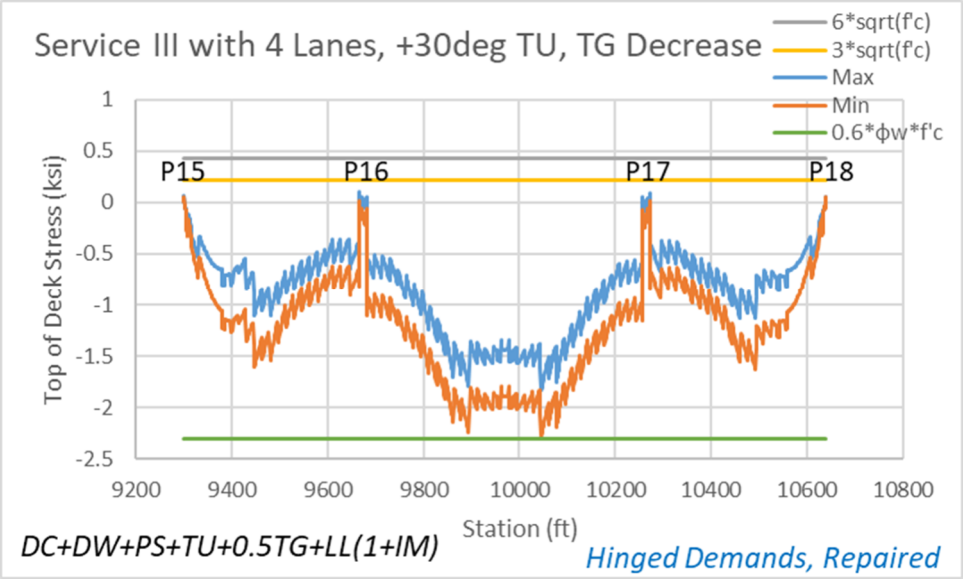


Service III, 4 lanes

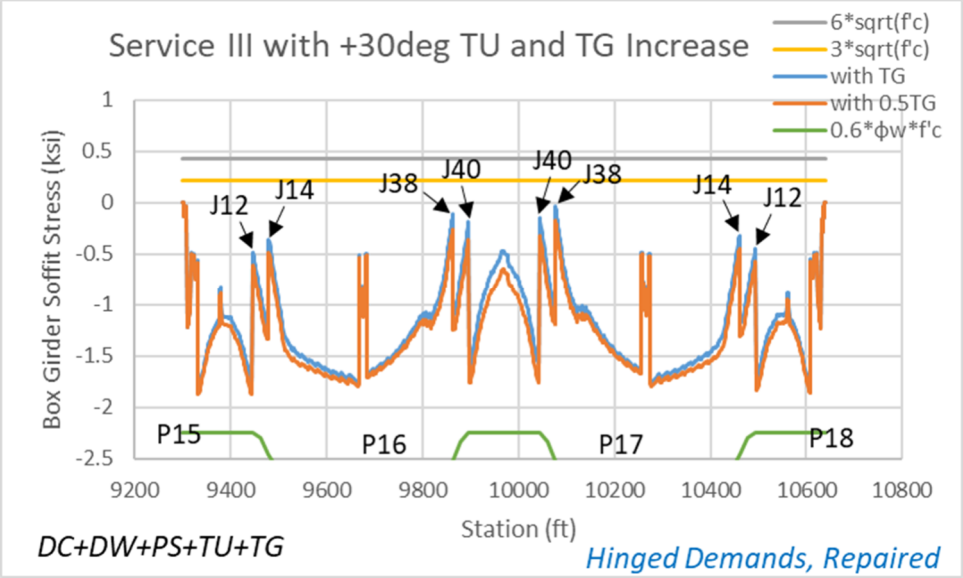
For the Joint 40 Soffit, the slenderness based on average thickness λw = 26.7 governs over the slenderness for the constant thickness region λw = 22.24 (see C.5.9.2.3.2a-1). Since λw > 25, ϕw is 0.75. Therefore, the applicable compressive limit is 0.6(0.75)5 = 2.25ksi.



Service III, no LL

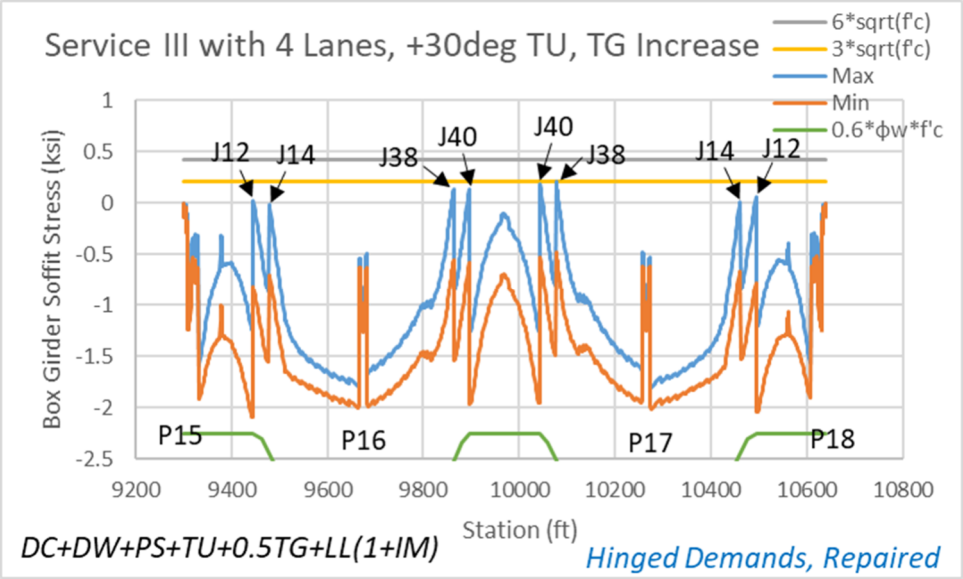


Service III, 4 lanes



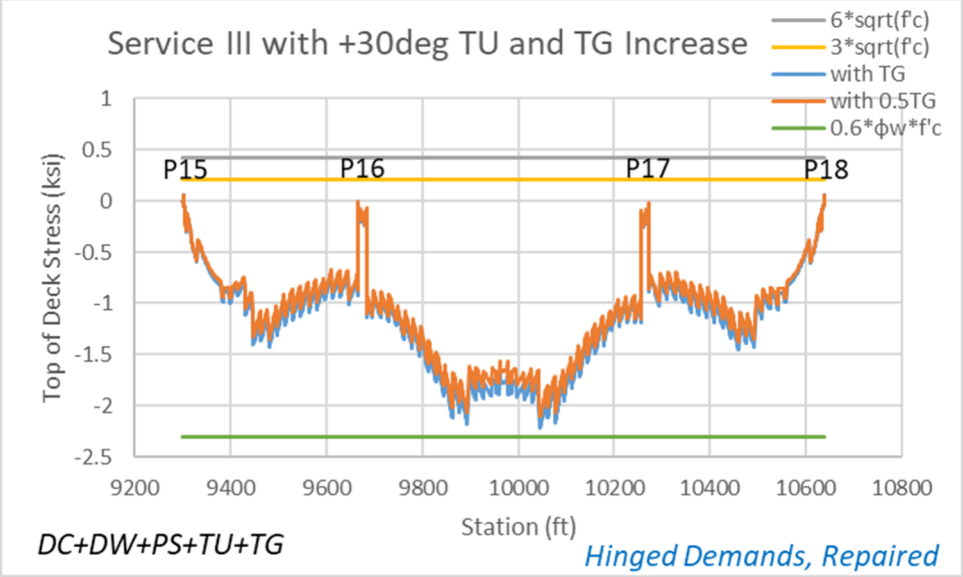
Service III, no LL

Thermal case +30deg TU and TG Deck Increase governs for soffit tension and deck compression in the side spans.

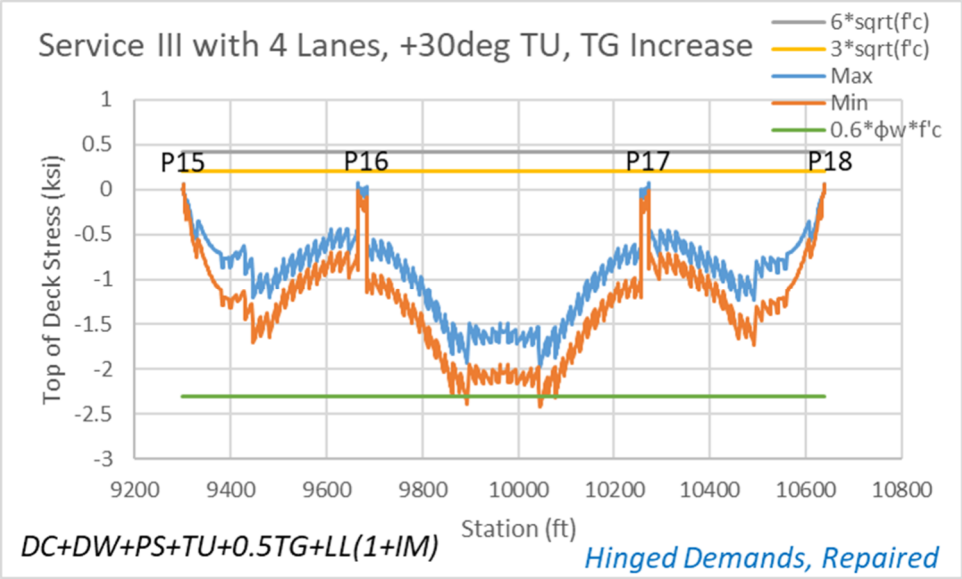


Service III, 4 lanes

Soffit tension stresses below 3√f'c occur in the side spans. There is some room to decrease the side span PT; however, this needs to be verified after including the effects of potential hinging and redistribution. Results would indicate that this behavior is less severe than at Joints 38. Reducing the side span PT would help with compressive stress exceedances at Joint 12.

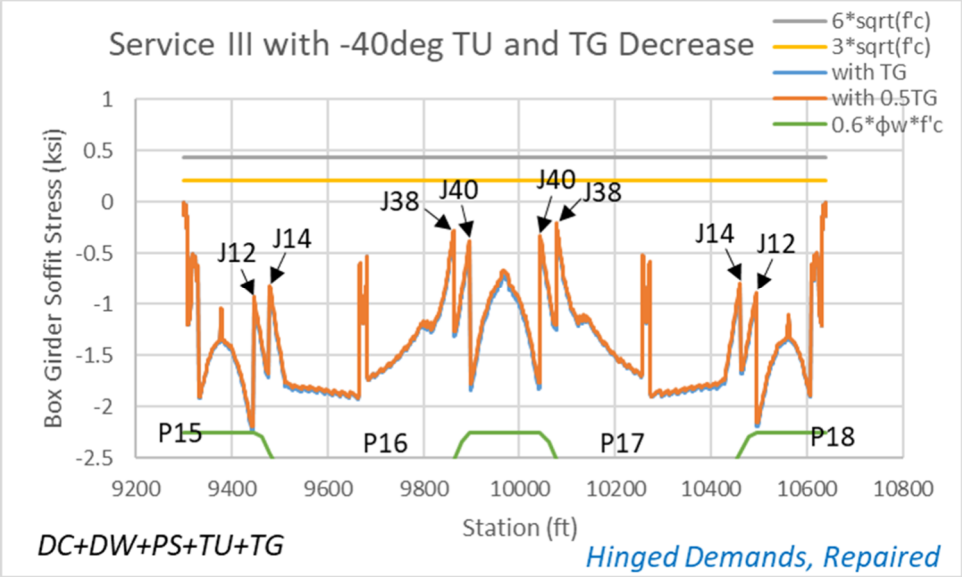


Service III, no LL



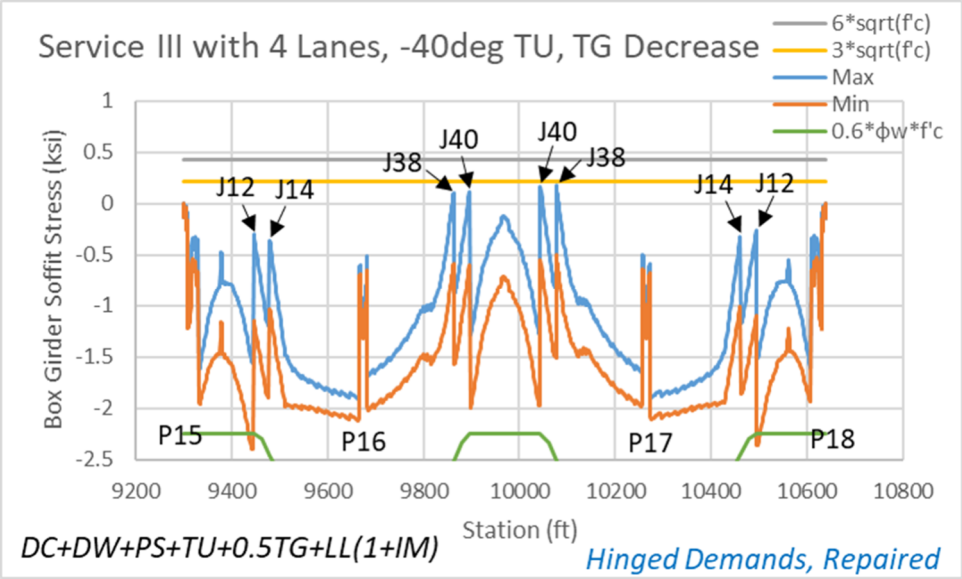
Service III, 4 lanes

Again, note the exceedance of the deck compressive stress limit near Joints 40.



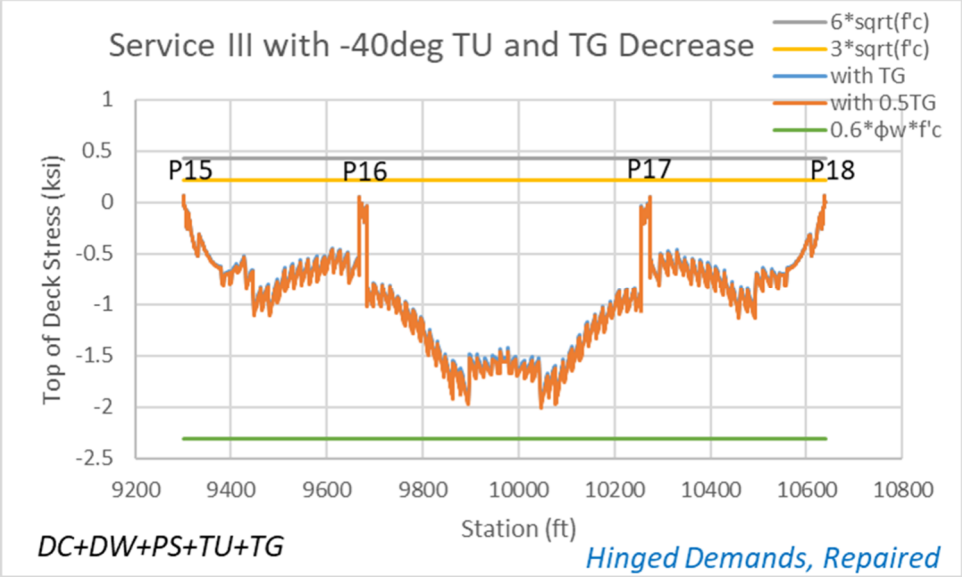
Service III, no LL

Thermal case -40deg TU and TG Deck Decrease maximizes soffit compression in the side spans.

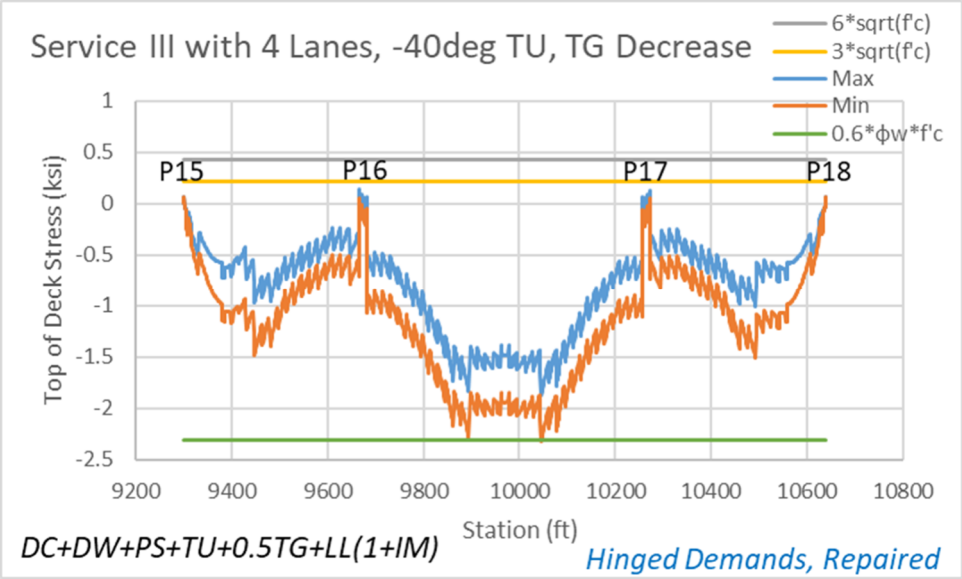


Service III, 4 lanes

For the Joint 12 Soffit, the slenderness based on average thickness λw = 26.78 governs over the slenderness for the constant thickness region λw = 22.31 (see C.5.9.2.3.2a-1). Since λw > 25, ϕw is 0.75. Therefore, the applicable compressive limit is 0.6(0.75)5 = 2.25ksi, which is exceeded at Joint 12 prior to considering the larger Service I demands. As previously discussed, side span soffit tension stresses at Joints 12 and 14 are well below 3√f'c, so reductions in the side span PT could help.



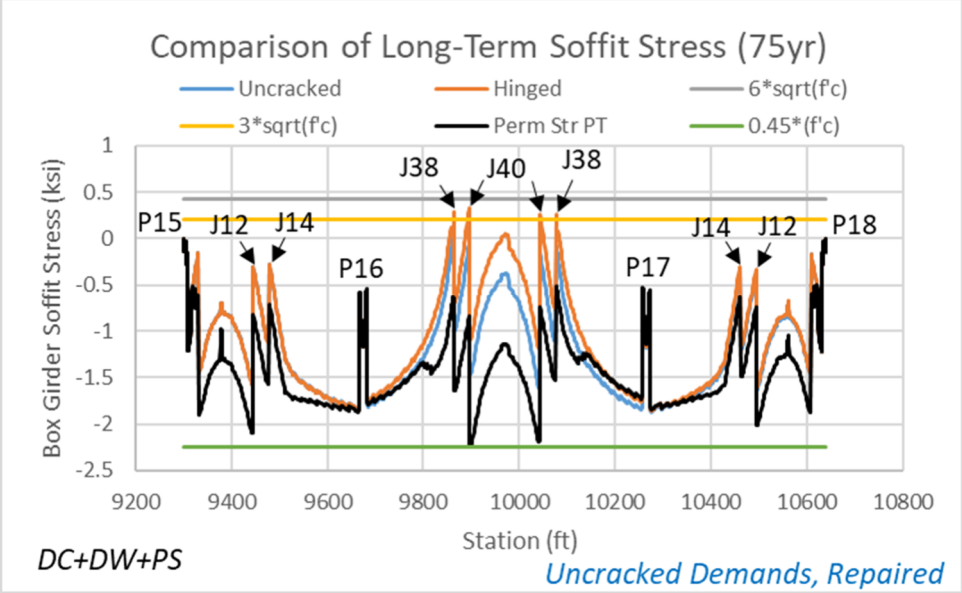
Service III, no LL



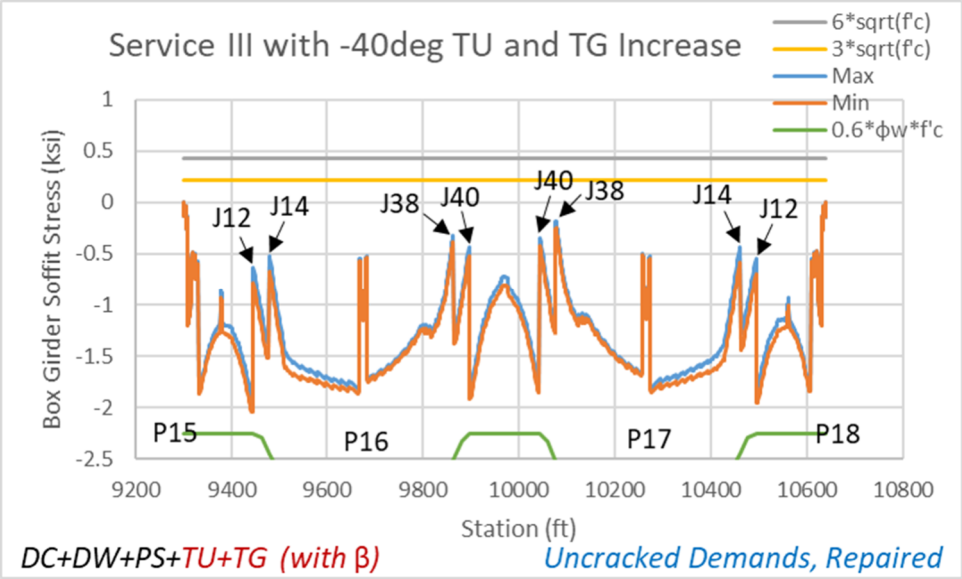
Service III, 4 lanes

Service III – Analysis with the Uncracked Model Demands

The following describes results from an analysis using the Uncracked Model. Results are shown for comparison purposes, since the demands are likely between the Uncracked and Hinged Model results.

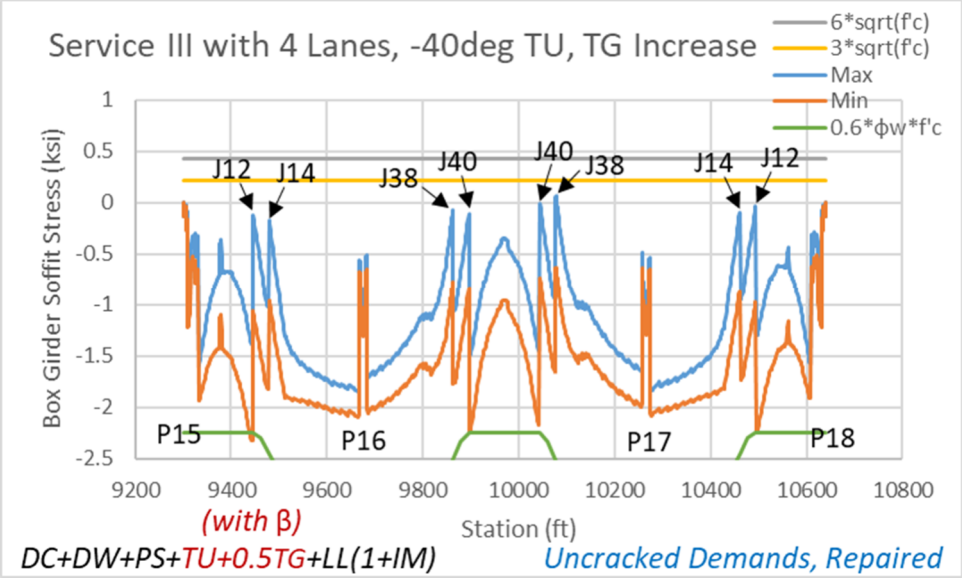


The applicable long-term compression limit of 0.45f'c = 2.25ksi just exceeds the maximum soffit compression stress at Joint 40. If the mainspan PT is governed by soffit tension in the Hinged analysis, then soffit compression demands in the Uncracked Model are increased, due to the lower positive moments. Note that if seismic governs, there is no room for additional PT without creating more significant issues for compressive stresses. Hinged and Uncracked model results prior to any retrofits are shown for comparison only.



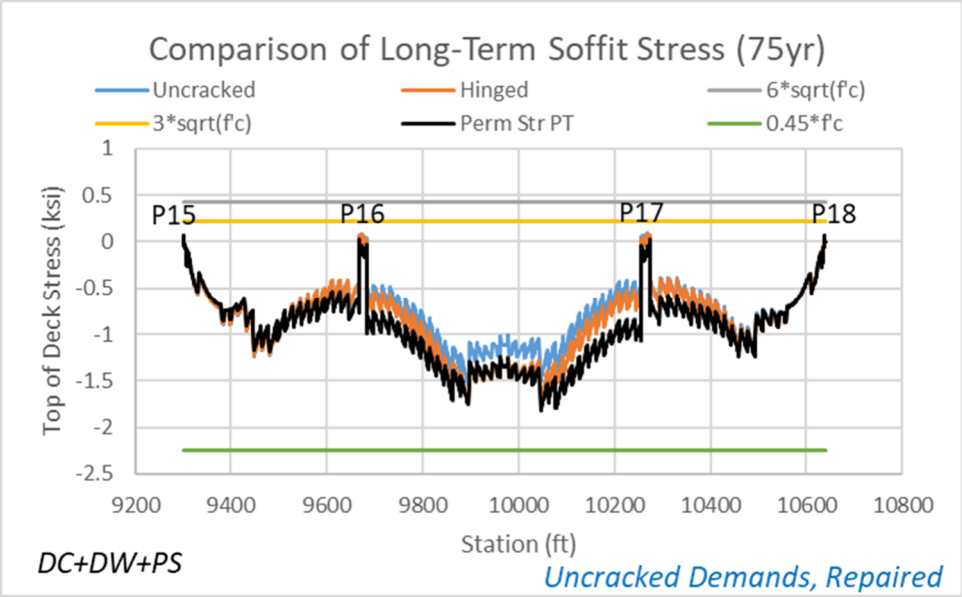
Service III, no LL

Soffit remains in compression without live load.

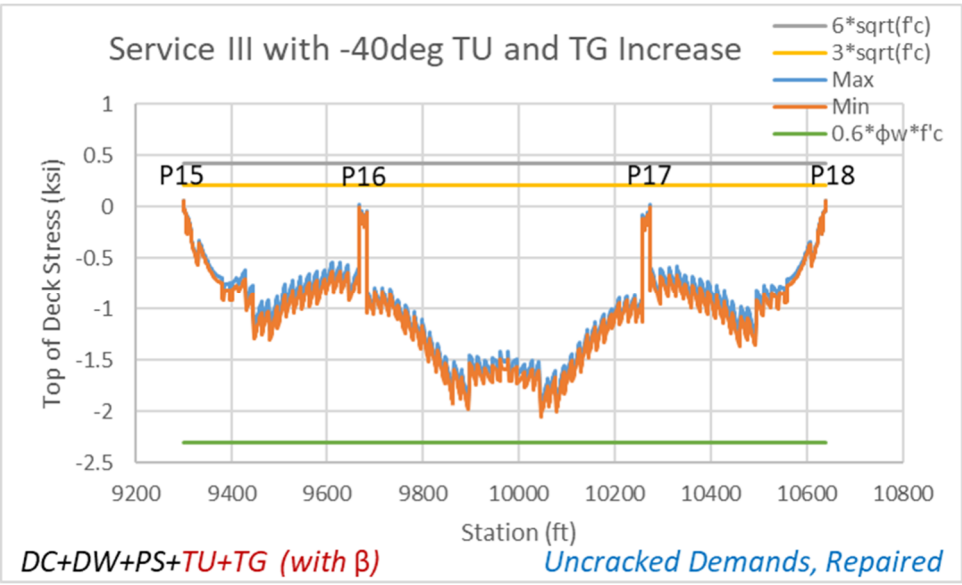


Service III, 4 Lanes

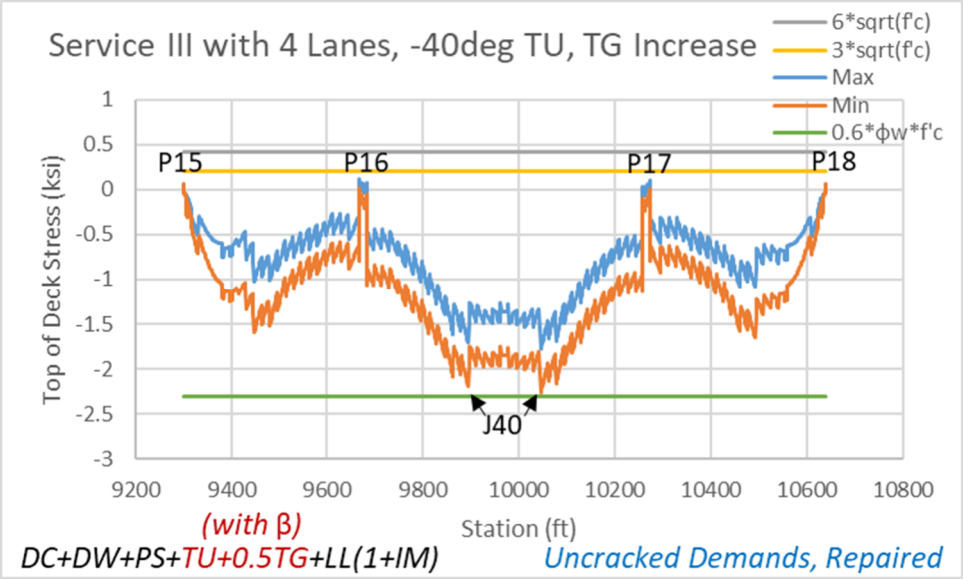
Main span soffit tension stresses are well below 3√f'c with the one (31-0.6) and one (19-0.6) per web. Thermal case -40deg TU and TG Deck Increase maximizes soffit tension in the mainspan region.



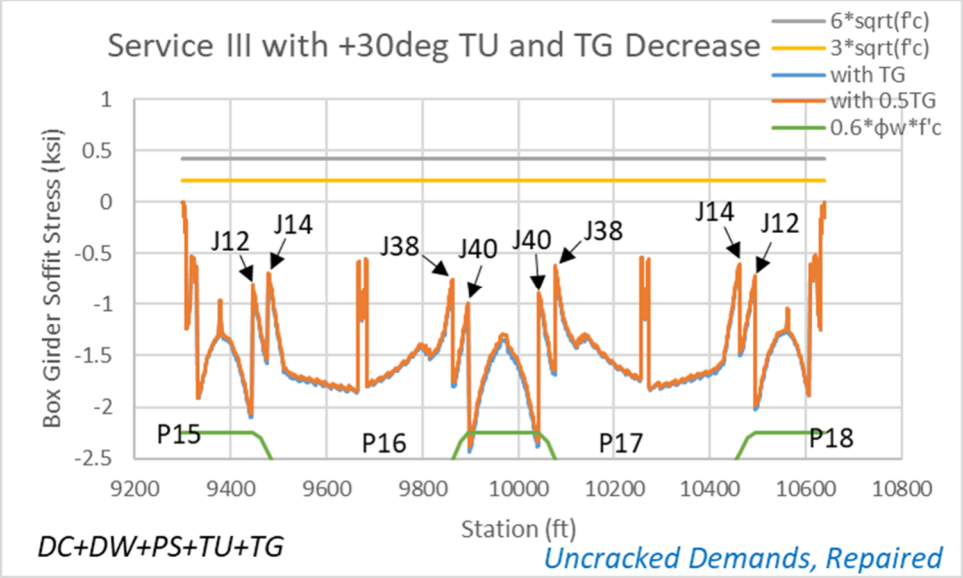
Applicable long-term compression limit of 0.45f'c = 2.25ksi satisfied for the deck. Hinged and Uncracked model results prior to any retrofits are shown for comparison only.



Service III, no LL

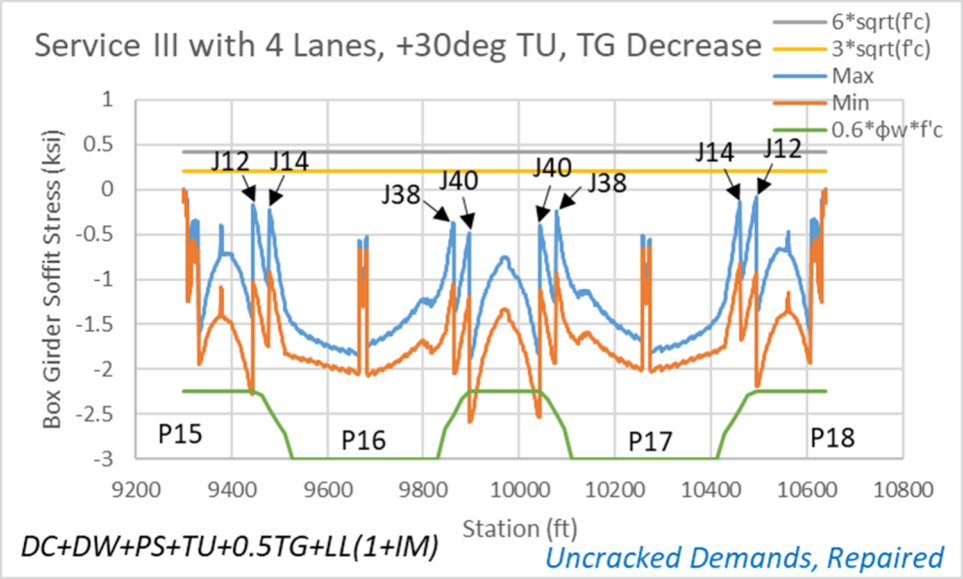


Service III, 4 Lanes



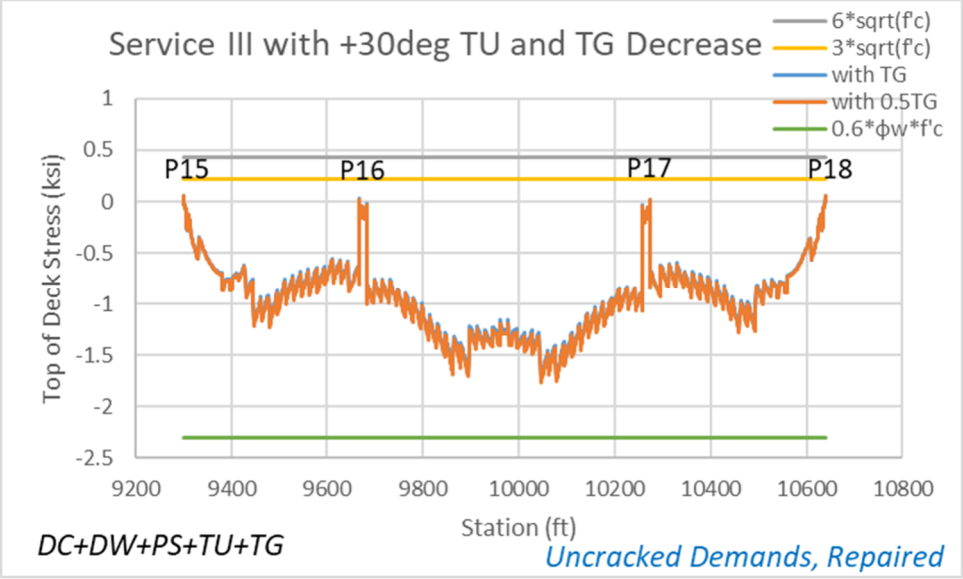
Service III, no LL

Thermal case +30deg TU and TG Deck Decrease maximizes soffit compression in the mainspan region.

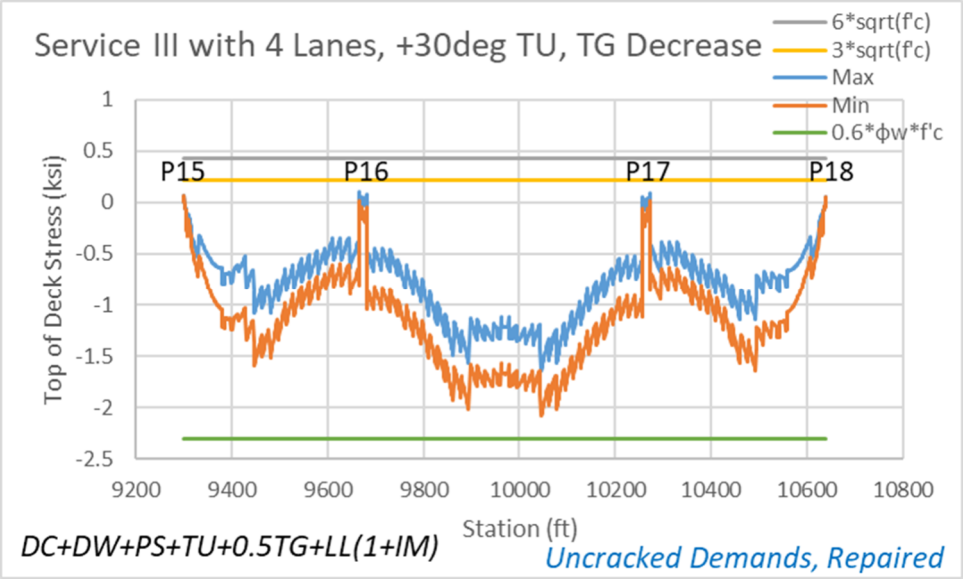


Service III, 4 lanes

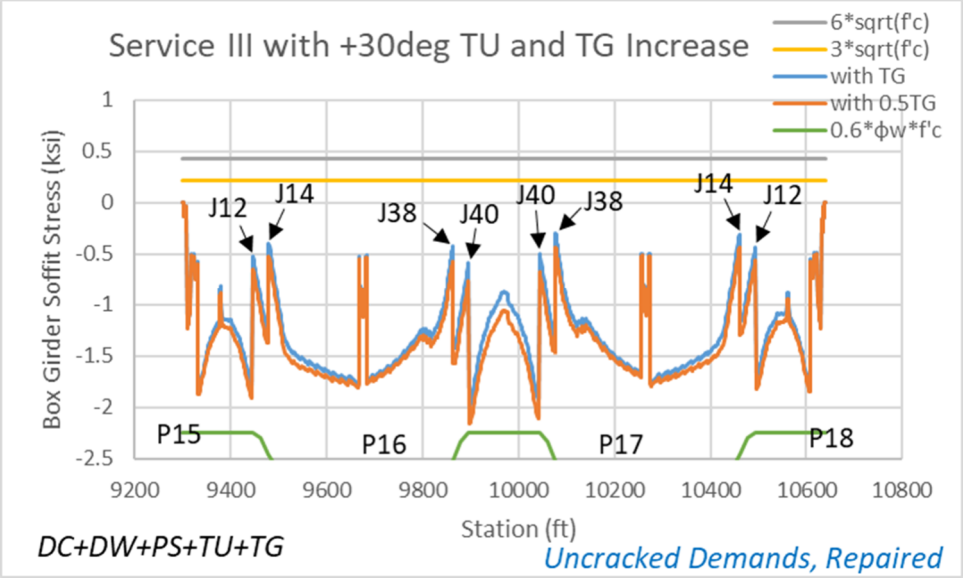
For the Joint 40 Soffit, the slenderness based on average thickness λw = 26.7 governs over the slenderness for the constant thickness region λw = 22.24 (see C.5.9.2.3.2a-1). Since λw > 25, ϕw is 0.75. Therefore, the applicable compressive limit is 0.6(0.75)5 = 2.25ksi. The soffit at Joint 40 is overstressed with 5ksi concrete, even before considering the higher Service I demands.



Service III, no LL

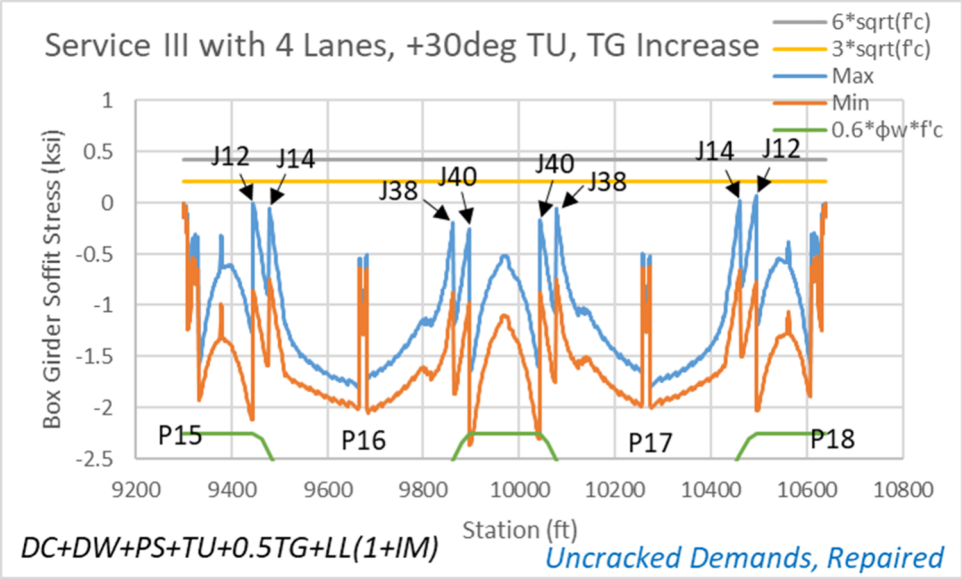


Service III, 4 lanes



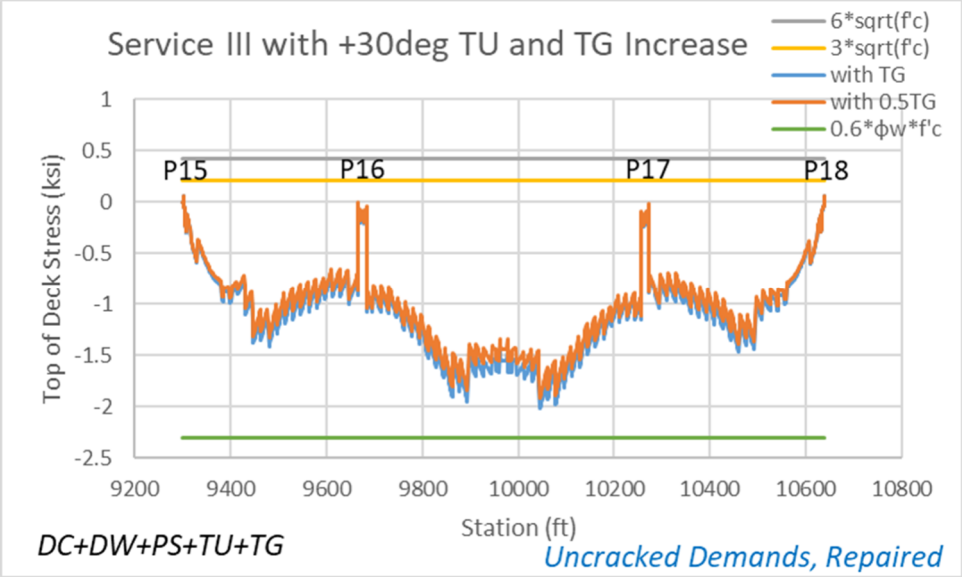
Service III, no LL

Thermal case +30deg TU and TG Deck Increase governs for soffit tension and deck compression in the side spans.

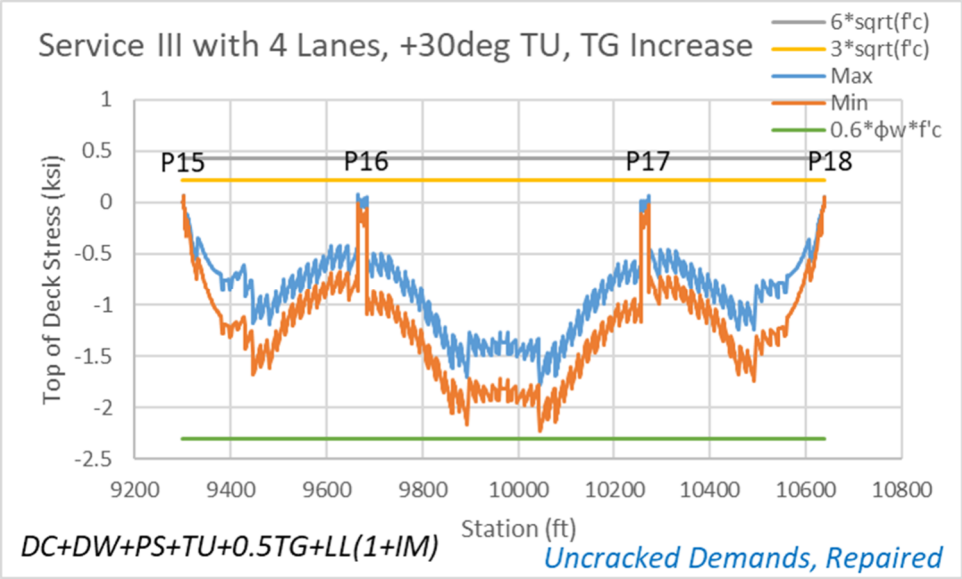


Service III, 4 lanes

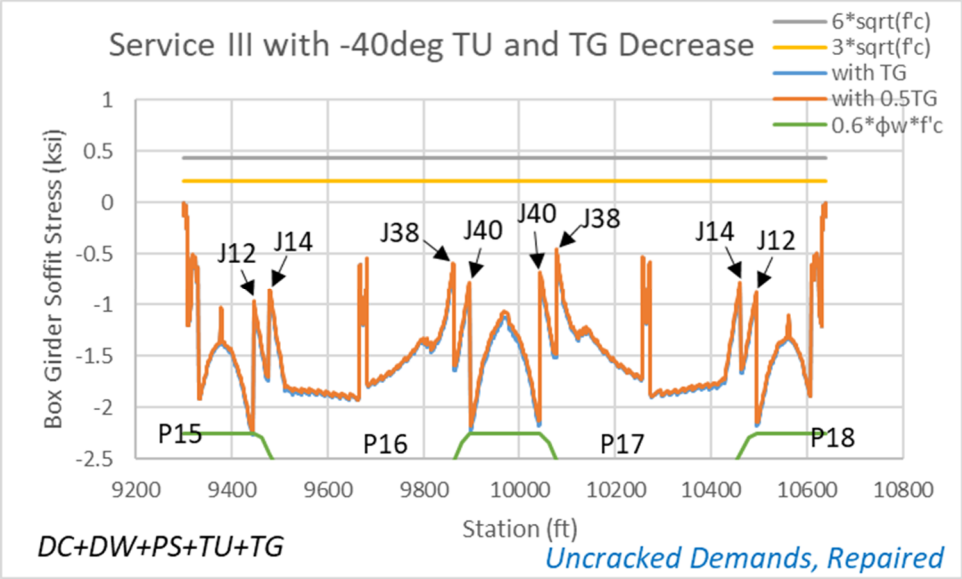
Soffit tension stresses below 3√f'c in the side spans with 4 lanes loaded.



Service III, no LL

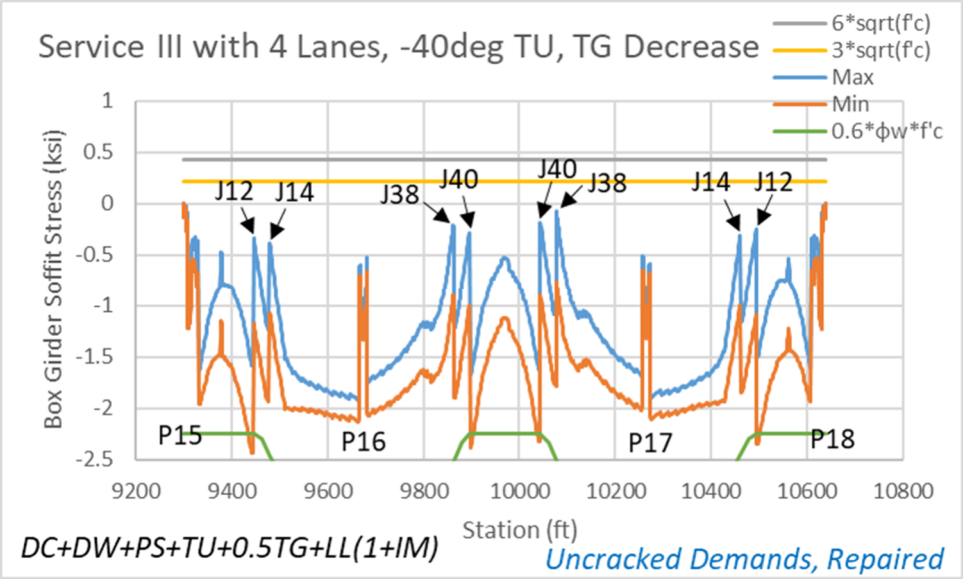


Service III, 4 lanes



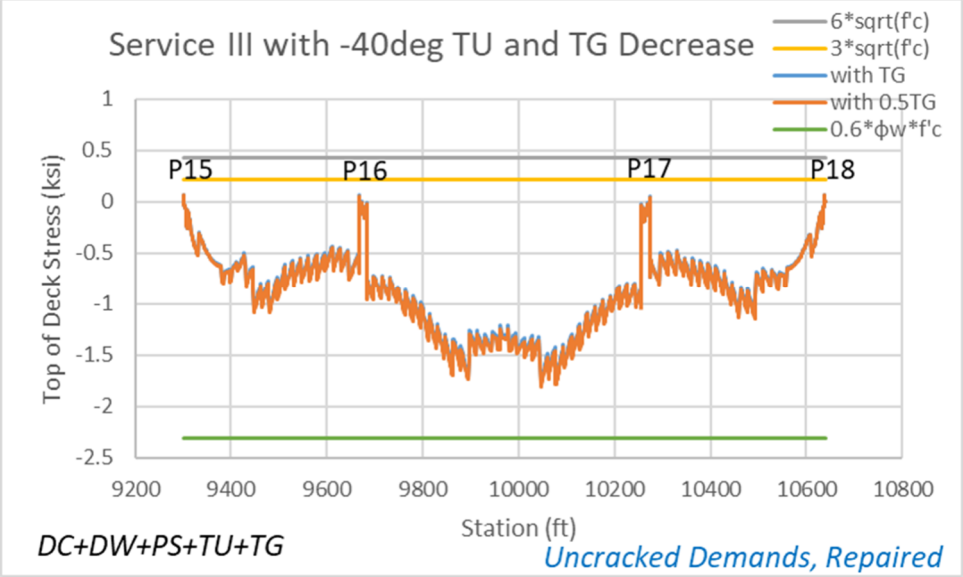
Service III, no LL

Thermal case -40deg TU and TG Deck Decrease maximizes soffit compression in the side spans.

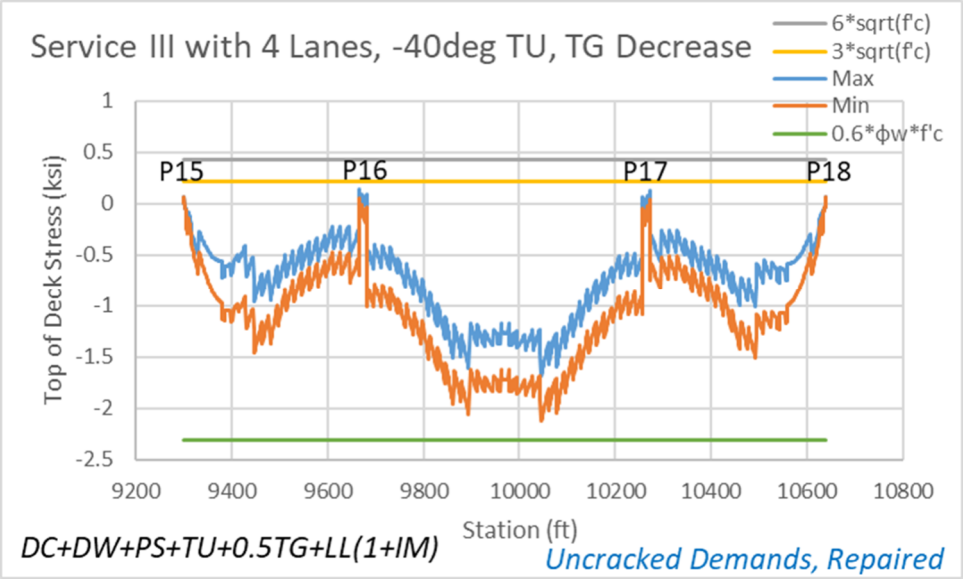


Service III, 4 lanes

For the Joint 12 soffit, the slenderness based on average thickness λw = 26.78 governs over the slenderness for the constant thickness region λw = 22.31 (see C.5.9.2.3.2a-1). Since λw > 25, ϕw is 0.75. Therefore, the applicable compressive limit is 0.6(0.75)5 = 2.25ksi. The soffit is overstressed at Joint 12 with 5ksi concrete, even before considering the higher Service I demands.



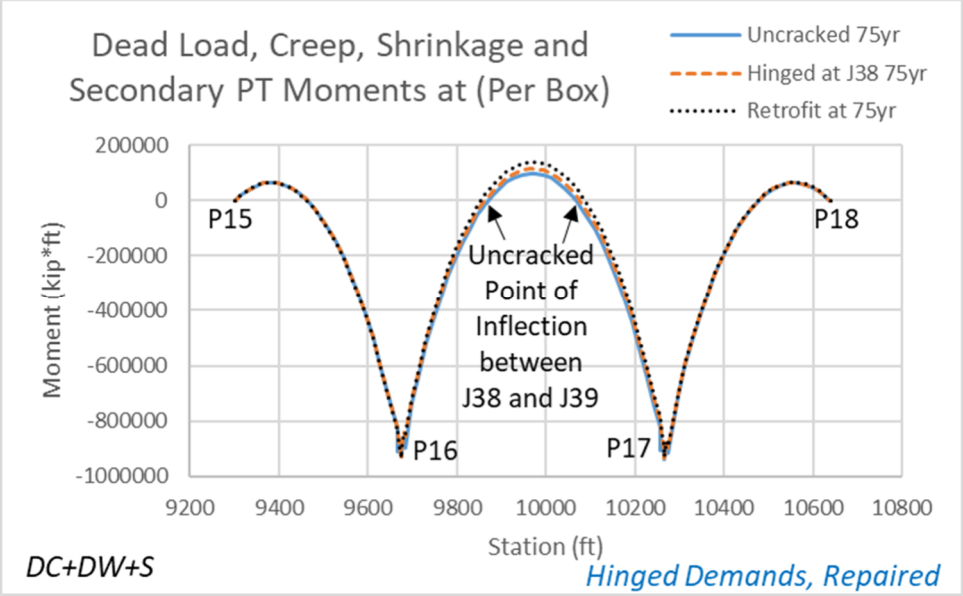
Service III, no LL

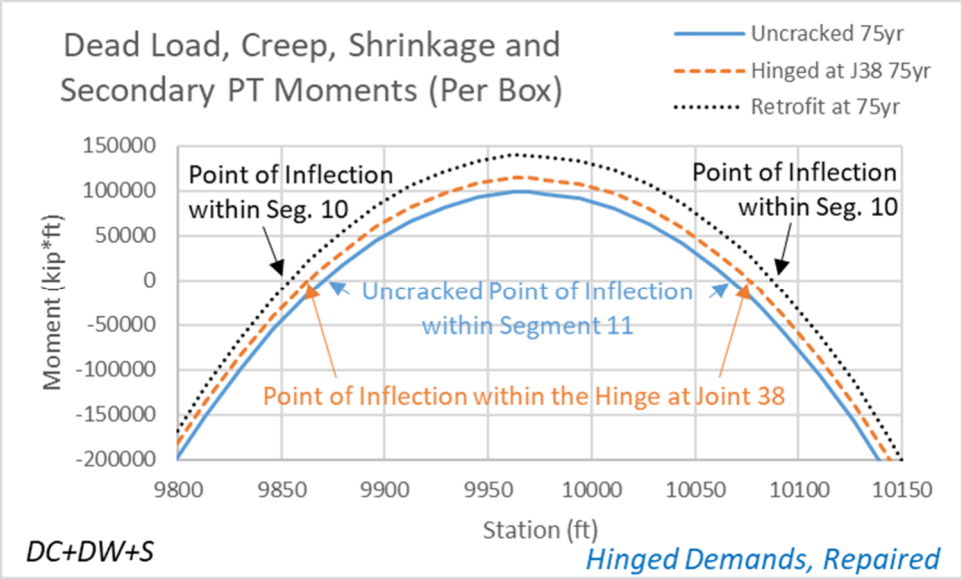


Service III, 4 lanes

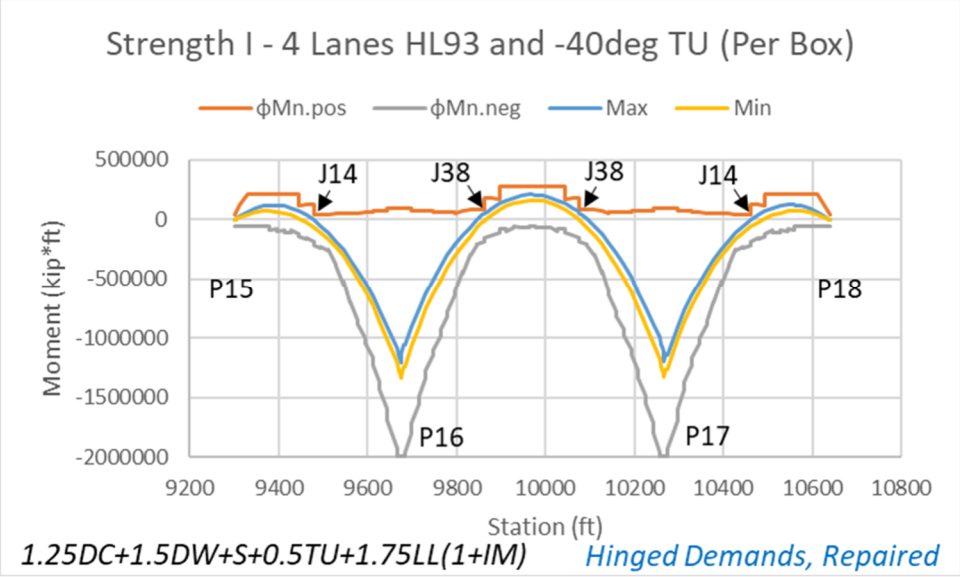
Strength I Moments – Analysis with the Hinged Model Demands Locked in by the Retrofit

Note the previous Uncracked and Hinged Models for the existing structure did not include the locked-in forces due to the midspan closure form travelers that lead to slight asymmetry in the response. Hinged and Uncracked model results prior to any retrofits are shown for comparison only in the figure below.

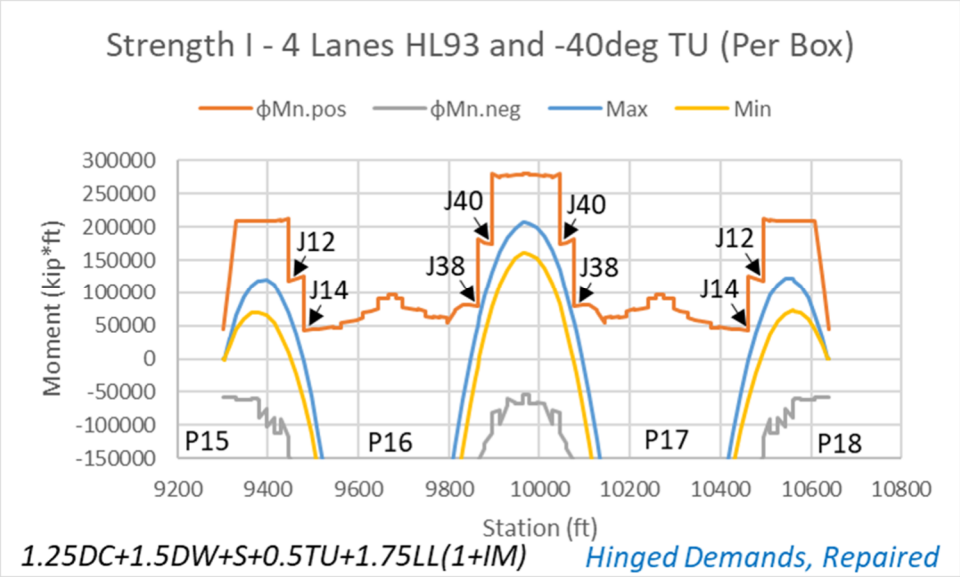




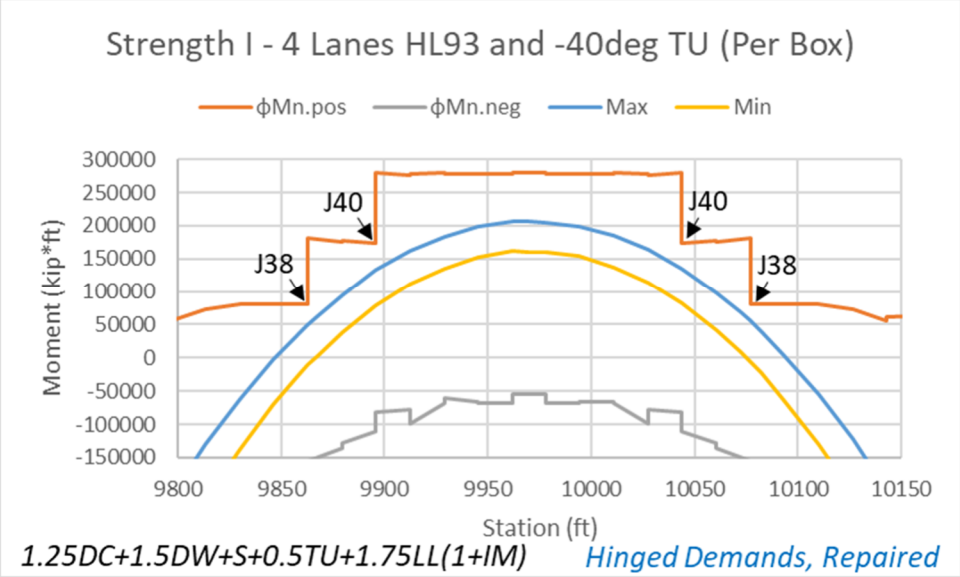
Moment-curvature analysis was performed at each joint along the bridge. Strain compatibility was assumed for the existing bonded PT; however, the external retrofit PT was conservatively assumed to maintain a constant force at the effective stress, f_{pe}, at 75yrs after all losses from the model results. For each thermal case considered and maximum/minimum DC load factor, the provided moment capacity exceeded the factored Strength I demands as shown in the following.



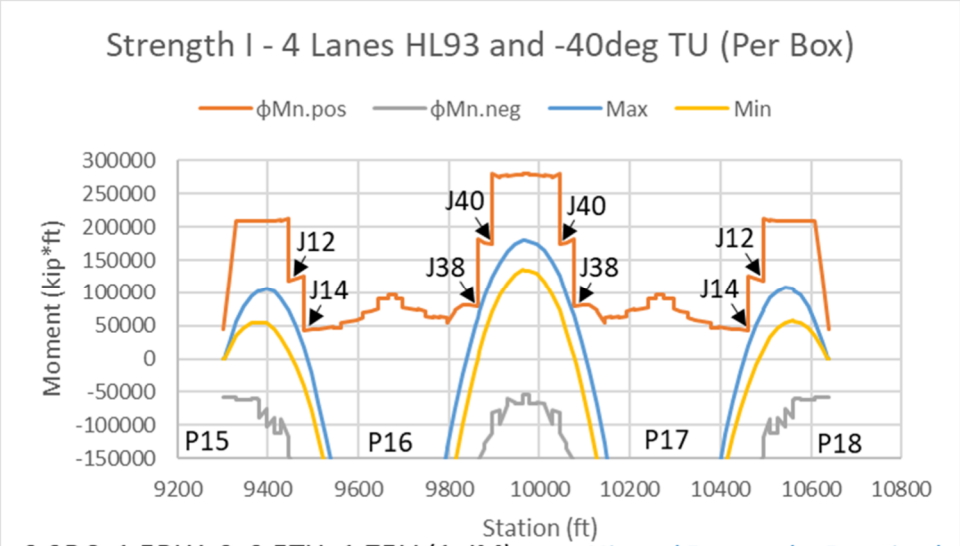
-TU with max DL factor



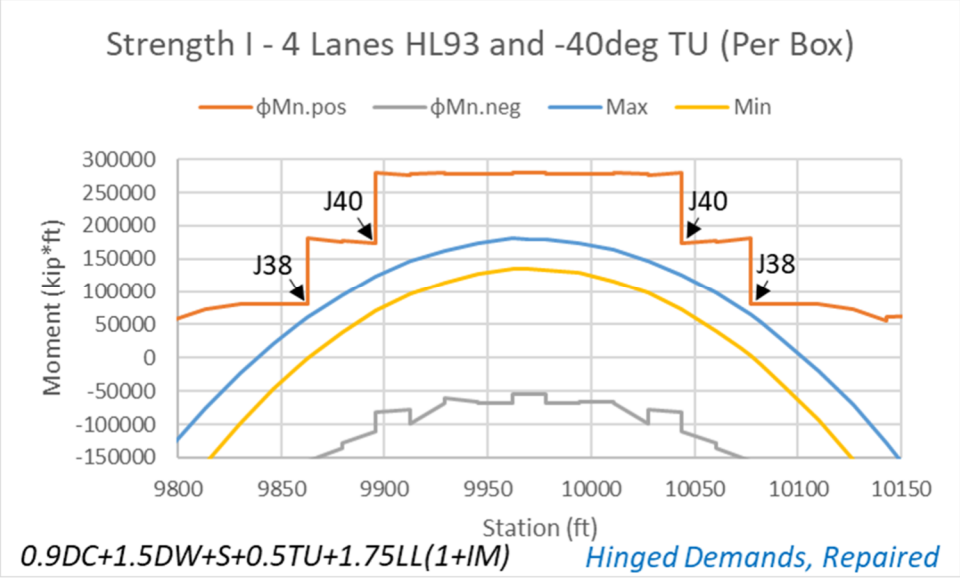
1.25DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Hinged Demands, Repaired* -TU with max DL factor



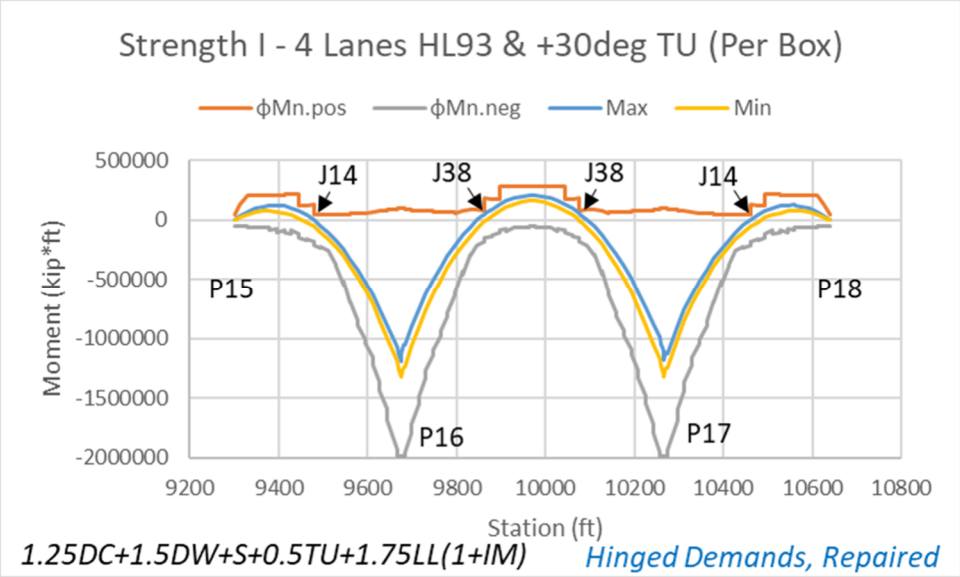
1.25DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Hinged Demands, Repaired* -TU with max DL factor



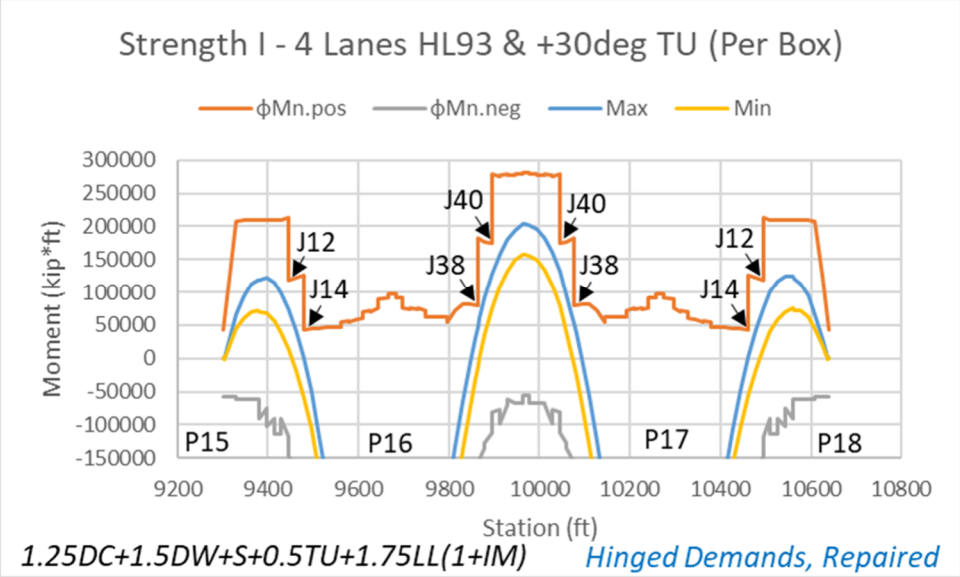
0.9DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Hinged Demands, Repaired* -TU with min DL factor



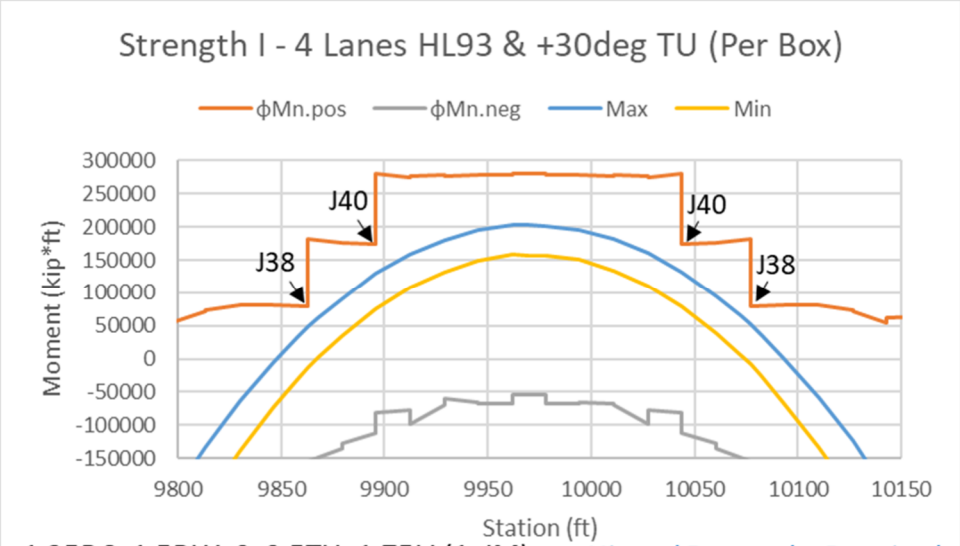
0.9DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Hinged Demands, Repaired* -TU with min DL factor



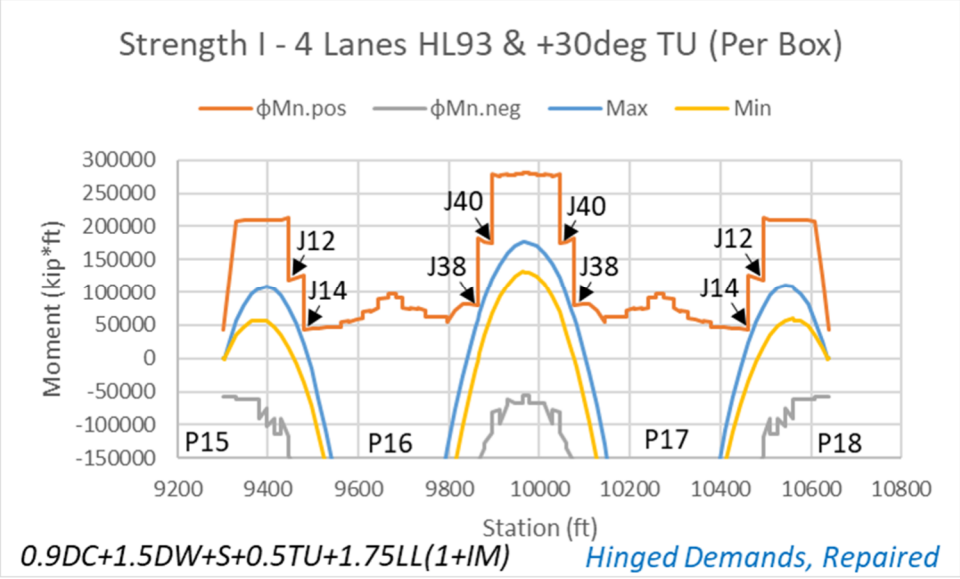
1.25DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Hinged Demands, Repaired* +TU with max DL factor



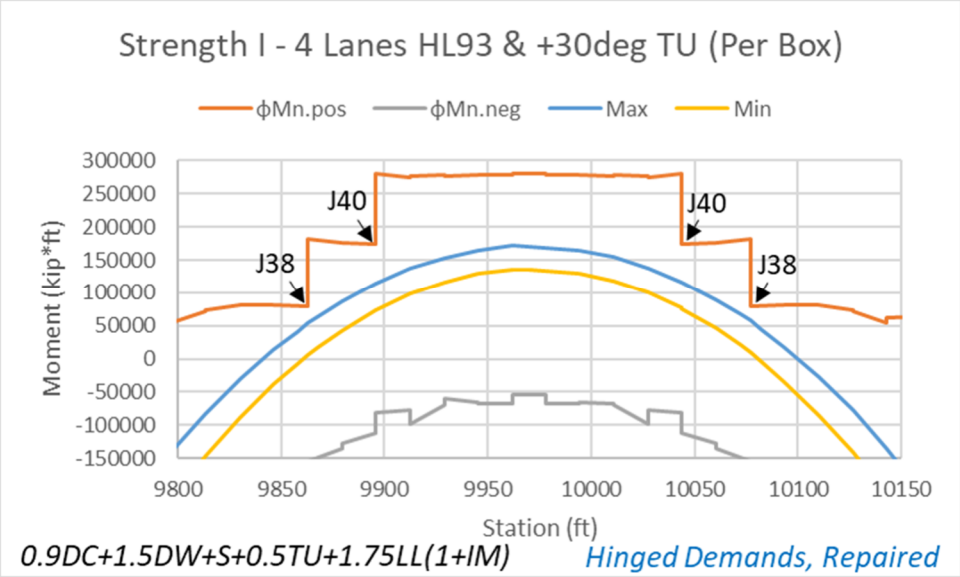
1.25DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Hinged Demands, Repaired* +TU with max DL factor



1.25DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Hinged Demands, Repaired* +TU with max DL factor



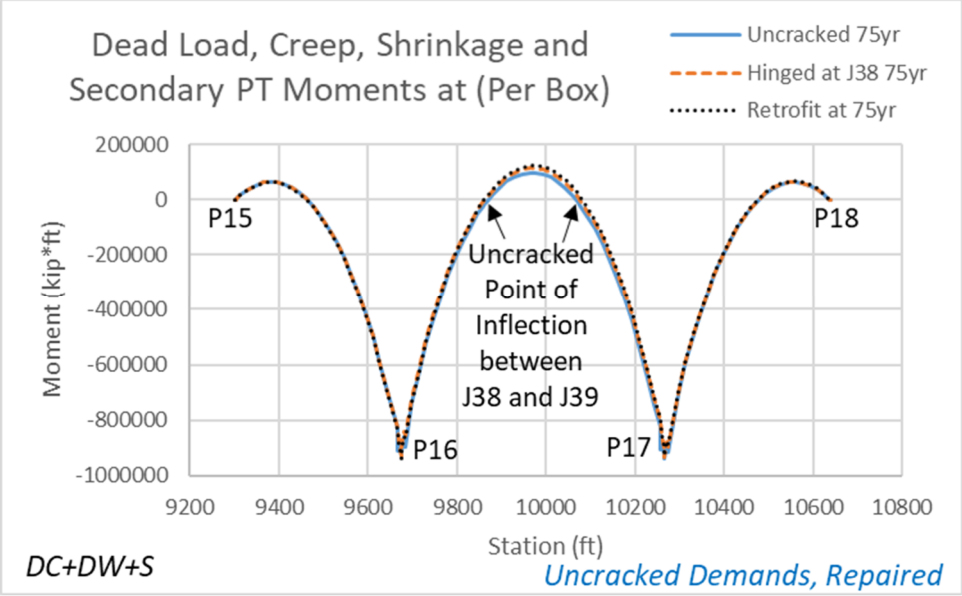
0.9DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Hinged Demands, Repaired* +TU with min DL factor

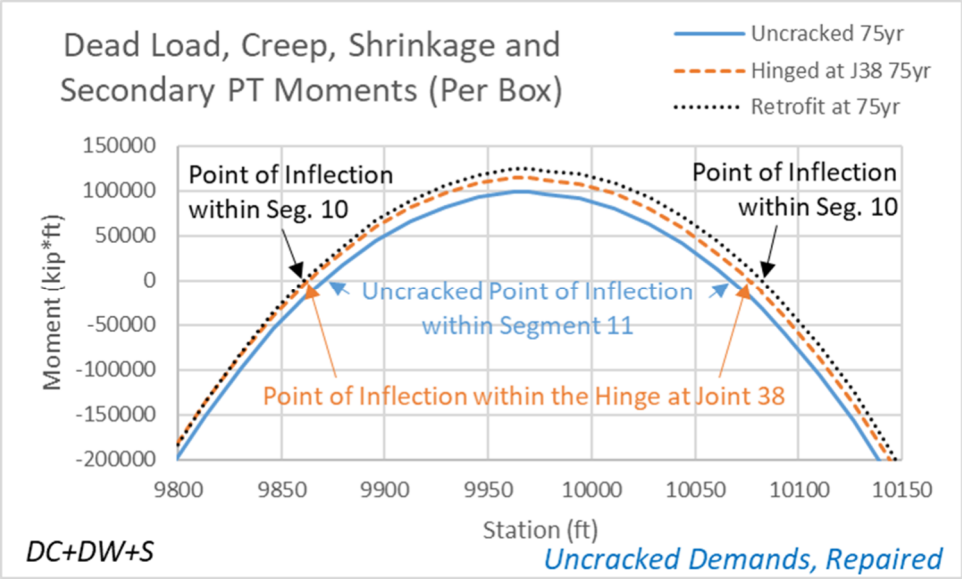


+TU with min DL factor

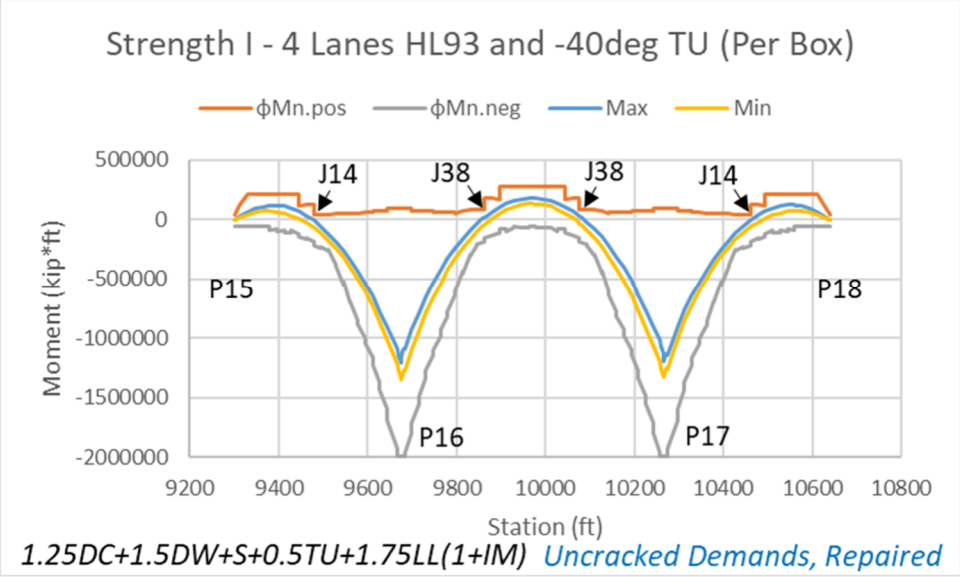
Strength I Moments – Analysis with the Uncracked Model

Note the previous Uncracked and Hinged Models for the existing structure did not include the locked-in forces due to the midspan closure form travelers that lead to slight asymmetry in the response. Hinged and Uncracked model results prior to any retrofits are shown for comparison only.

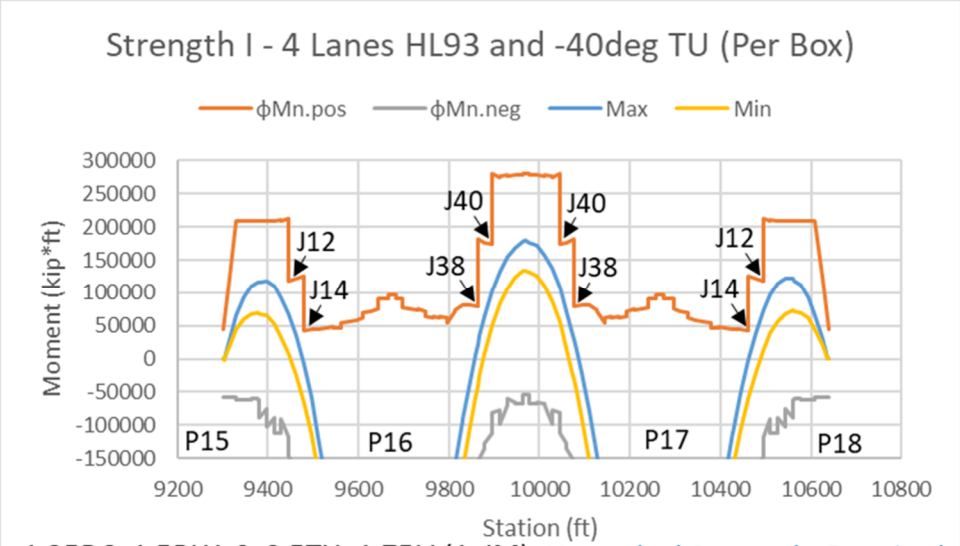




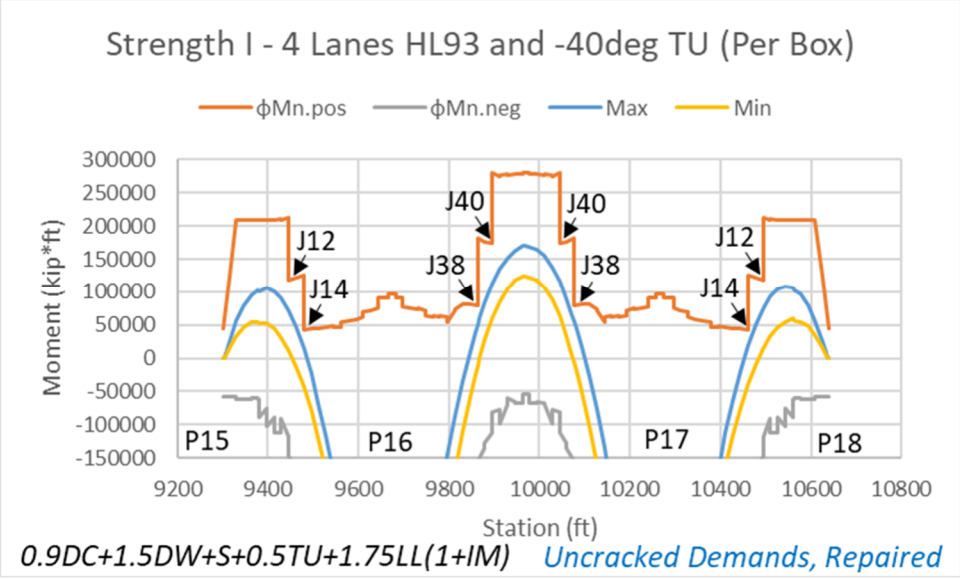
For each thermal case considered and maximum/minimum DC load factor, the provided moment capacity exceeded the factored Strength I demands as shown in the following.



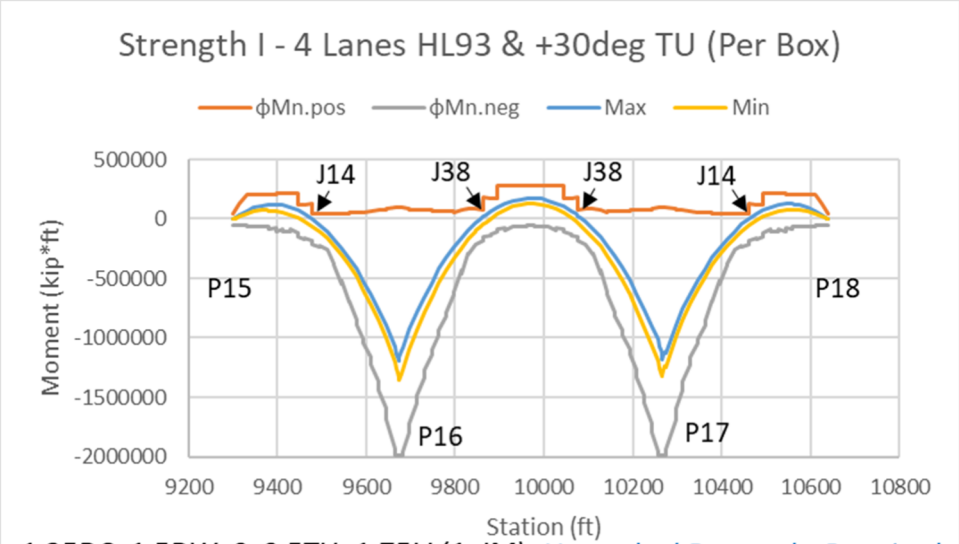
-TU with max DL factor



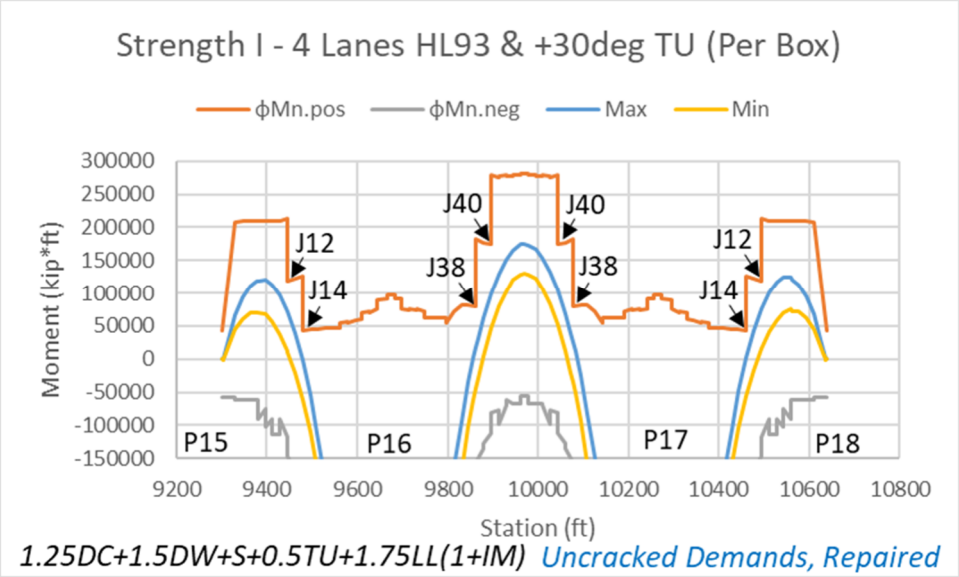
1.25DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Uncracked Demands, Repaired* -TU with max DL factor



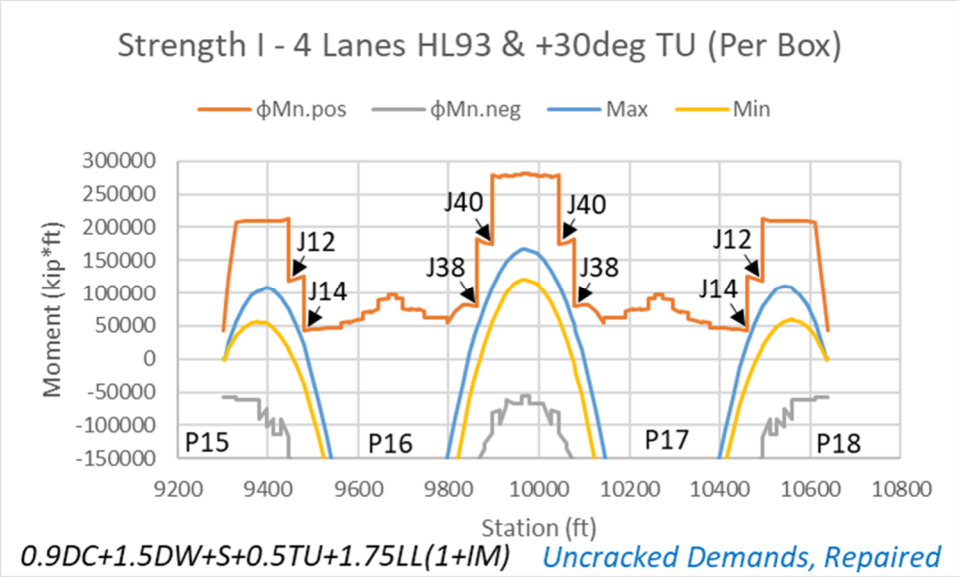
0.9DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Uncracked Demands, Repaired* -TU with min DL factor



1.25DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Uncracked Demands, Repaired* +TU with max DL factor



1.25DC+1.5DW+S+0.5TU+1.75LL(1+IM) *Uncracked Demands, Repaired* +TU with max DL factor



+TU with min DL factor

Strength I Shear – Analysis with the Hinged Model Demands Locked in by the Retrofit

The design shear capacity was calculated according to LRFD 8th Ed. § 5.7.3.3. The minimum tension force resultant required to develop the considered shear resistance was evaluated according to LRFD 8th Ed. § 5.7.3.5. A summary of capacity/demand ratios at key locations is provided in the following table.

Factored Strength I axial forces were considered. Live load axial forces in equilibrium with the influence maximum and minimum shears were considered. Uniform thermal (TU) effects included friction developed at the supports of Piers 15 and 18.

The original shear capacity at Joints 38 computed according to LRFD 8th Ed. § 5.7.3.3 satisfied the Strength I demands with four HL93 lanes loaded per box girder. The provided tension force resultant considering only the mild reinforcement and post-tensioning at the bottom of the section was lower than that required by LRFD 8th Ed. § 5.7.3.5; however, an adequate tension force resultant is obtained when also considering the longitudinal soffit retrofit CFRP. Sample calculations from YP 5/27/20 regarding the CFRP longitudinal factored resistance considered appear below.

The fraction of the mechanism shear resistance that can be developed was conservatively scaled by the ratio of the provided to required tensile force resultant, considering only the conventional reinforcement and post-tensioning. This conservative reduced shear capacity was found to exceed the factored demands.

	Vu with Mu.max with Hinged Model Demands			
	Original	Tension Resultant		Reduced
Location	$\phi V_n/V_u$	$T_{provided}$ / $T_{required}$	ΔT (kips)	$\phi V_n^R/V_u$
P15 J2	1.08	1.74	-	-
P15 J3	1.14	1.16	-	-
P16 J38	1.15	1.03	-	-
P16 J40	1.45	1.03	-	-
P16 J41	1.38	1.15	-	-
P17 J41	1.31	1.42	-	-
P17 J40	1.42	1.01	-	-
P17 J38	1.17	0.97	297	1.13
P18 J3	1.12	1.16	-	-
P18 J2	1.06	1.73	-	-
Per YP 5/27/20 Calc for Longitudinal CFRP, 3 layers provides 1425 kips of factored resistance. Satisfying ΔT if bonded.				

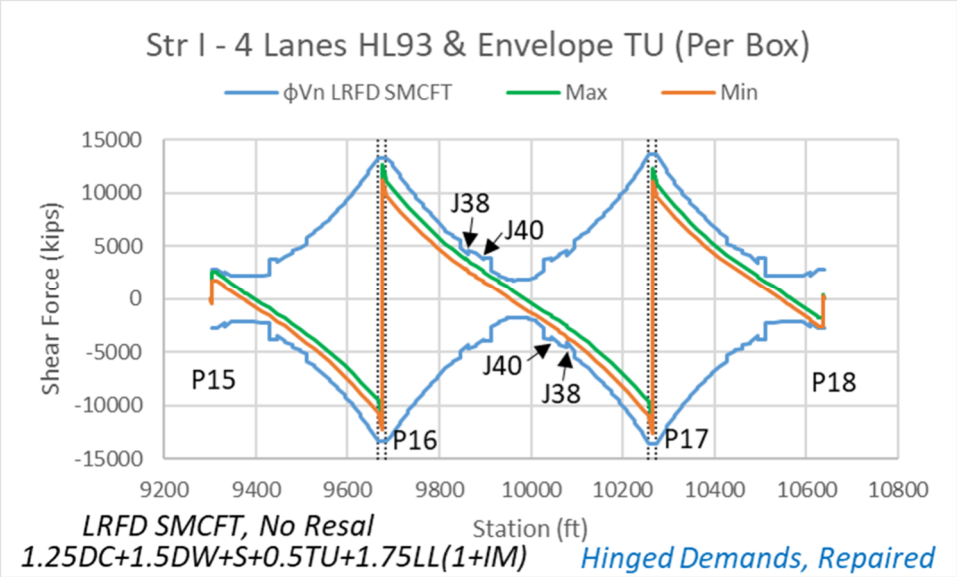
Still 297 kips short without including any soffit CFRP. Note that there is no room for additional PT given the compressive stress exceedances. Evaluations including stresses in the unbonded retrofit tendons greater than fpe that exist due to deformation compatibility would decrease or potentially eliminate the gap in the provided tension force resultant.

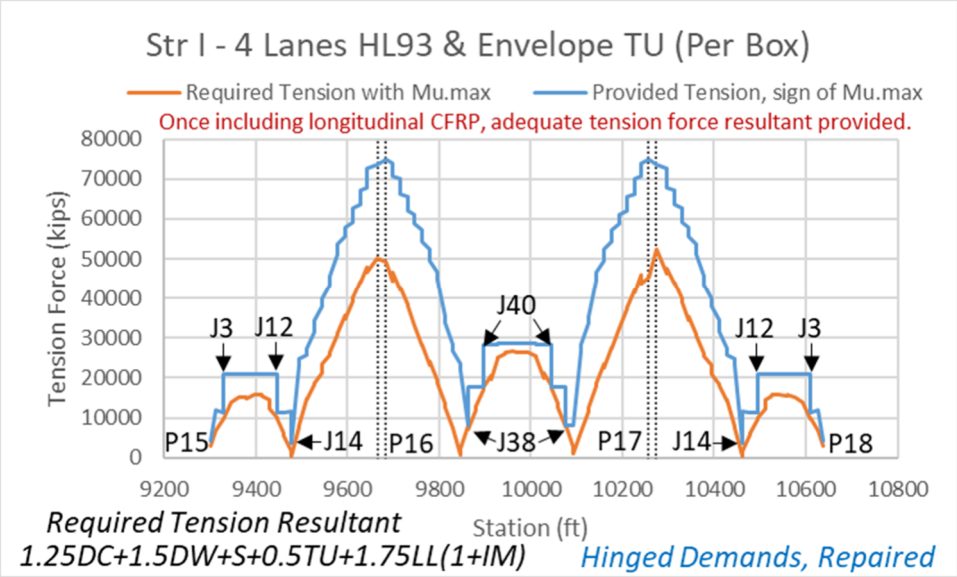
Resistance per one layer per unit width $N_{r.frp1} := \phi_{frp.f} \cdot f_{fef} \cdot t_f = 1.802 \ \frac{kip}{in}$

Soffit width $b_{frp} := 24 \cdot ft - 2 \cdot ft = 22 \ ft$

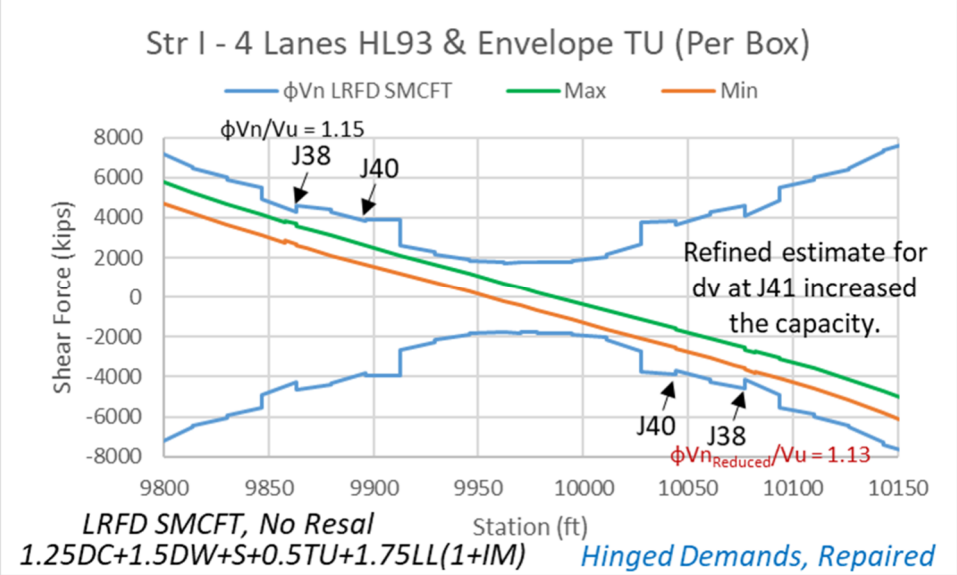
from YP 5/27/20

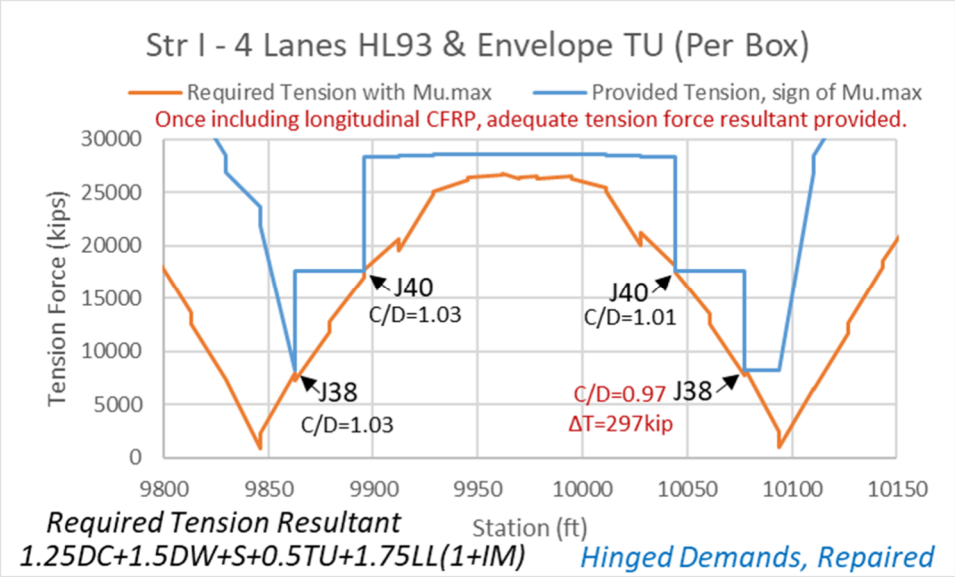
Considered longitudinal CFRP factored resistance = 1.8 kip/in * (22*12 in) * 3 = 1425 kips



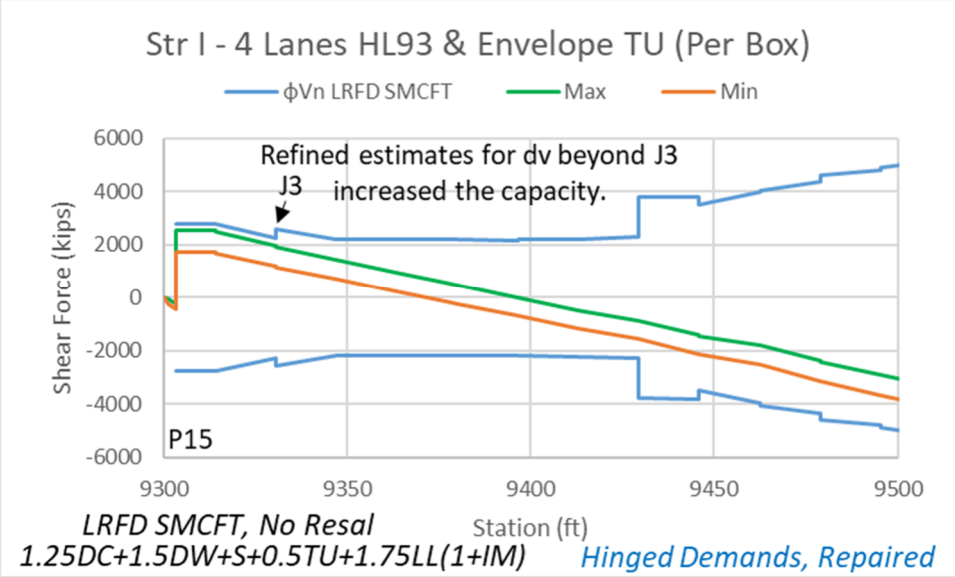


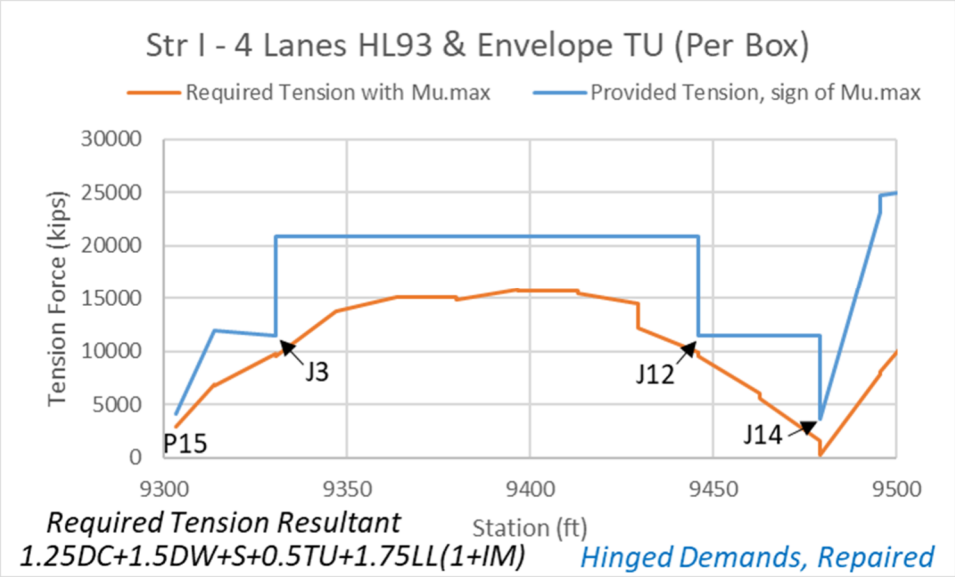
M.max = most positive



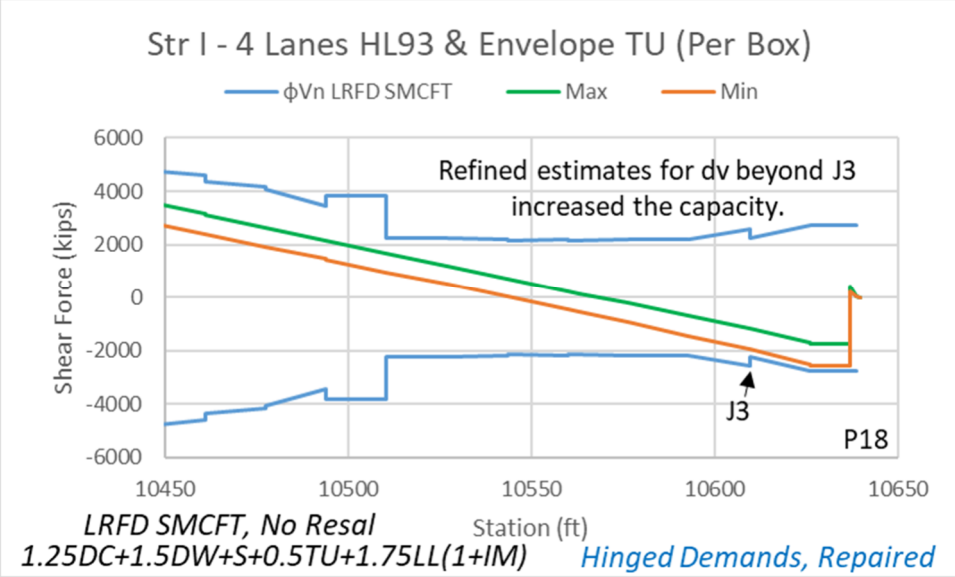


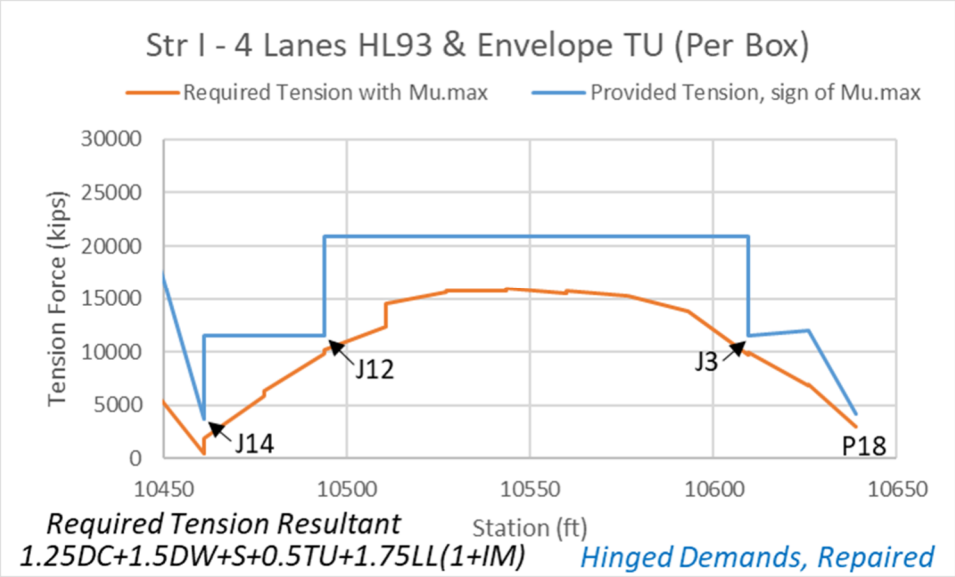
M.max = most positive



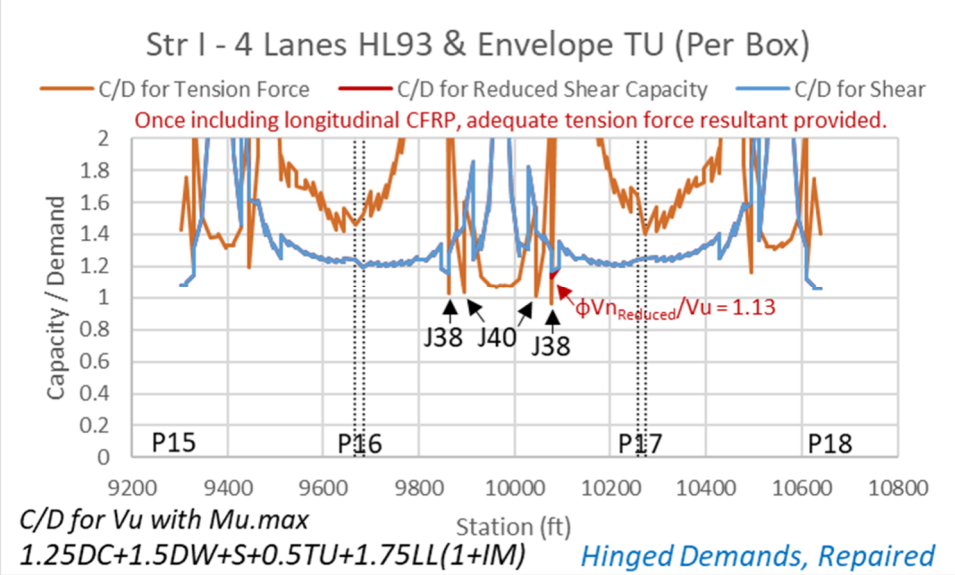


M.max = most positive

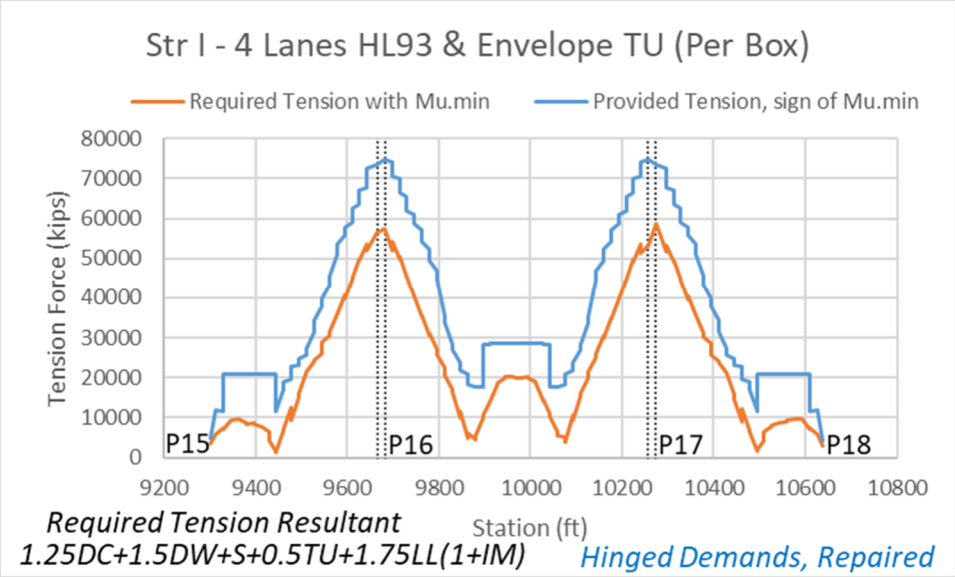




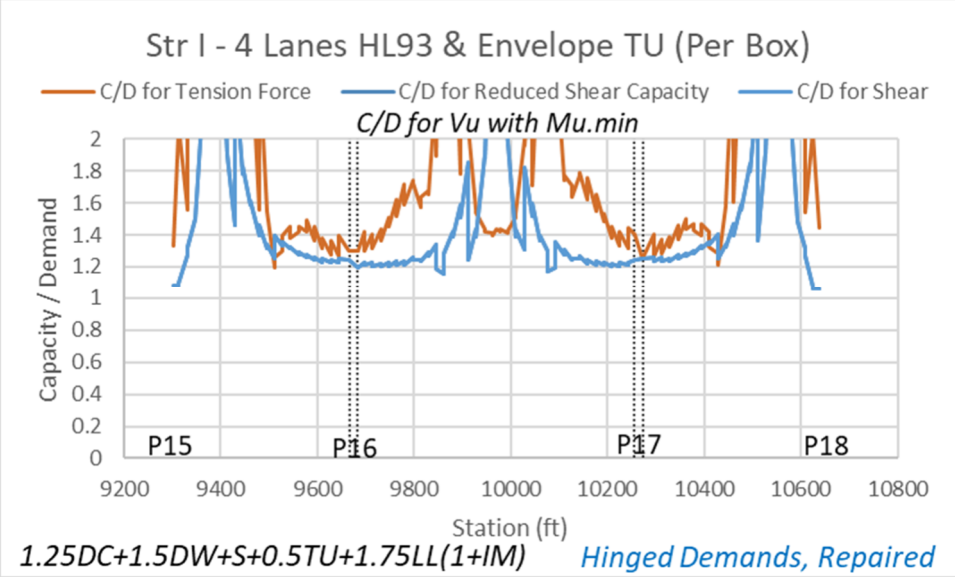
M.max = most positive



M.max = most positive



M.min = most negative

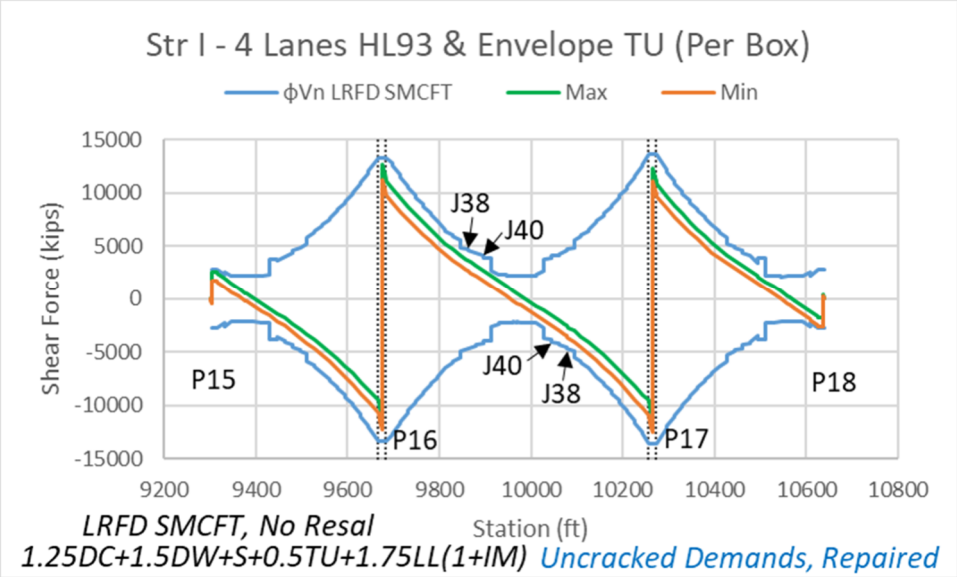


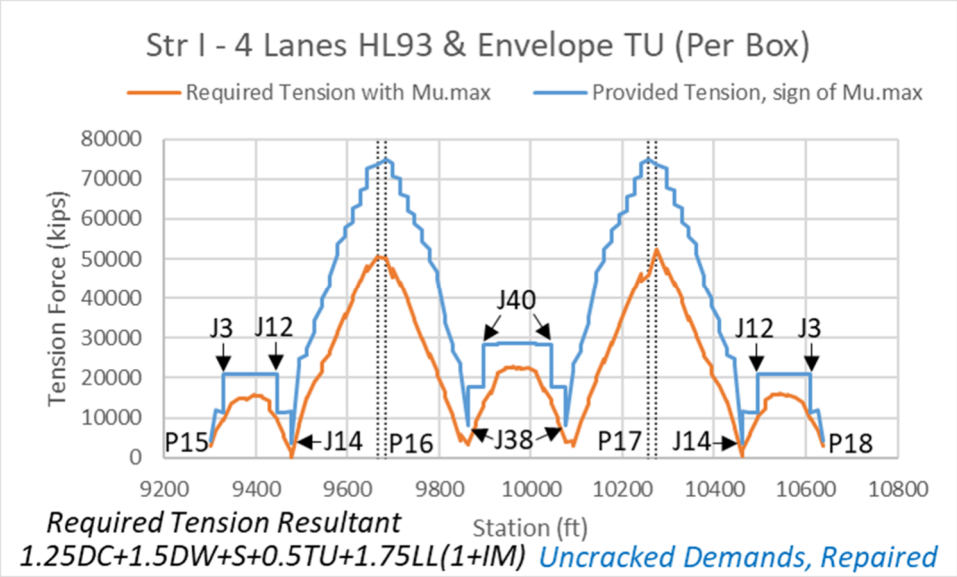
M.min = most negative

Strength I Shear – Analysis with the Uncracked Model Demands

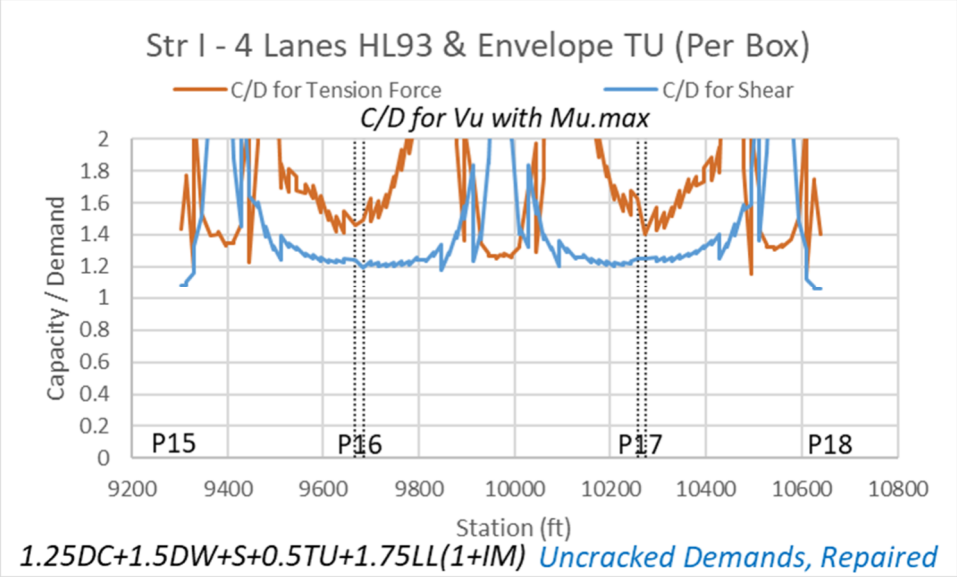
The design shear capacity was calculated according to LRFD 8th Ed. § 5.7.3.3. The minimum tension force resultant required to develop the considered shear resistance was evaluated according to LRFD 8th Ed. § 5.7.3.5. A summary of capacity/demand ratios at key locations is provided below. The computed shear capacity and provided tension force resultants were found to satisfy the Strength I demands with four HL93 lanes loaded per box girder.

	Vu with Mu.max with Uncracked Model Demands		
	Original	Tension Resultant	Reduced
Location	$\phi V_n/V_u$	$T_{provided} / T_{required}$	$\phi V_n^R/V_u$
P15 J2	1.09	1.75	-
P15 J3	1.16	1.17	-
P16 J38	1.29	2.58	-
P16 J40	1.58	1.36	-
P16 J41	1.42	1.37	-
P17 J41	1.32	1.70	-
P17 J40	1.62	1.29	-
P17 J38	1.32	2.03	-
P18 J3	1.12	1.15	-
P18 J2	1.05	1.72	-

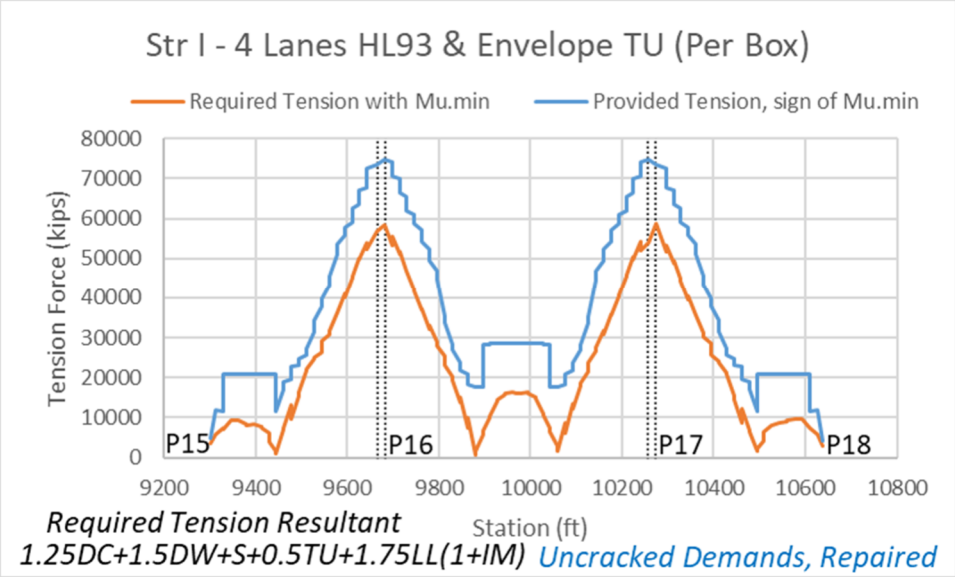




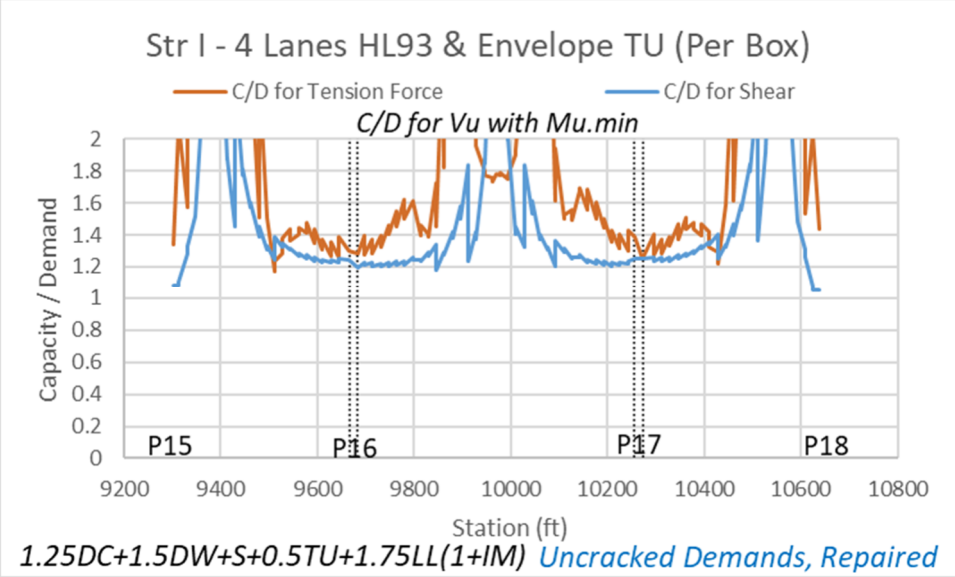
M.max = most positive



M.max = most positive



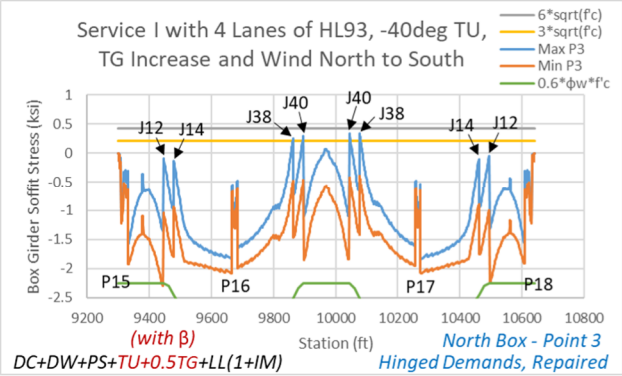
M.min = most negative

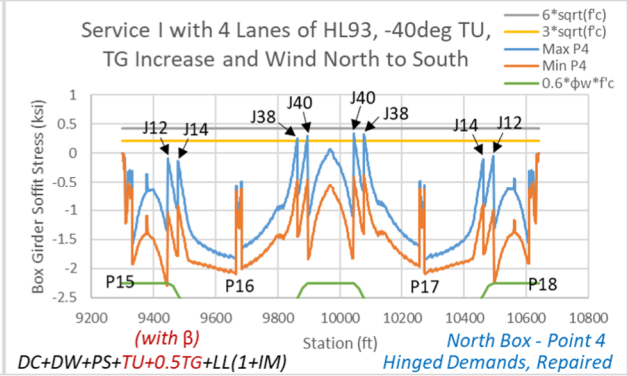


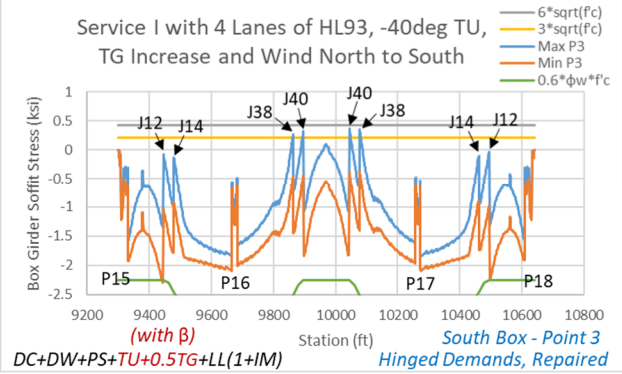
M.min = most negative

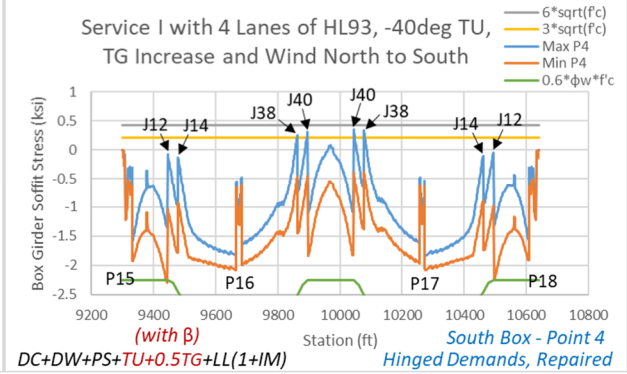
Service I – Analysis with the Hinged Model Demands Locked in by the Retrofit

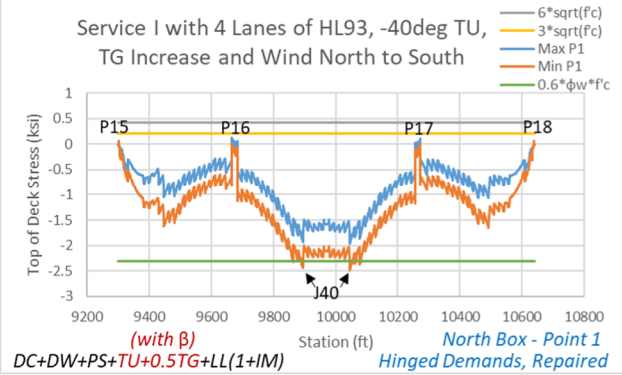
In the following figures, stress limits are calculated using the original 5ksi design concrete strength. Locations for the stress points and a description of the envelope procedure used to calculate the WS and WL effects were previously discussed. Stresses are only provided for 0deg transverse wind. Results are provided for the North windward box and South leeward boxes. A symmetric arrangement of four lanes per box was considered to maximize live load effects. Note that only evaluation for compressive stress limits are required for Service I, but tensile limits for Service III are shown for reference.

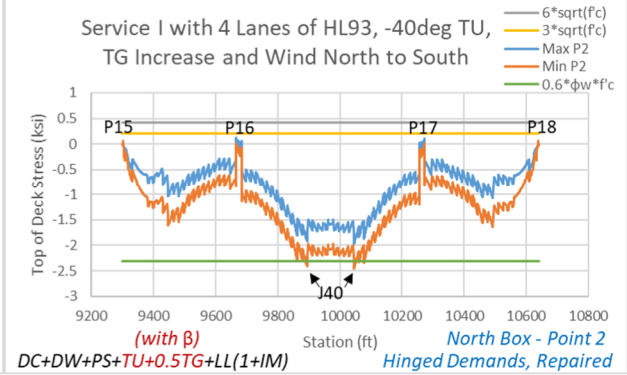


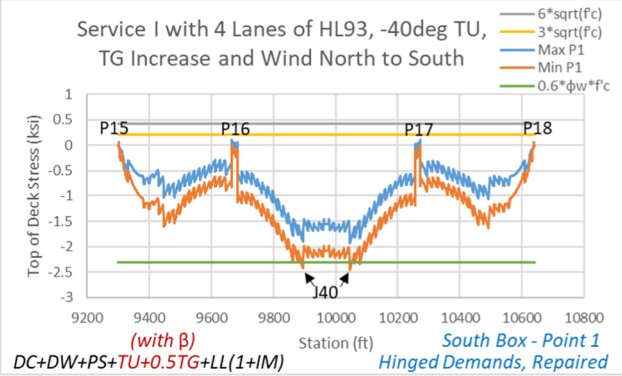


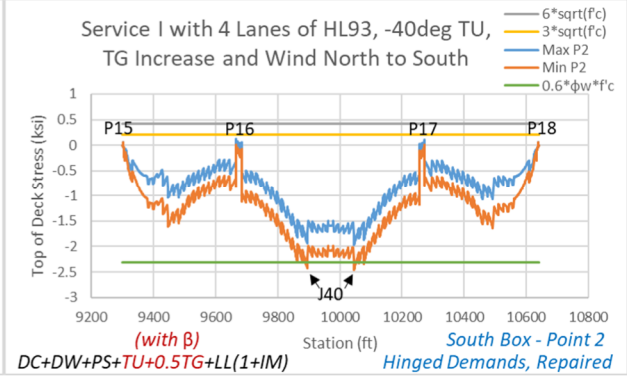




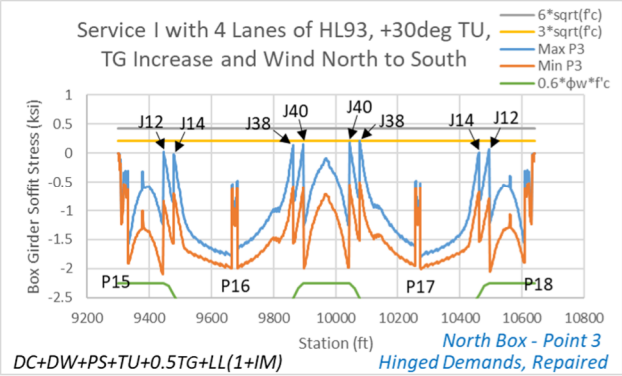


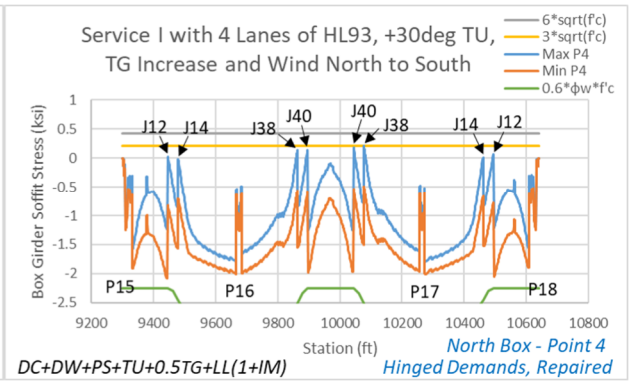


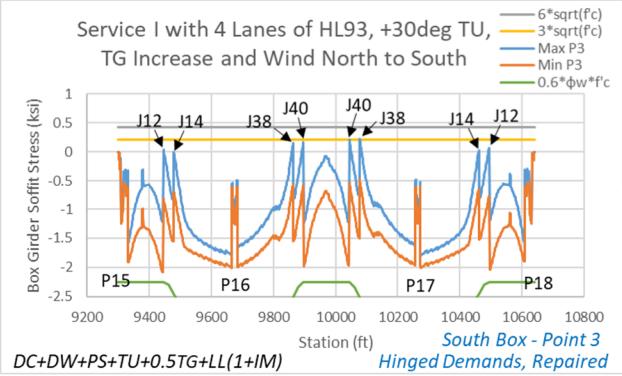


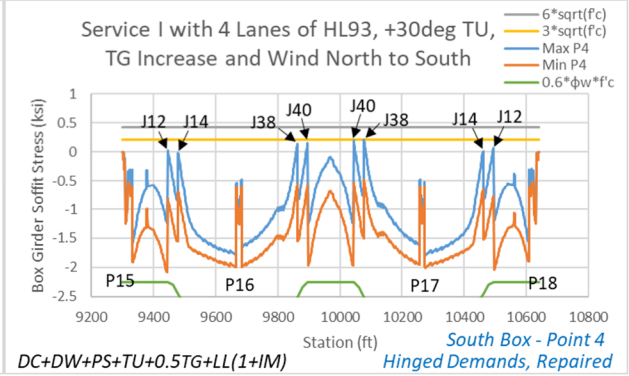


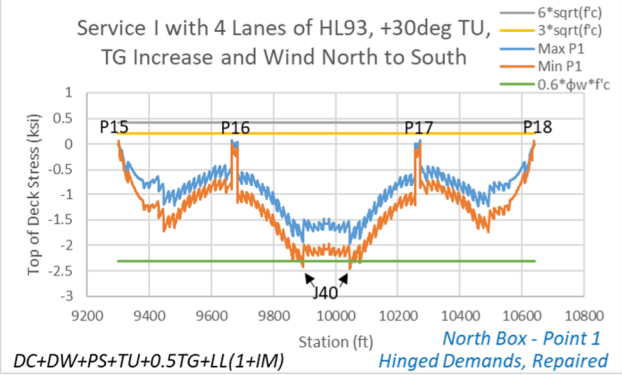
Results above are for the -TU and TG Deck Increase thermal case that maximizes main span soffit tension and deck compression. The main span Service I deck compressive stresses exceed applicable limits as shown.

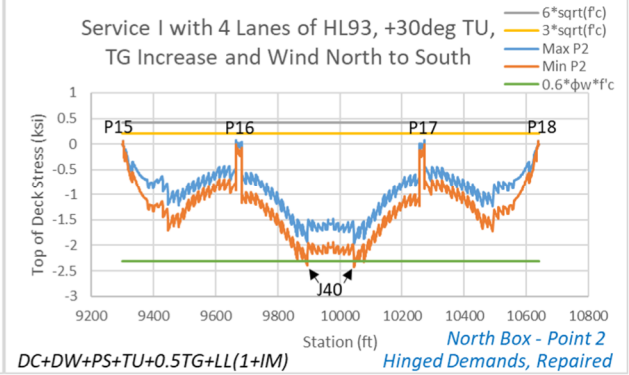


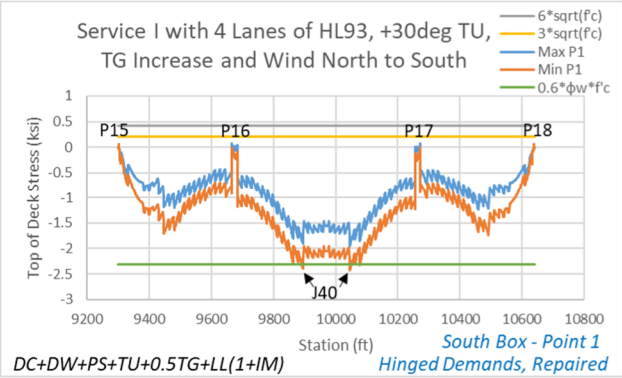


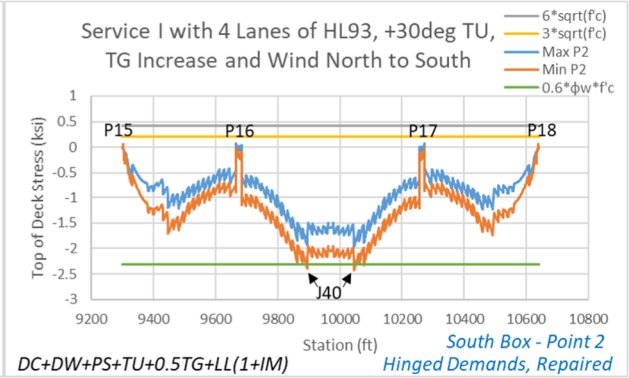




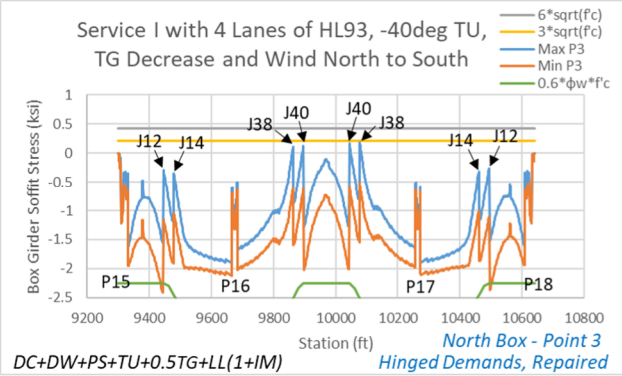


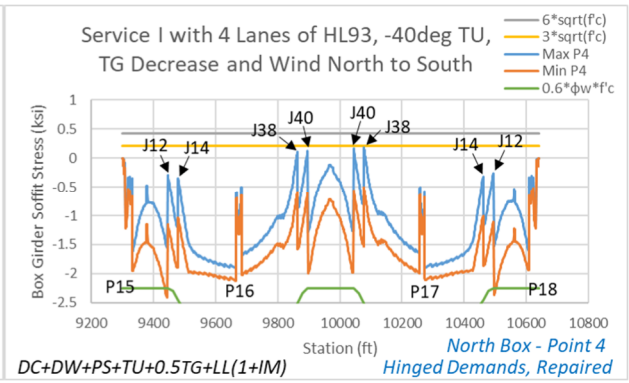


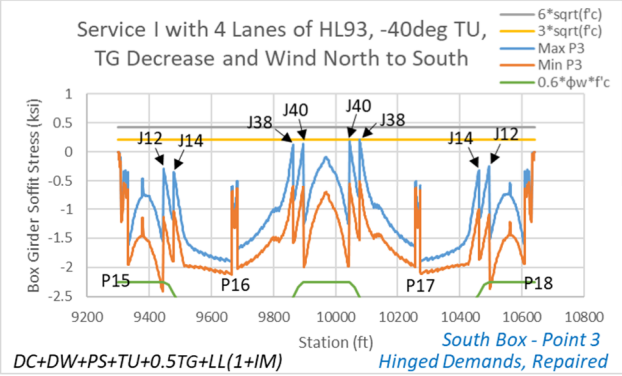


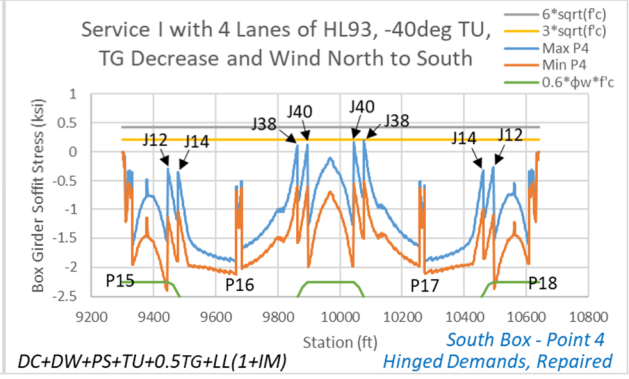


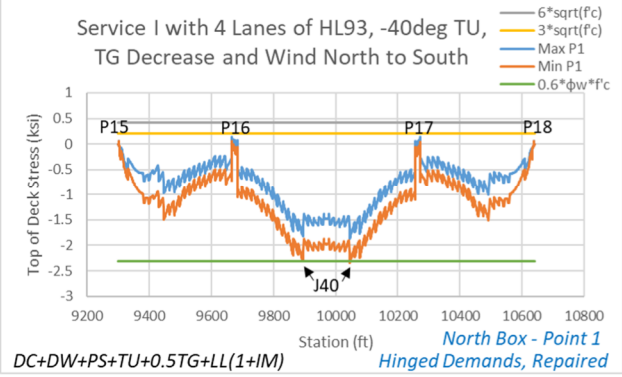
Results above are for the +TU and TG Deck Increase case that maximizes soffit tension and deck compression in the side spans. The main span deck compressive stress exceedance was noted previously.

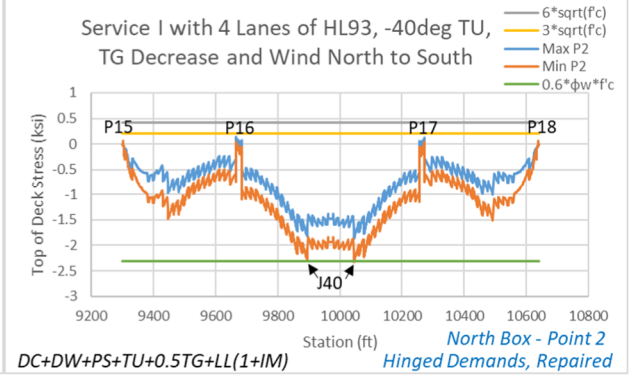


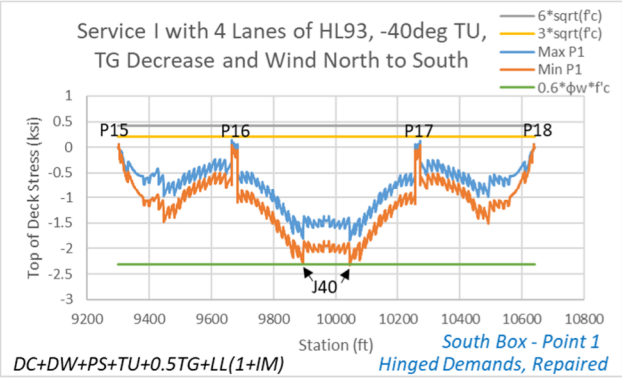


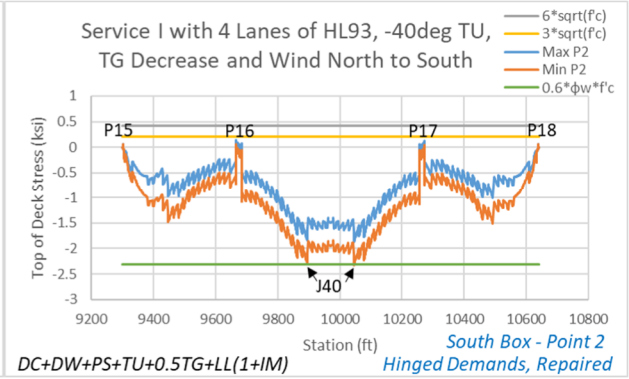




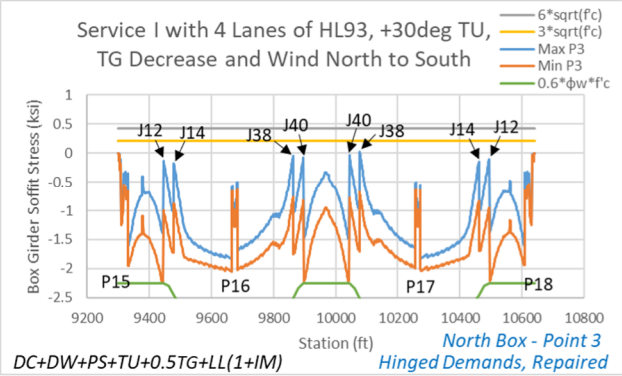


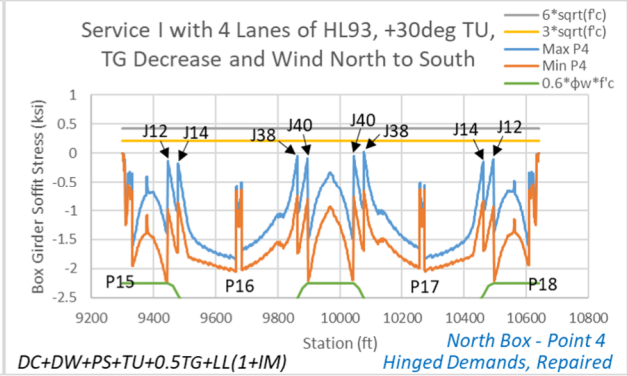


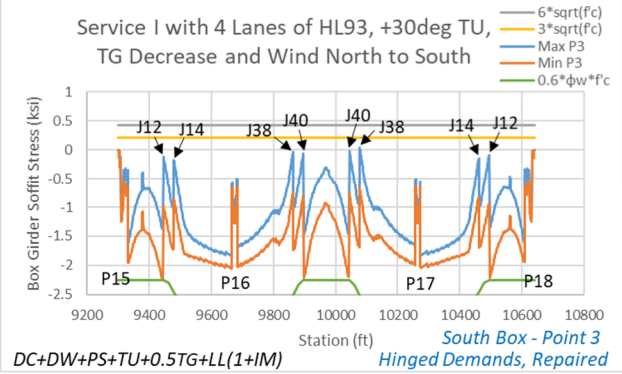


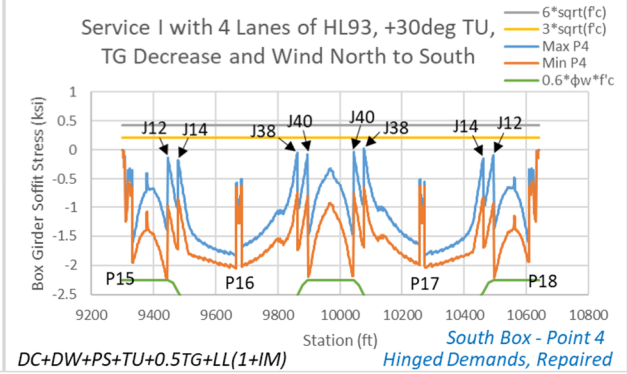


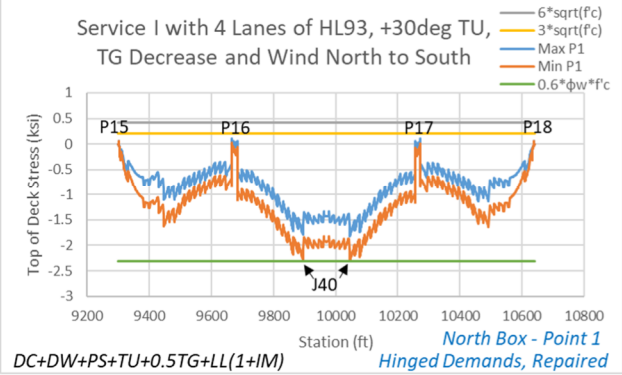
Results above are for the -TU and TG Deck Decrease case that maximizes soffit compression in the side spans. The Service I side span soffit stresses exceed applicable limits as shown.

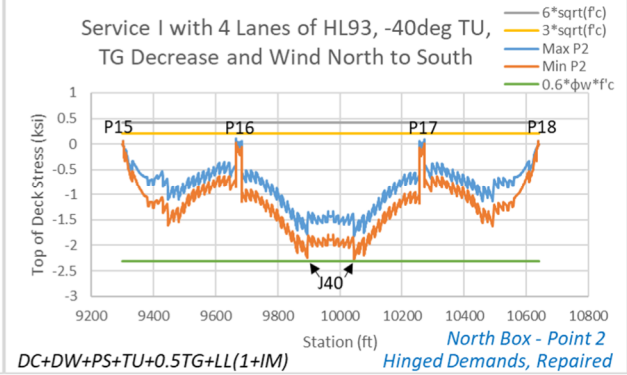


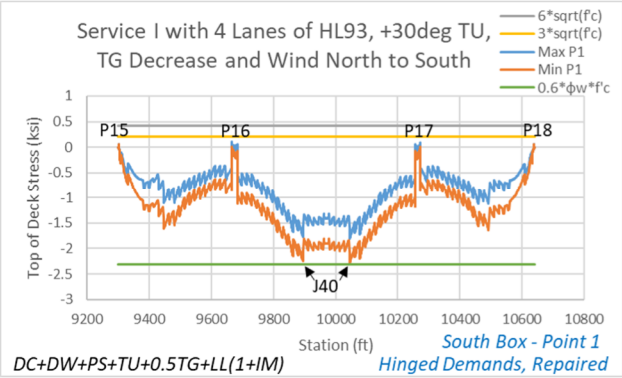


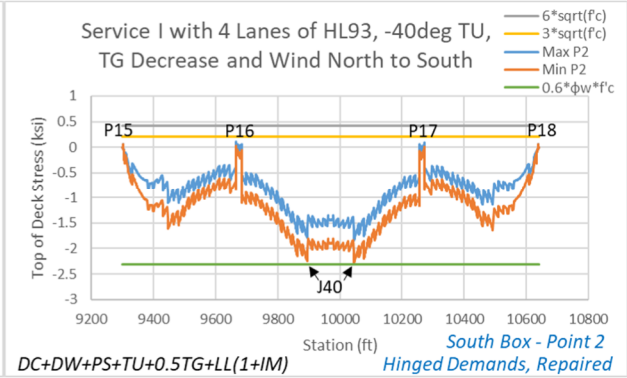






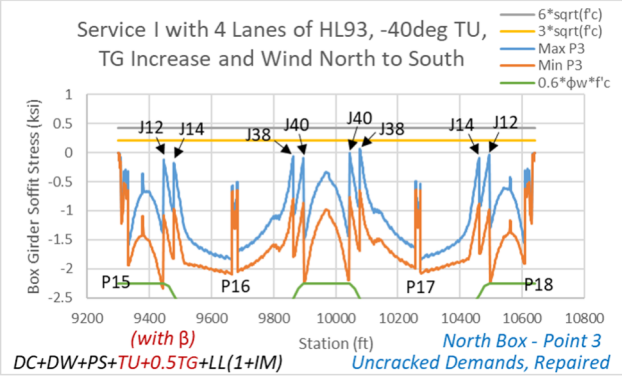


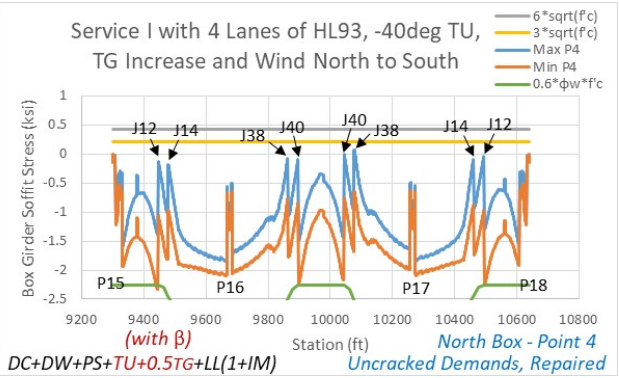


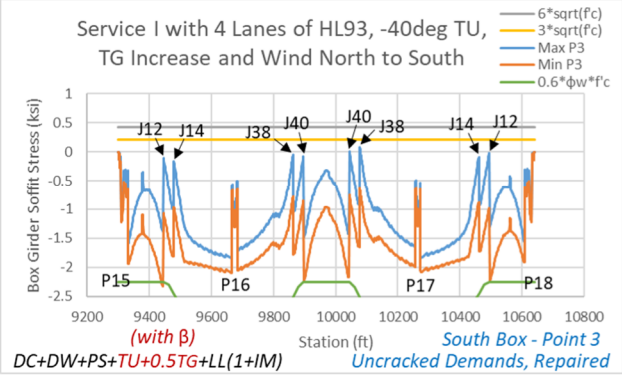


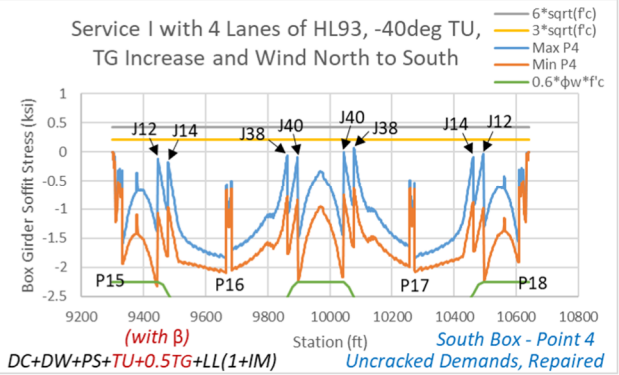
Results above are for the +TU and TG Deck Decrease case that maximizes main span soffit compression. The main span soffit compressive stresses just reach applicable limits. Note that demands from the uncracked analysis maximize main span soffit compression, due to reduced positive moments.

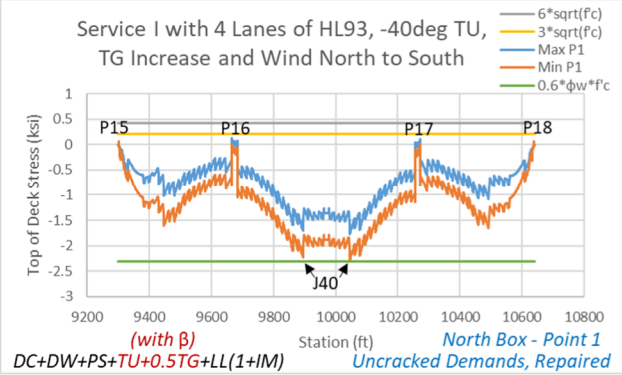
Service I – Analysis with the Uncracked Model Demands

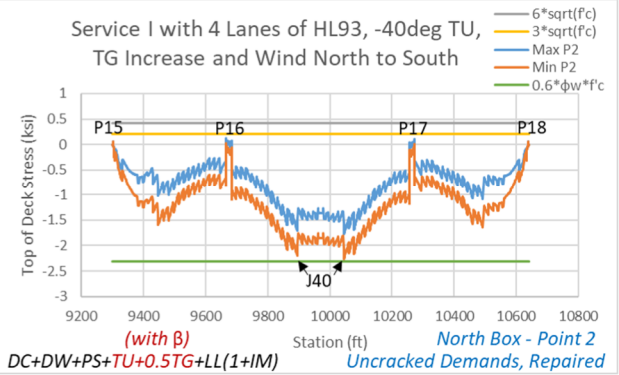


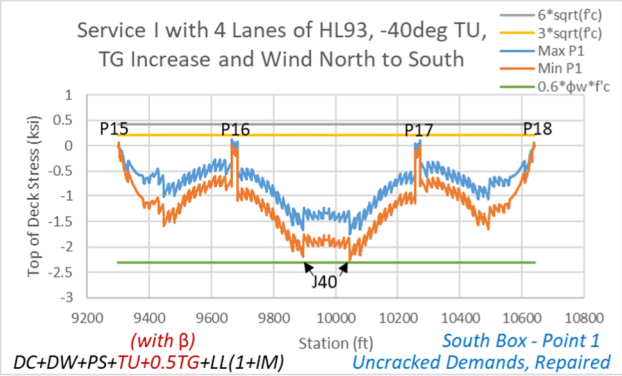


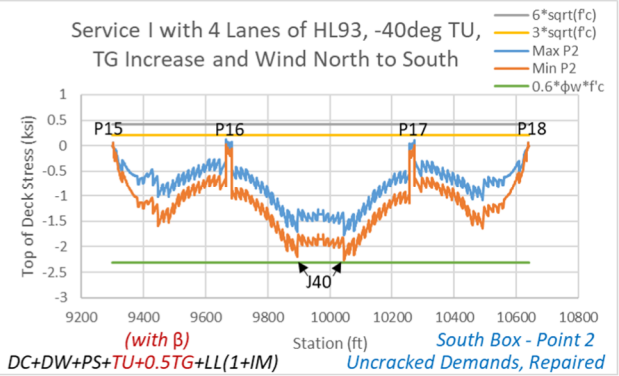




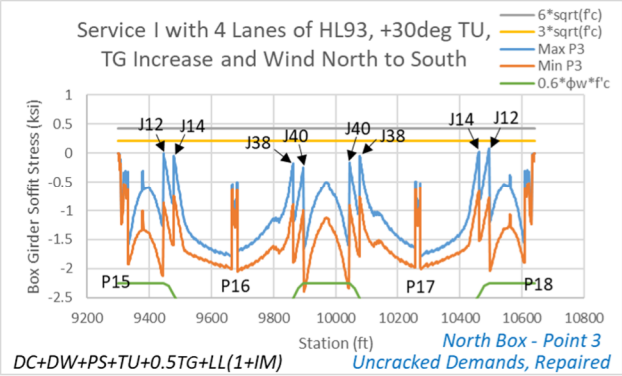


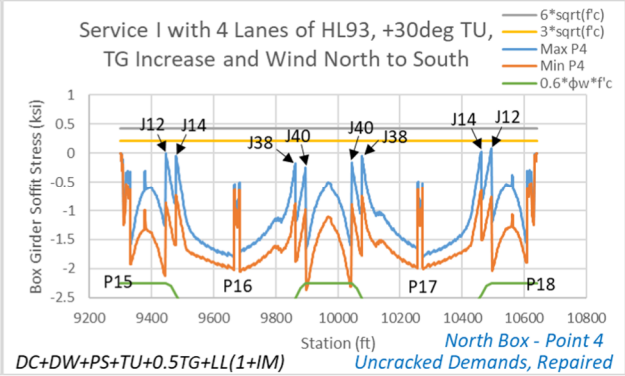


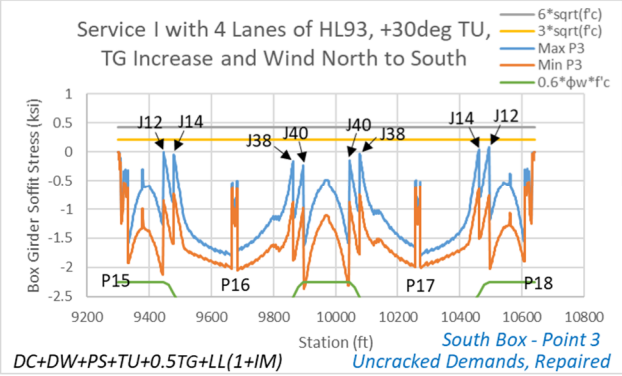


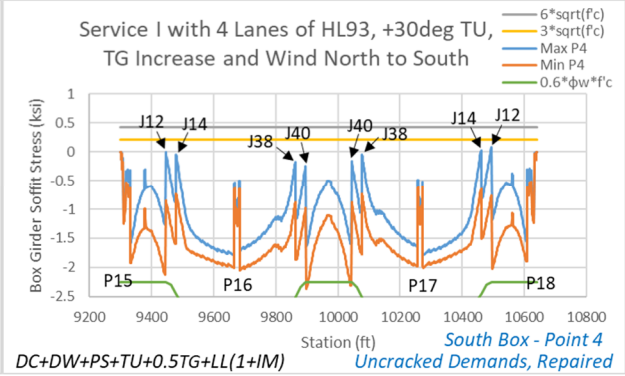


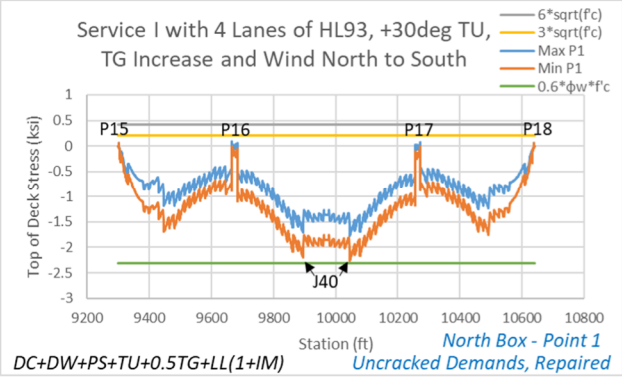
Deck compressive stress exceedances form the hinged model do not appear in the uncracked analysis due to decreased positive moments. Deck stresses are just below applicable limits at Joint 40.

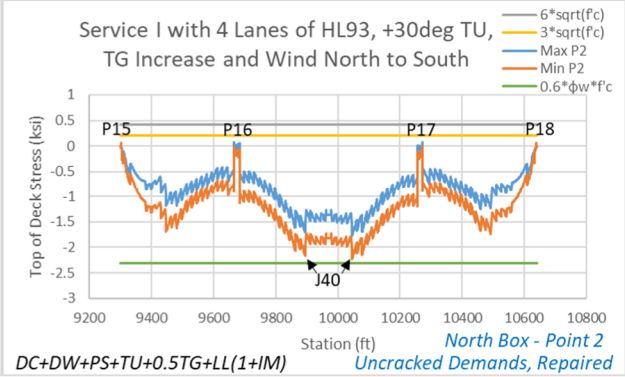


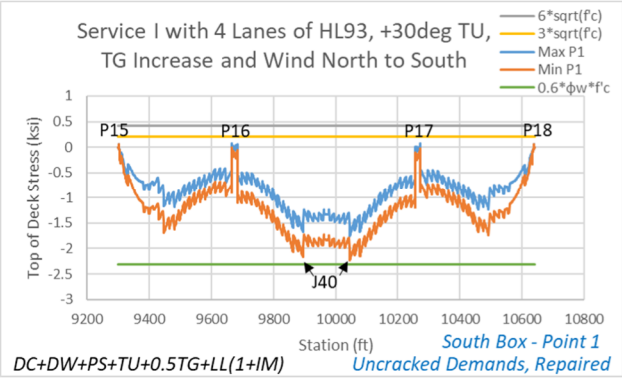


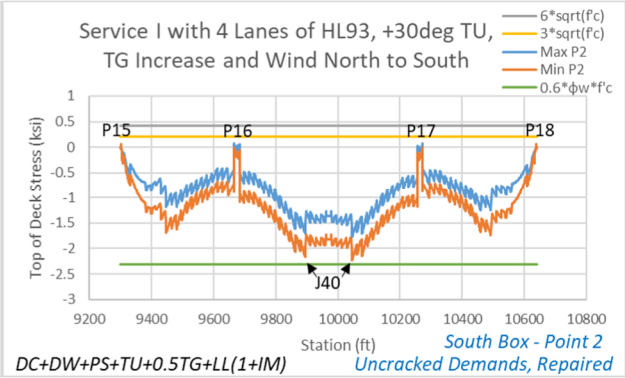




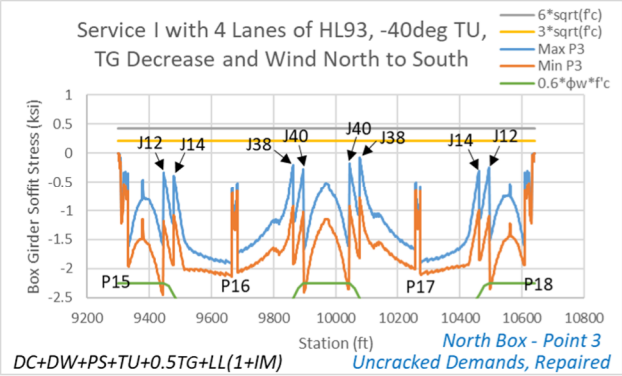


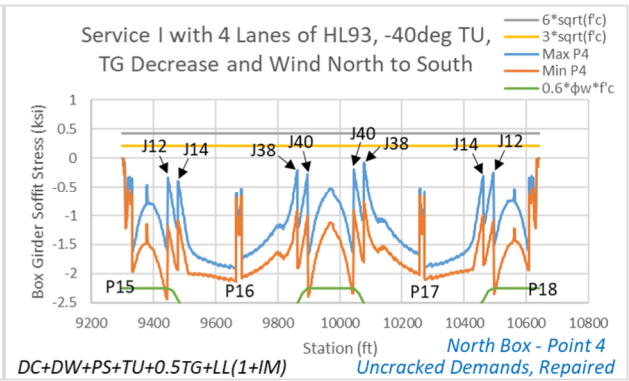


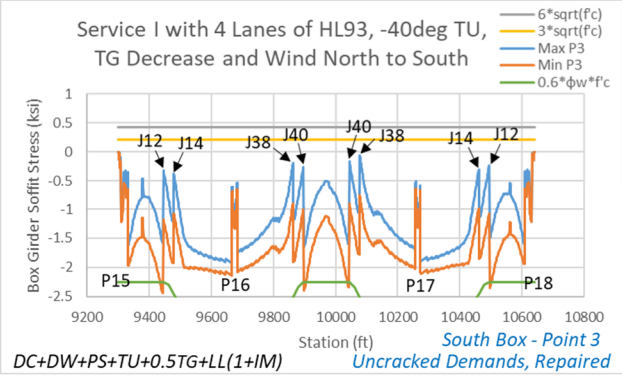


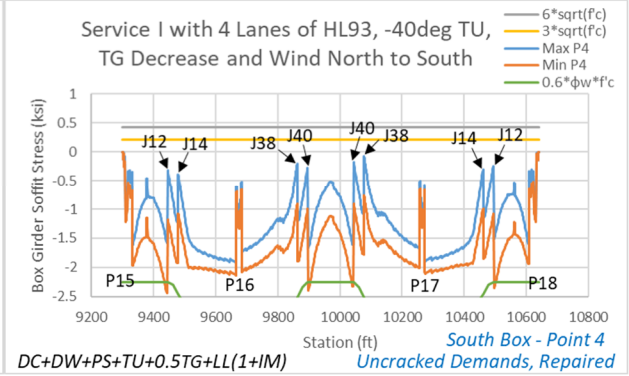


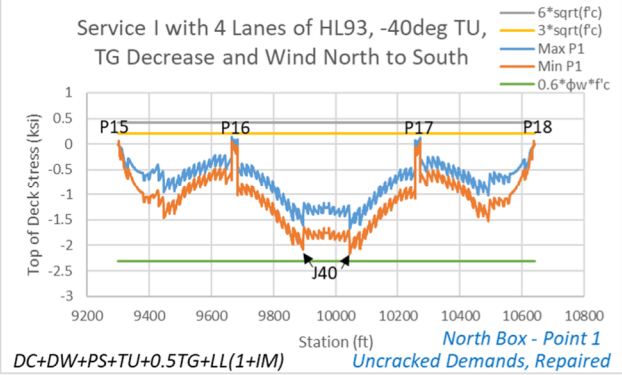
Results above are for the +TU and TG Deck Increase case that maximizes soffit tension and deck compression in the side spans. The main span soffit compressive stress exceedance is maximized under a separate thermal case.

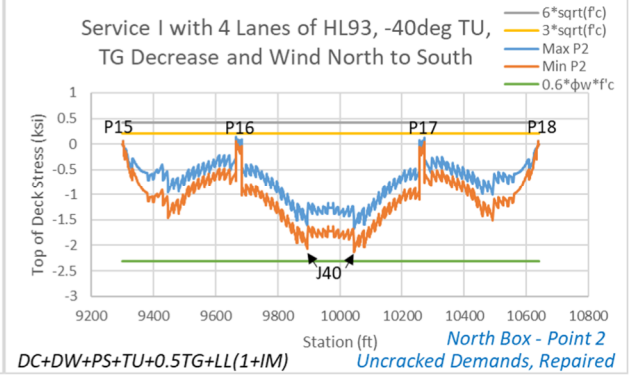


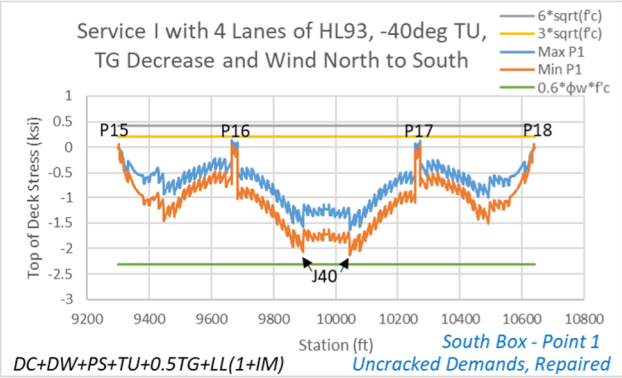


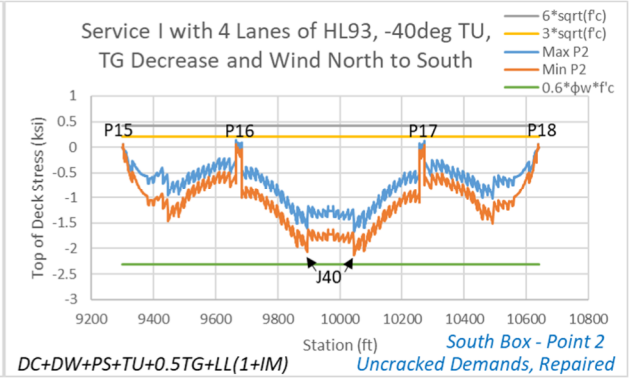




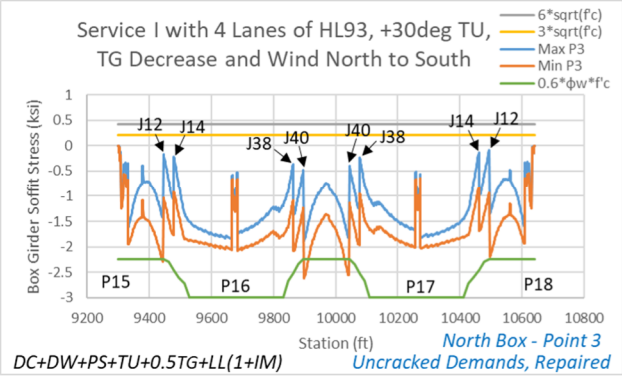


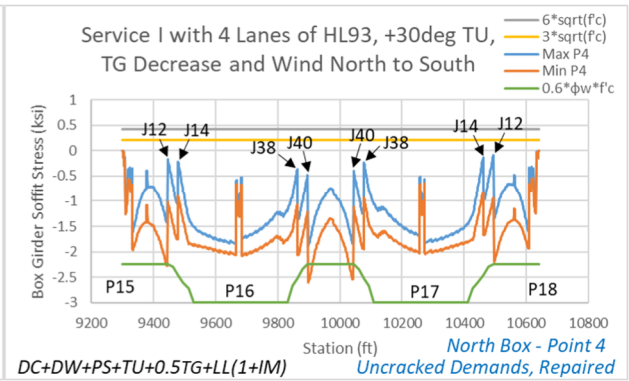


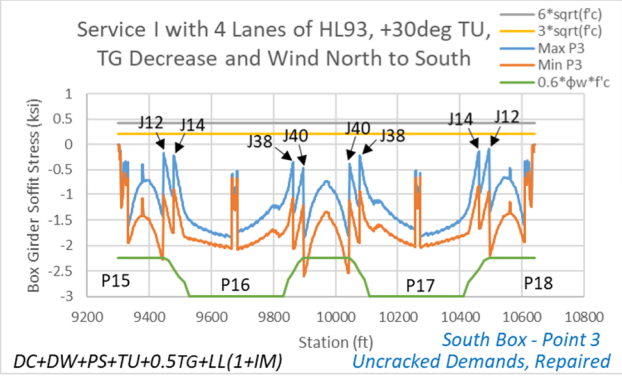


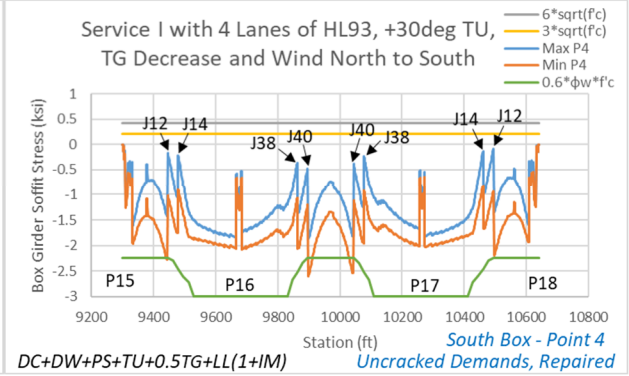


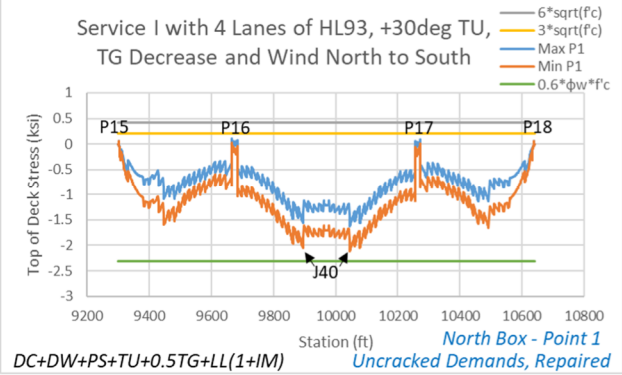
Results above are for the -TU and TG Deck Decrease case that maximizes soffit compression in the side spans. The Service I side span soffit stresses exceed applicable limits as shown.

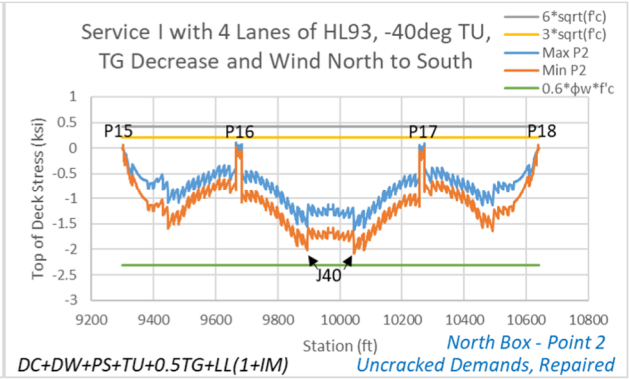


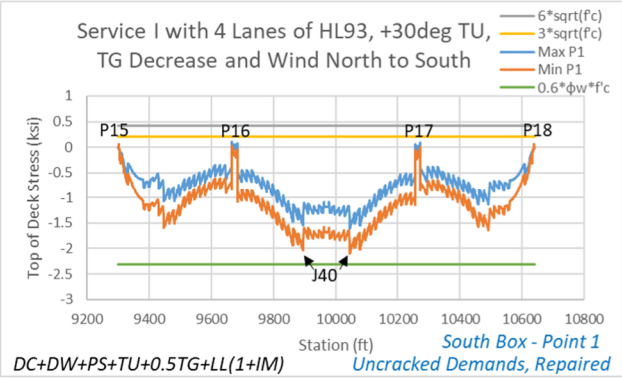


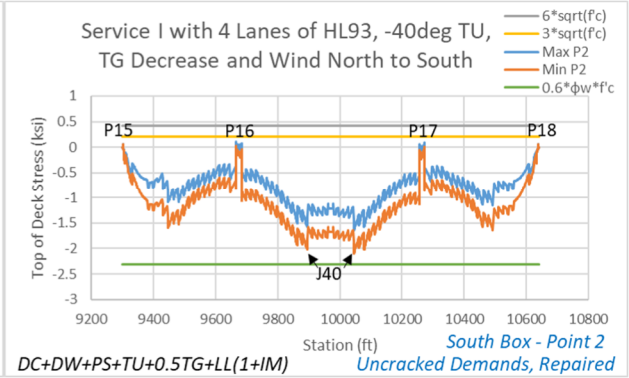








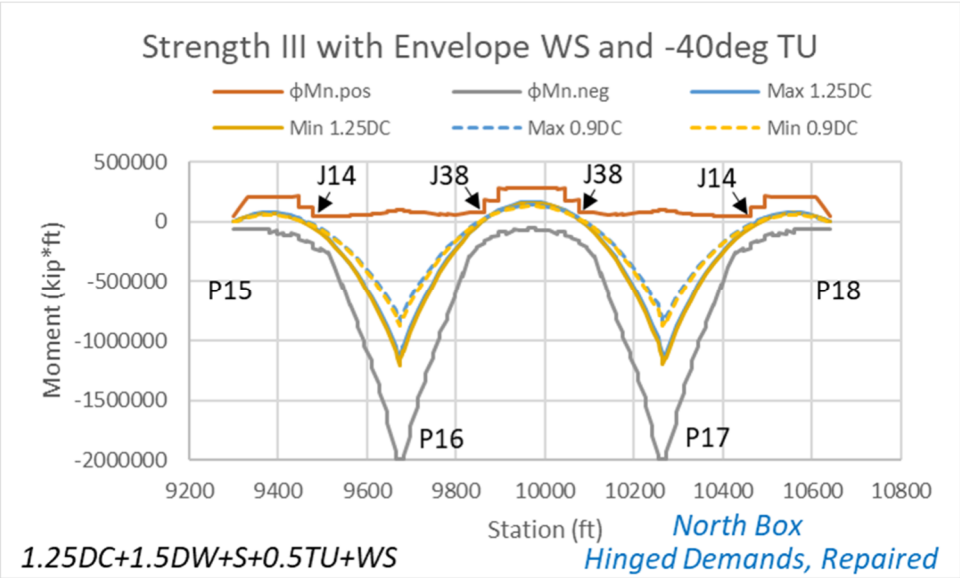


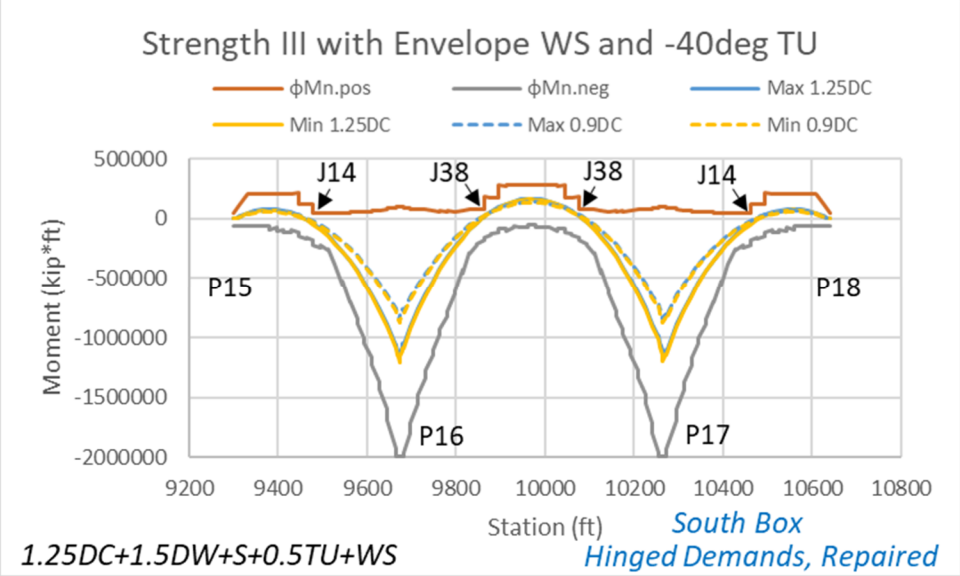


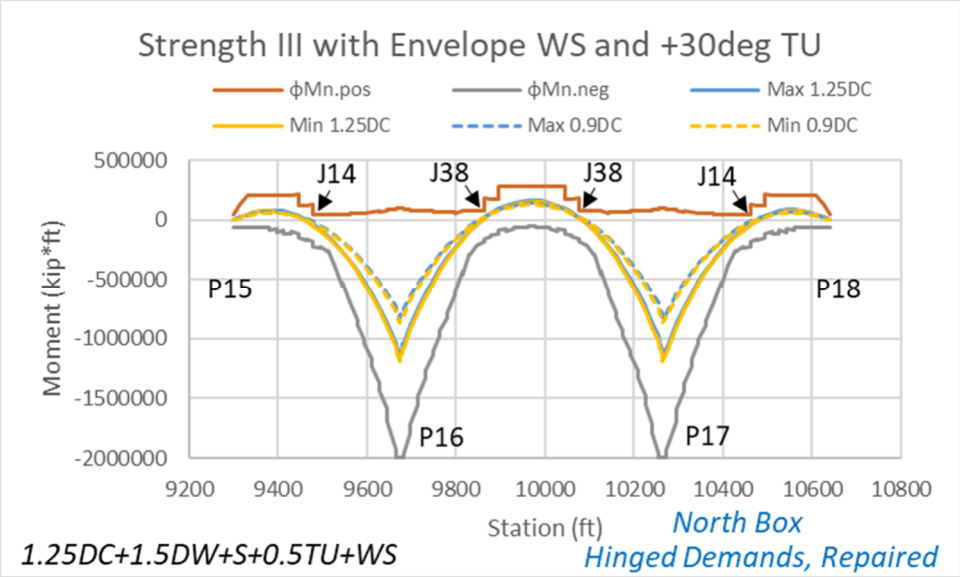
Results above are for the +TU and TG Deck Decrease case that maximizes main span soffit compression. The Service I main span soffit compressive stresses exceed applicable limits as shown.

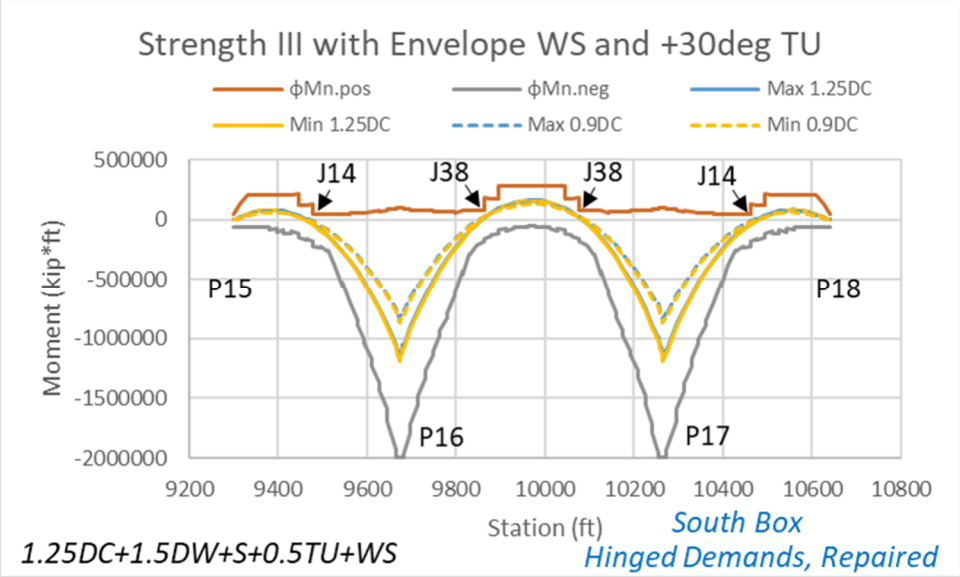
Strength III Moments – Analysis with the Hinged Model Demands

Results are provided for both the North windward and South leeward boxes. Procedures used to determine the envelope WS effects were previously discussed.



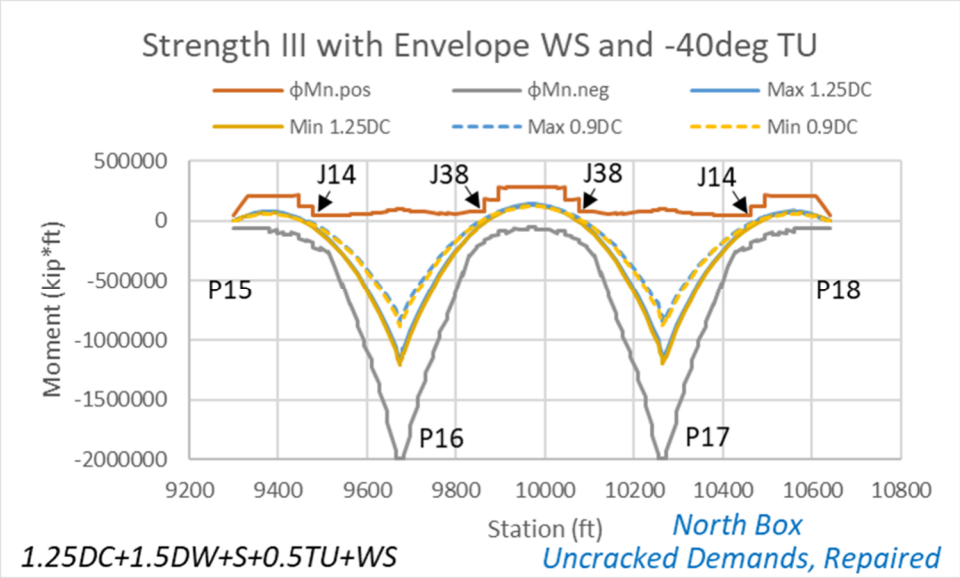


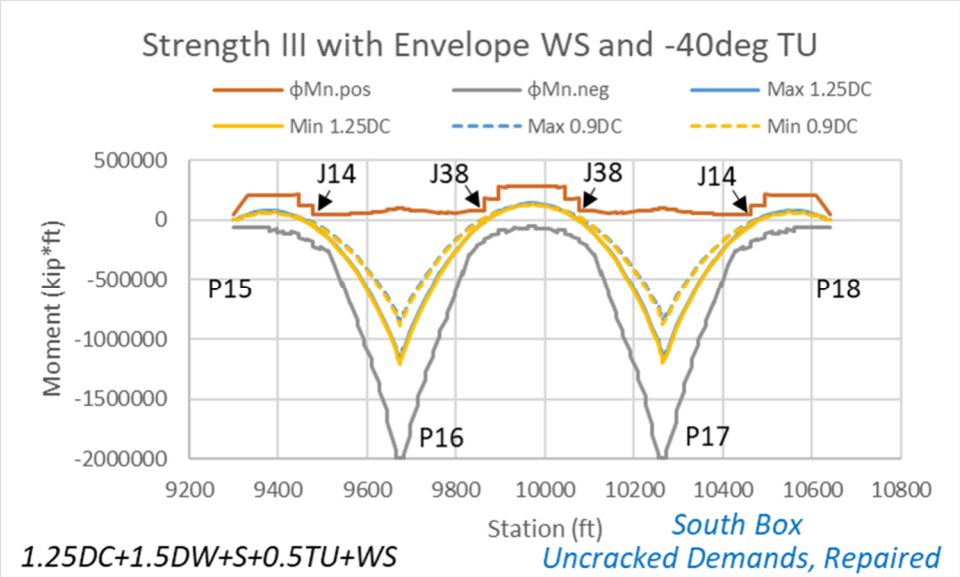

Strength III with Envelope WS and +30deg TU
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
500000
0
-500000
-1000000
-1500000
-2000000
J14
J38
J38
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
North Box
1.25DC+1.5DW+S+0.5TU+WS
Hinged Demands, Repaired

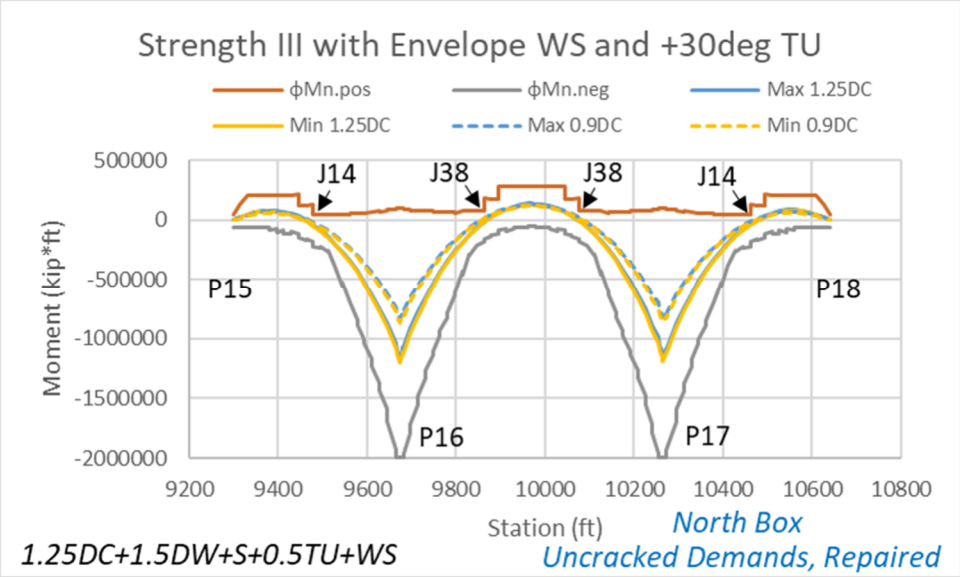

Strength III with Envelope WS and +30deg TU
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
500000
0
-500000
-1000000
-1500000
-2000000
J14
J38
J38
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
South Box
1.25DC+1.5DW+S+0.5TU+WS
Hinged Demands, Repaired

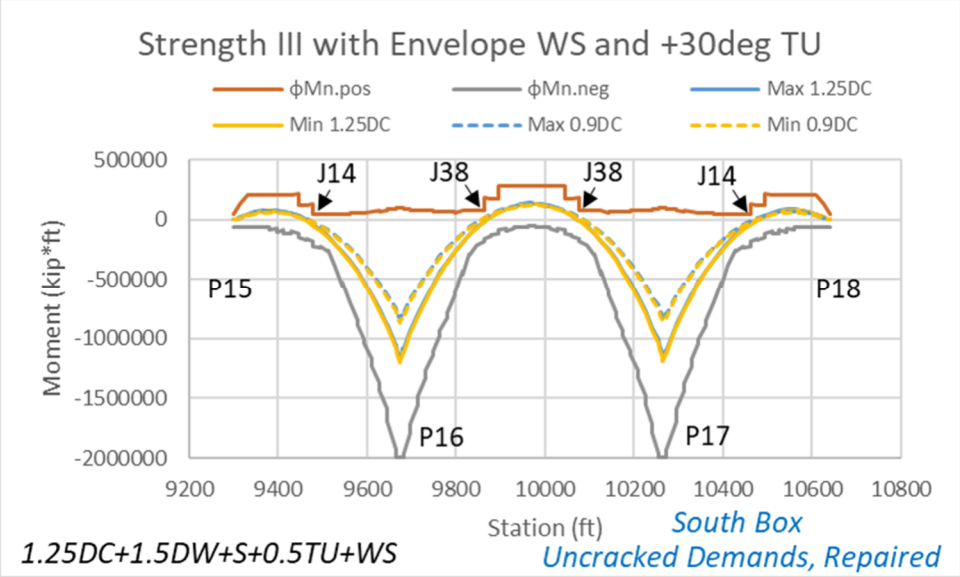
Strength III Moments – Analysis with the Uncracked Model Demands

Results are provided for both the North windward and South leeward boxes.



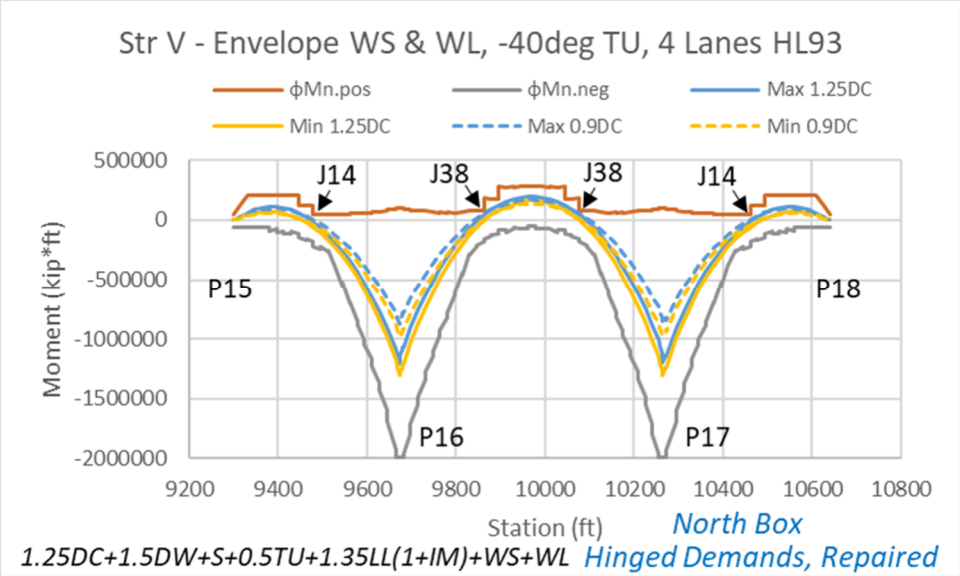


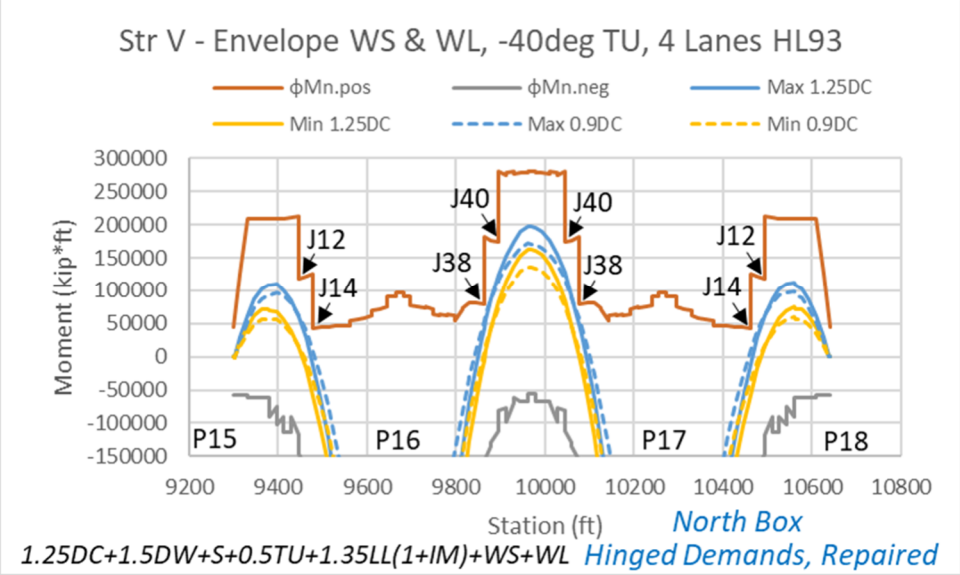

Strength III with Envelope WS and +30deg TU
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
500000
0
-500000
-1000000
-1500000
-2000000
J14
J38
J38
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
North Box
1.25DC+1.5DW+S+0.5TU+WS
Uncracked Demands, Repaired

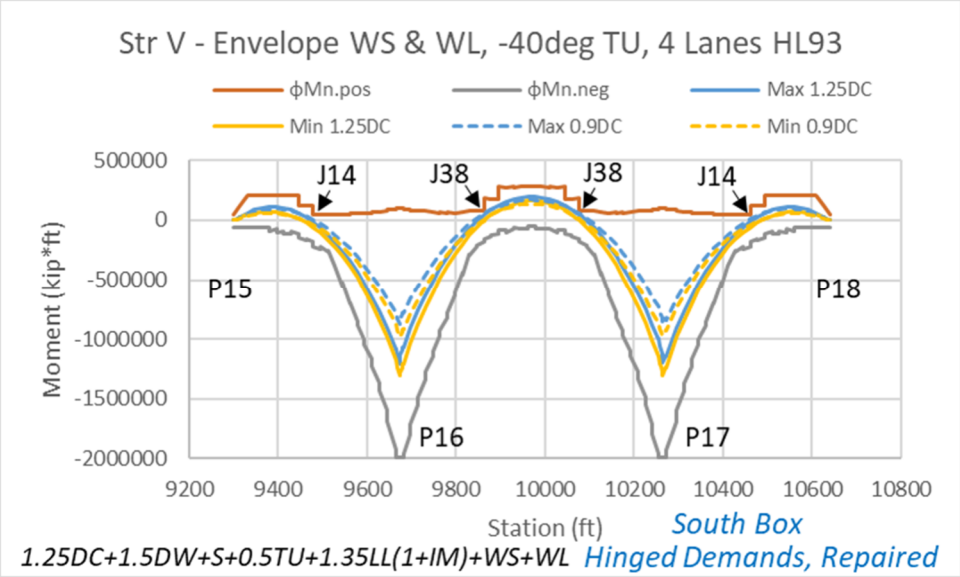

Strength III with Envelope WS and +30deg TU
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
500000
0
-500000
-1000000
-1500000
-2000000
J14
J38
J38
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
South Box
1.25DC+1.5DW+S+0.5TU+WS
Uncracked Demands, Repaired

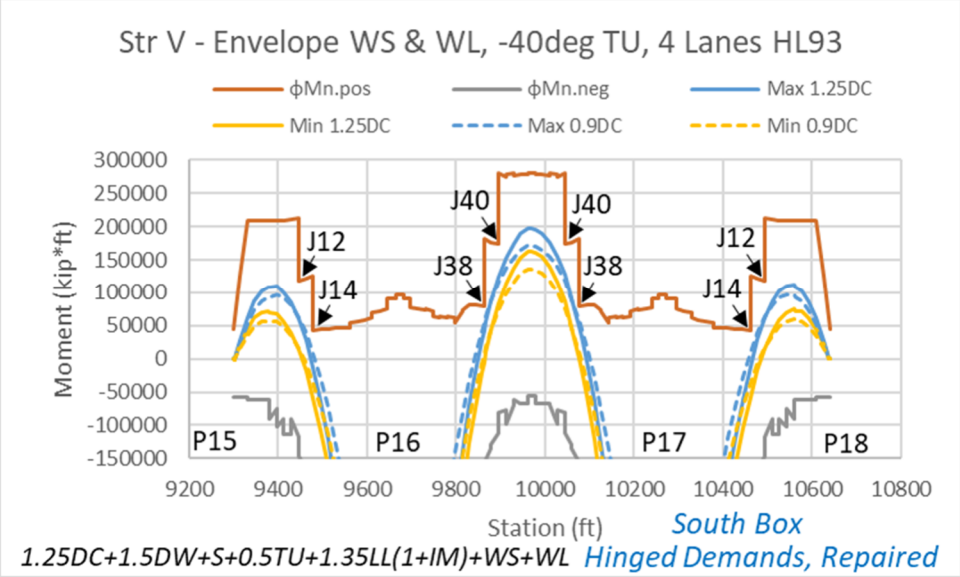
Strength V Moments – Analysis with the Hinged Model Demands

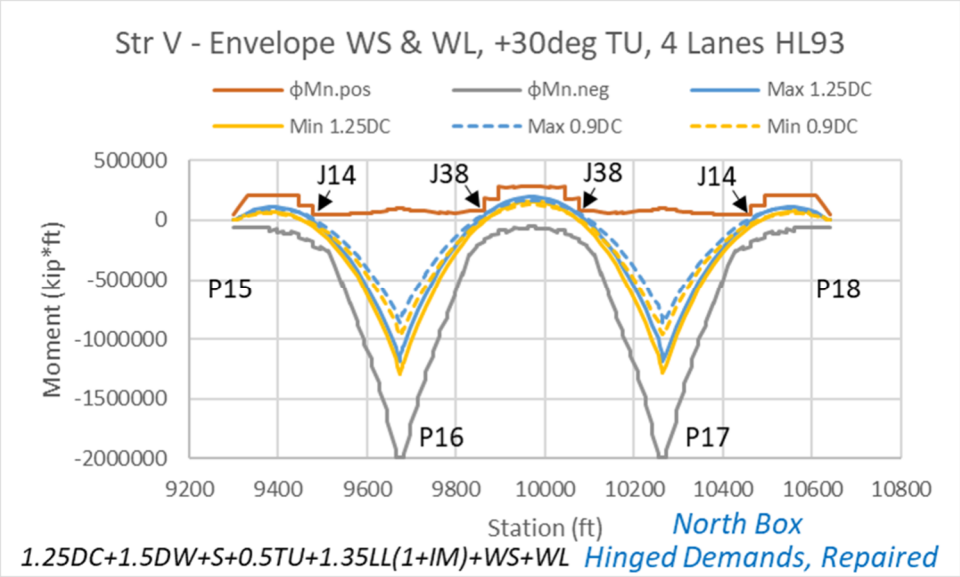
Results are provided for both the North windward and South leeward boxes. Procedures used to determine the envelope effects of WS and WL were previously discussed. A symmetric arrangement of four lanes per box was considered to maximize live load effects.

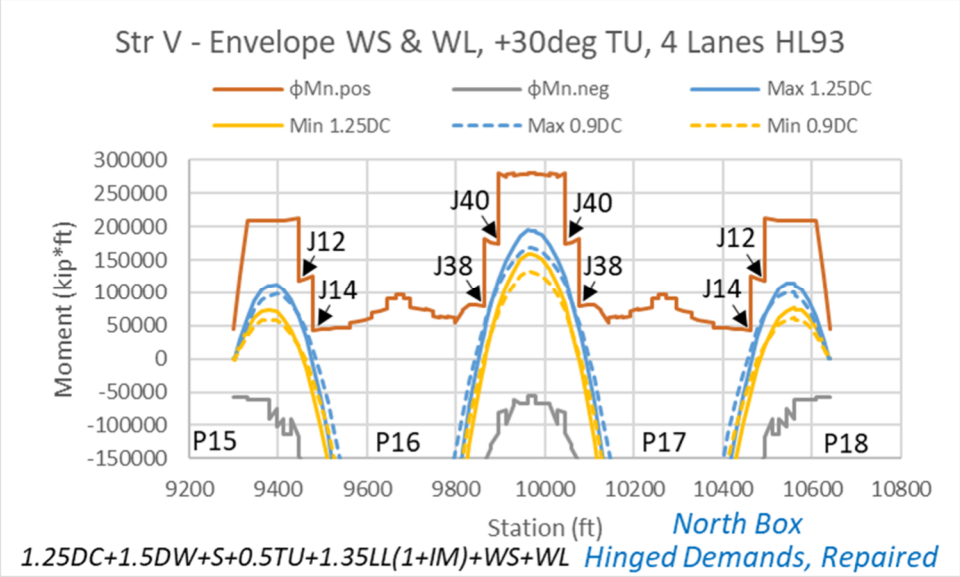


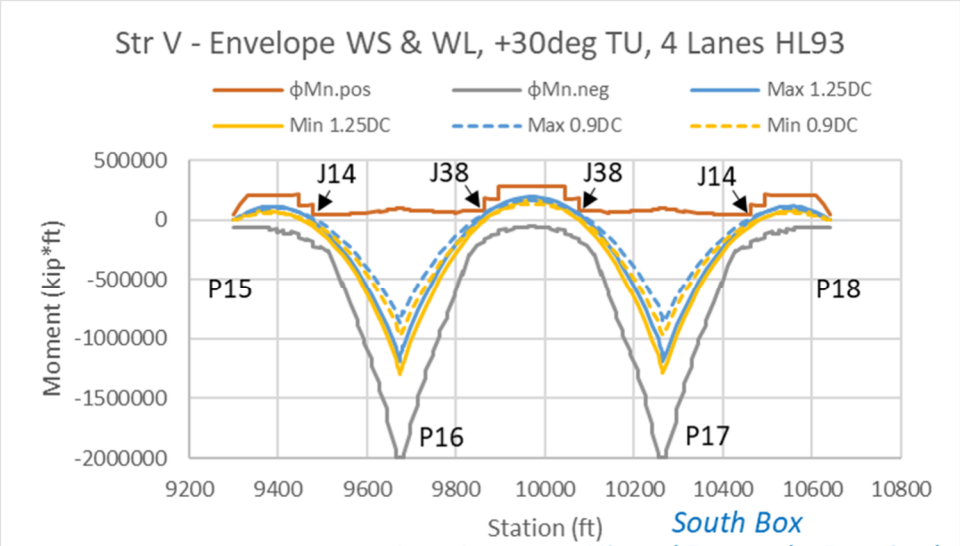


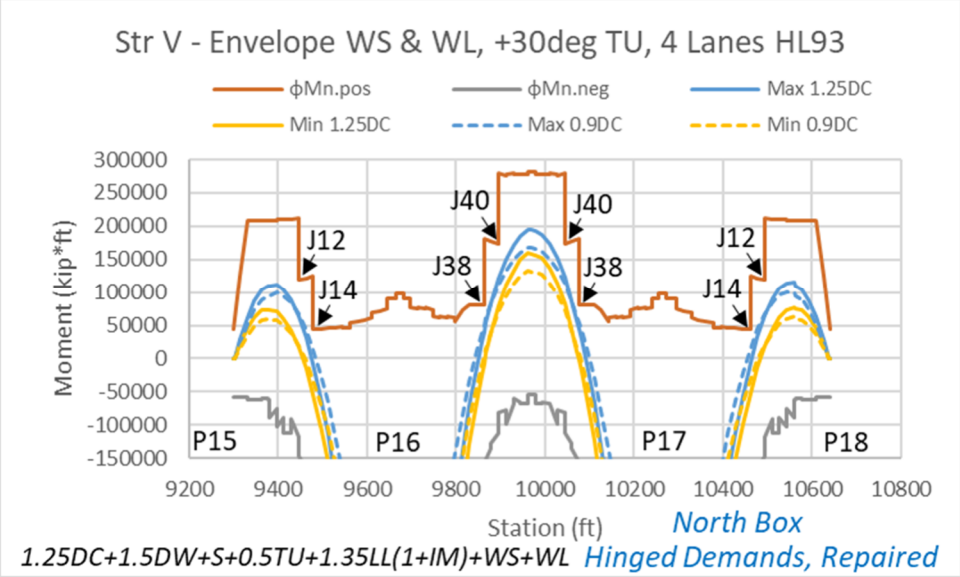

Str V - Envelope WS & WL, -40deg TU, 4 Lanes HL93
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
500000
0
-500000
-1000000
-1500000
-2000000
J14
J38
J38
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
South Box
1.25DC+1.5DW+S+0.5TU+1.35LL(1+IM)+WS+WL
Hinged Demands, Repaired


Str V - Envelope WS & WL, -40deg TU, 4 Lanes HL93
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
300000
250000
200000
150000
100000
50000
0
-50000
-100000
-150000
J40
J40
J12
J12
J38
J38
J14
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
South Box
1.25DC+1.5DW+S+0.5TU+1.35LL(1+IM)+WS+WL
Hinged Demands, Repaired


Str V - Envelope WS & WL, +30deg TU, 4 Lanes HL93
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
500000
0
-500000
-1000000
-1500000
-2000000
J14
J38
J38
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
North Box
1.25DC+1.5DW+S+0.5TU+1.35LL(1+IM)+WS+WL
Hinged Demands, Repaired

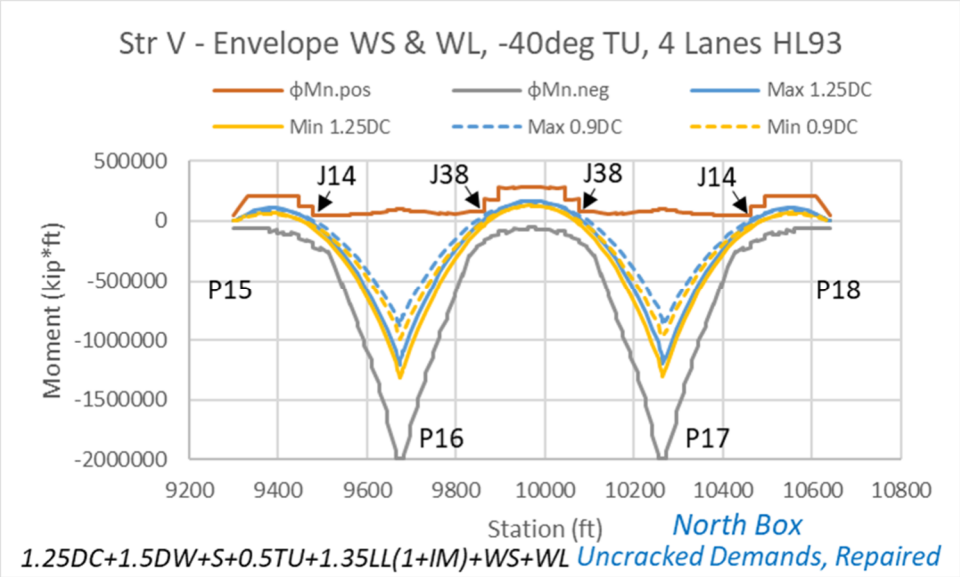

Str V - Envelope WS & WL, +30deg TU, 4 Lanes HL93
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
300000
250000
200000
150000
100000
50000
0
-50000
-100000
-150000
J40
J40
J12
J12
J38
J38
J14
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
North Box
1.25DC+1.5DW+S+0.5TU+1.35LL(1+IM)+WS+WL
Hinged Demands, Repaired

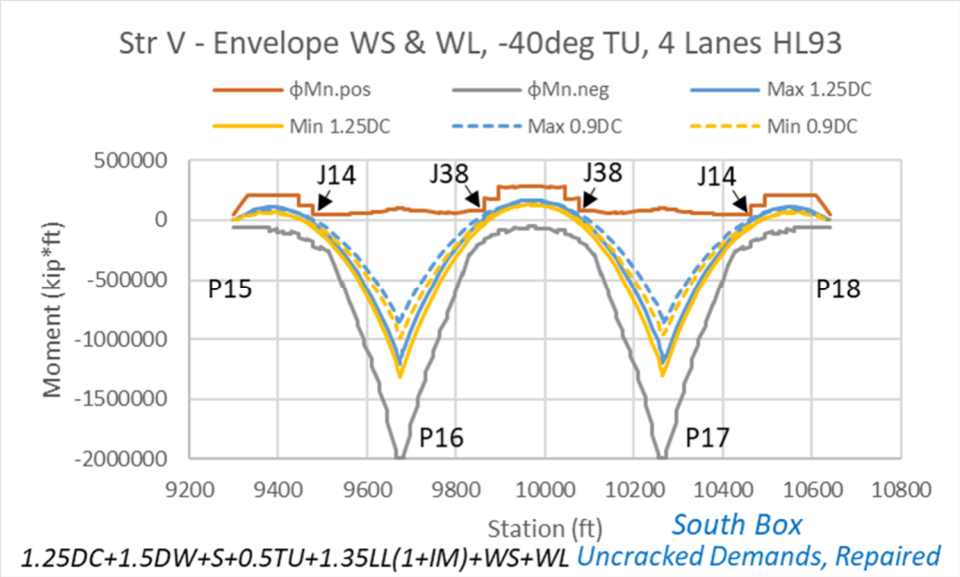

Str V - Envelope WS & WL, +30deg TU, 4 Lanes HL93
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
500000
0
-500000
-1000000
-1500000
-2000000
J14
J38
J38
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
South Box
1.25DC+1.5DW+S+0.5TU+1.35LL(1+IM)+WS+WL
Hinged Demands, Repaired

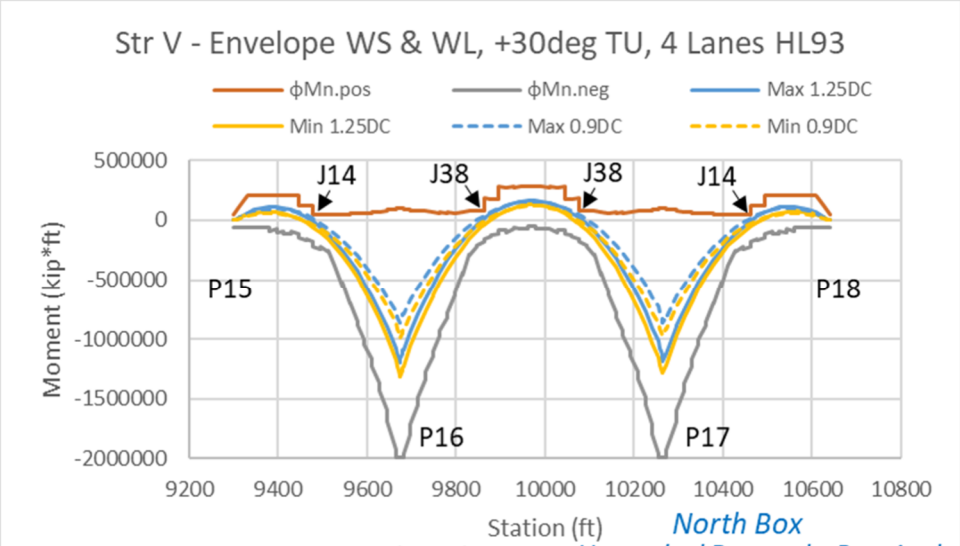

Str V - Envelope WS & WL, +30deg TU, 4 Lanes HL93
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
300000
250000
200000
150000
100000
50000
0
-50000
-100000
-150000
J40
J40
J12
J38
J38
J12
J14
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
North Box
1.25DC+1.5DW+S+0.5TU+1.35LL(1+IM)+WS+WL
Hinged Demands, Repaired

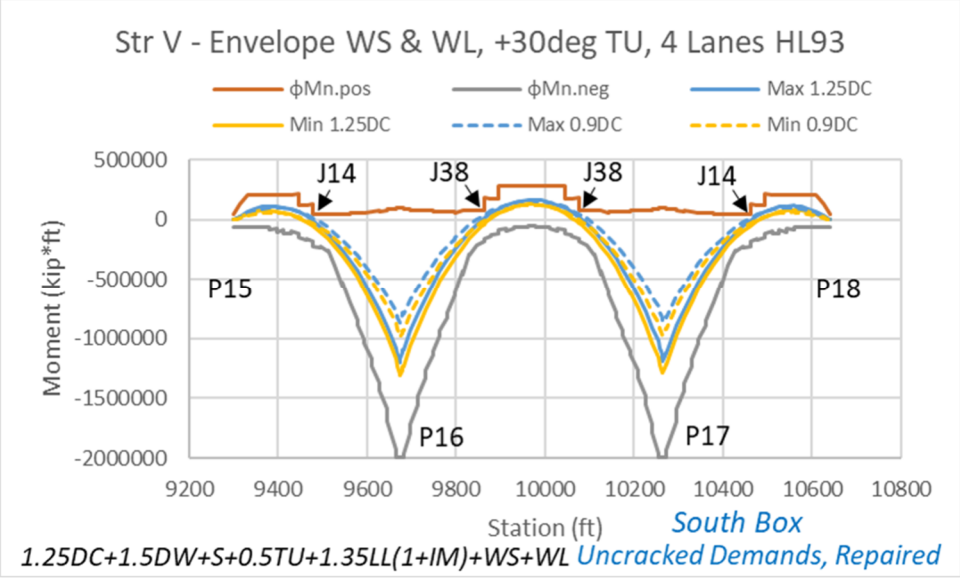
Strength V Moments – Analysis with the Uncracked Model Demands

Results are provided for both the North windward and South leeward boxes.






Str V - Envelope WS & WL, +30deg TU, 4 Lanes HL93
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
500000
0
-500000
-1000000
-1500000
-2000000
J14
J38
J38
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
North Box
1.25DC+1.5DW+S+0.5TU+1.35LL(1+IM)+WS+WL Uncracked Demands, Repaired

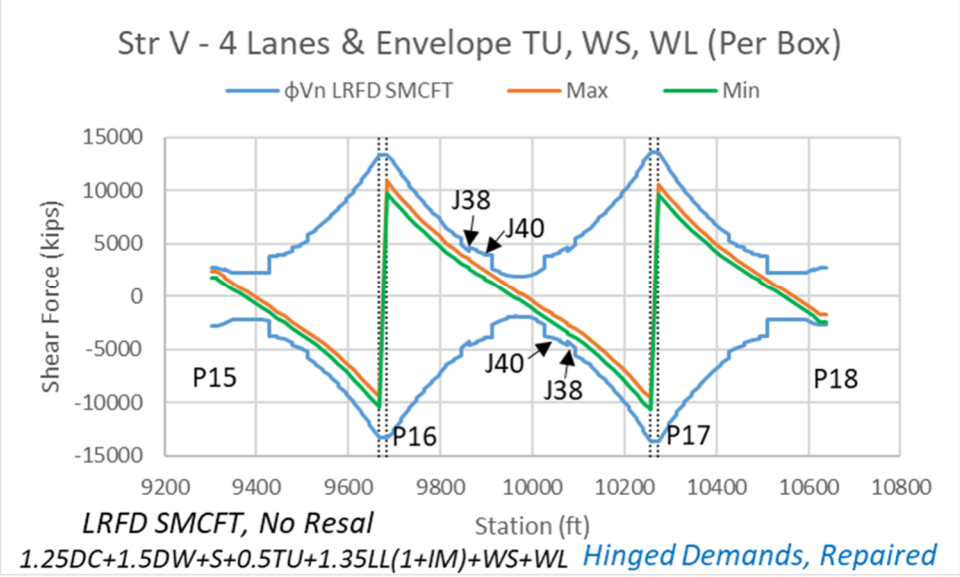

Str V - Envelope WS & WL, +30deg TU, 4 Lanes HL93
ϕMn.pos
ϕMn.neg
Max 1.25DC
Min 1.25DC
Max 0.9DC
Min 0.9DC
Moment (kip*ft)
500000
0
-500000
-1000000
-1500000
-2000000
J14
J38
J38
J14
P15
P16
P17
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
South Box
1.25DC+1.5DW+S+0.5TU+1.35LL(1+IM)+WS+WL Uncracked Demands, Repaired

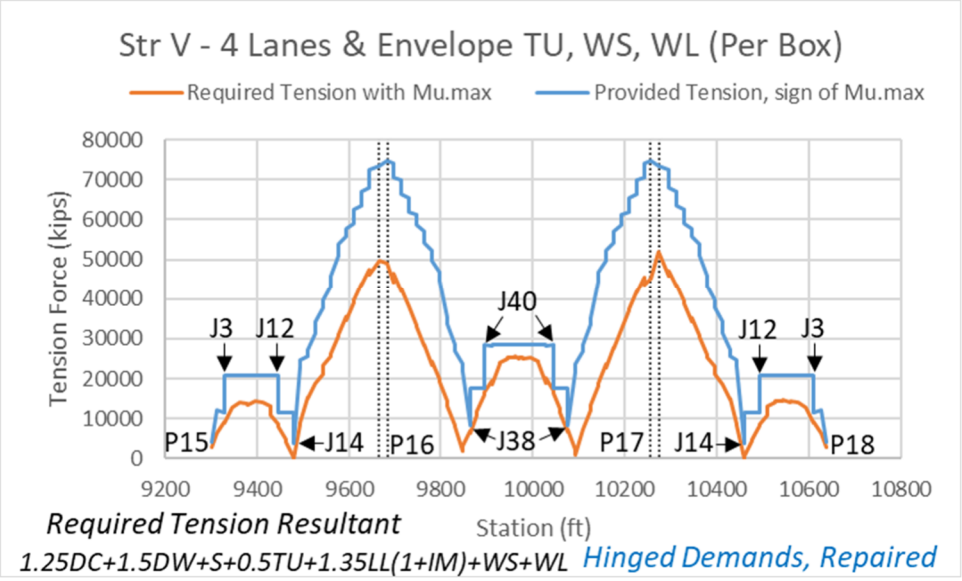
Strength V Shear – Analysis with the Hinged Model Demands

An envelope of north and south box load effects and uniform thermal (TU) cases was considered to determine the max/min shear and max/min moments. Live load axial forces were taken as those in equilibrium with max/min influence shear results for calculation of the factored axial force.

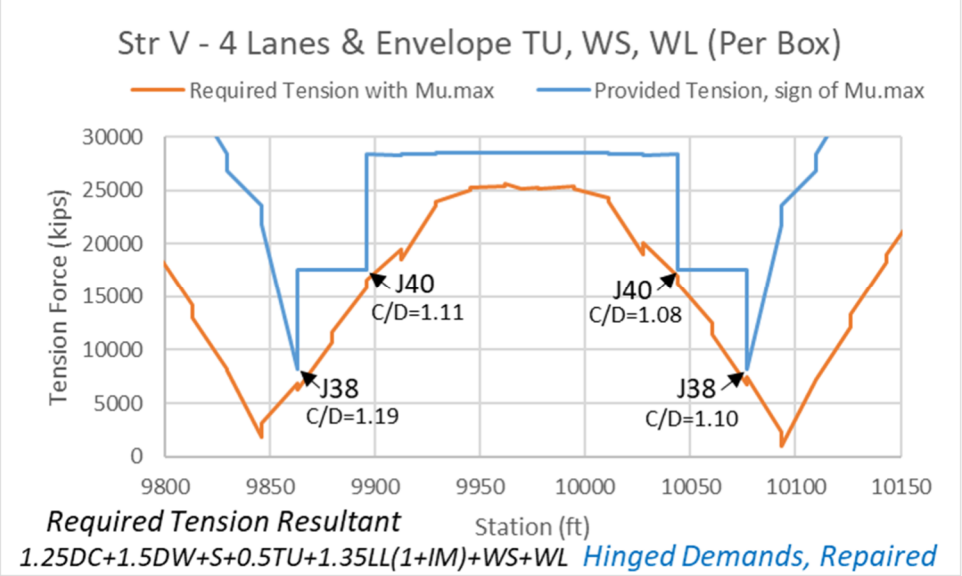
	Vu with Mu.max			
	Original	Tension Resultant		Reduced
Location	$\phi V_n/V_u$	$T_{provided} / T_{required}$	ΔT (kips)	$\phi V_n^R/V_u$
P15 J2	1.15	1.87	-	-
P15 J3	1.37	1.25	-	-
P16 J38	1.21	1.19	-	-
P16 J40	1.64	1.11	-	-
P16 J41	1.55	1.20	-	-
P17 J41	1.40	1.50	-	-
P17 J40	1.61	1.08	-	-
P17 J38	1.23	1.10	-	-
P18 J3	1.36	1.24	-	-
P18 J2	1.13	1.86	-	-

As shown, Strength V shear demands do not control over those from Strength I.

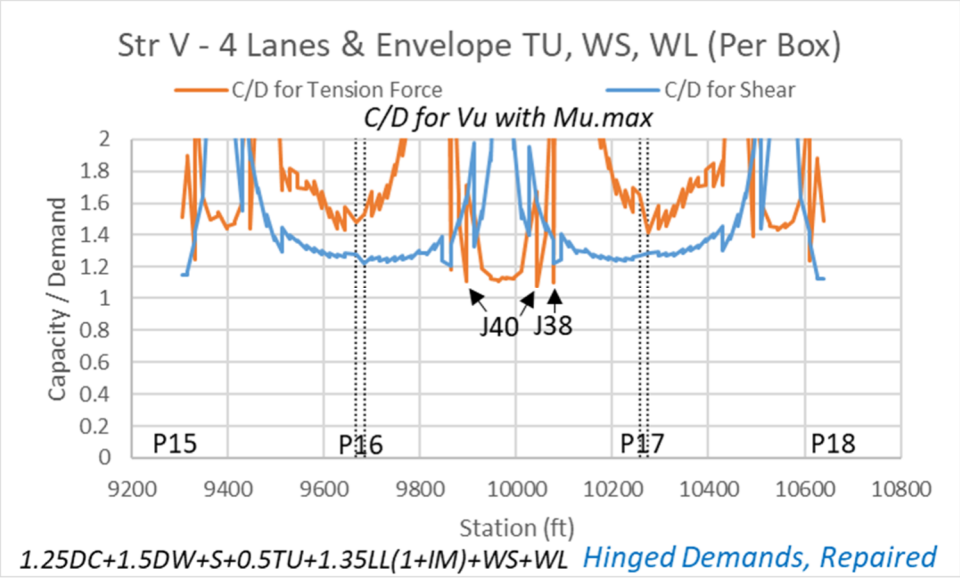




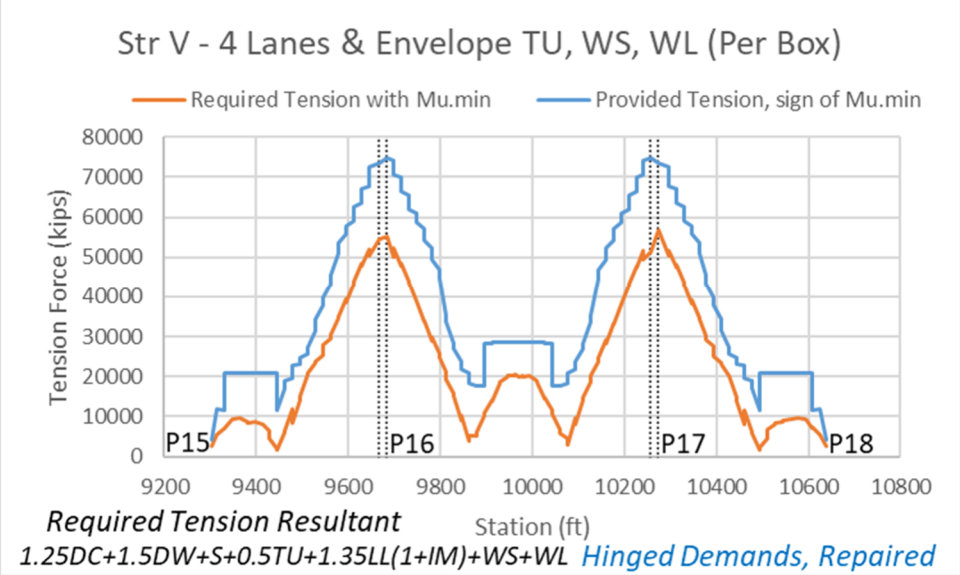
M.max = most positive



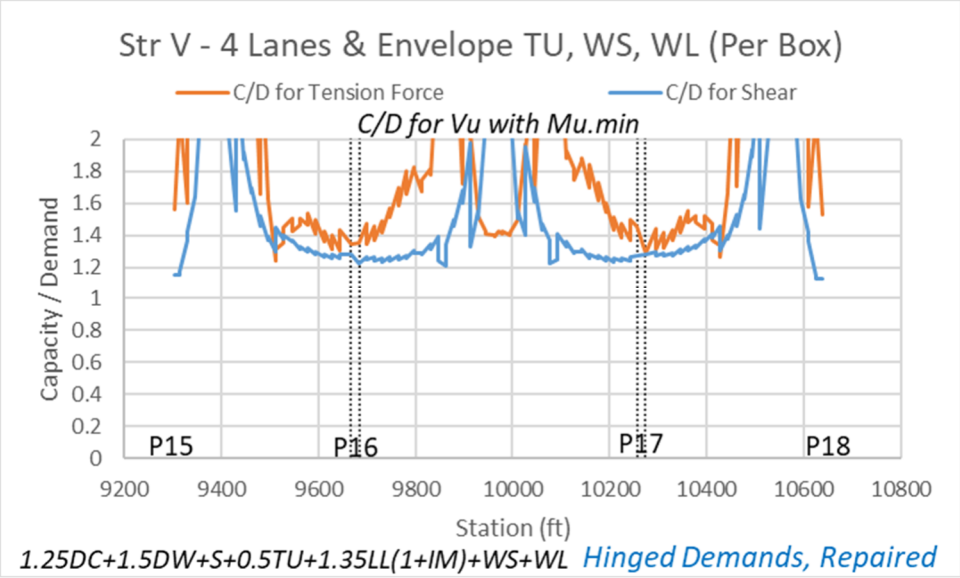
M.max = most positive



M.max = most positive



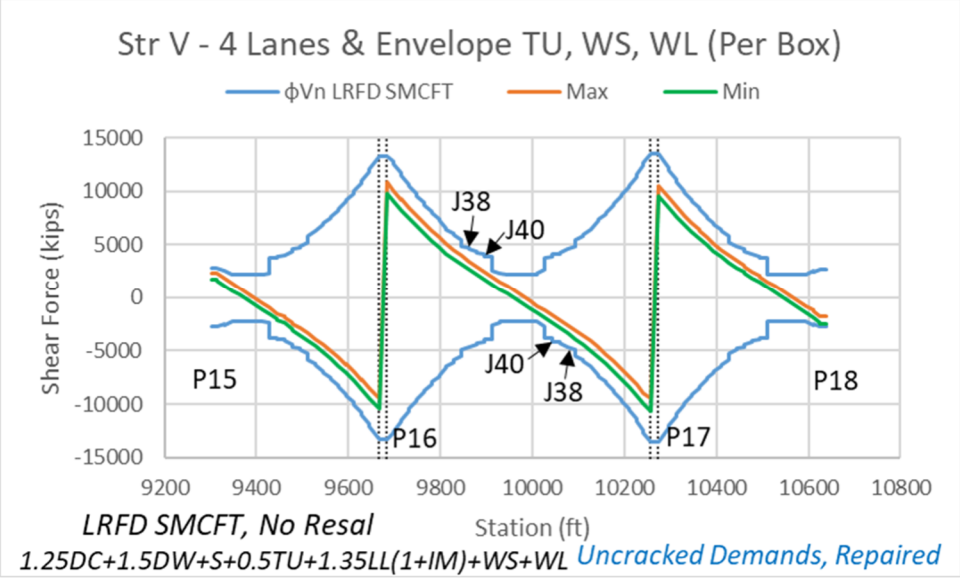
M.min = most negative

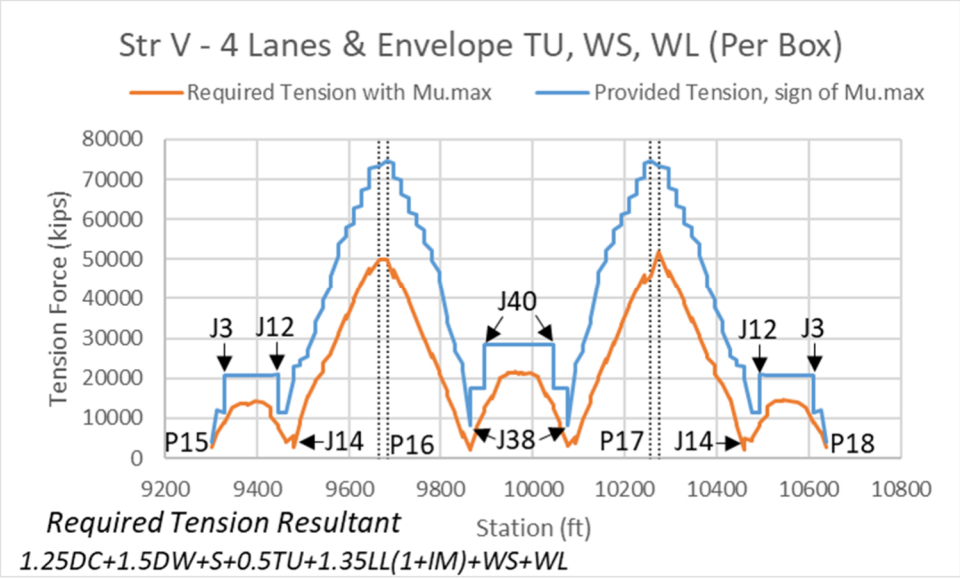


M.min = most negative

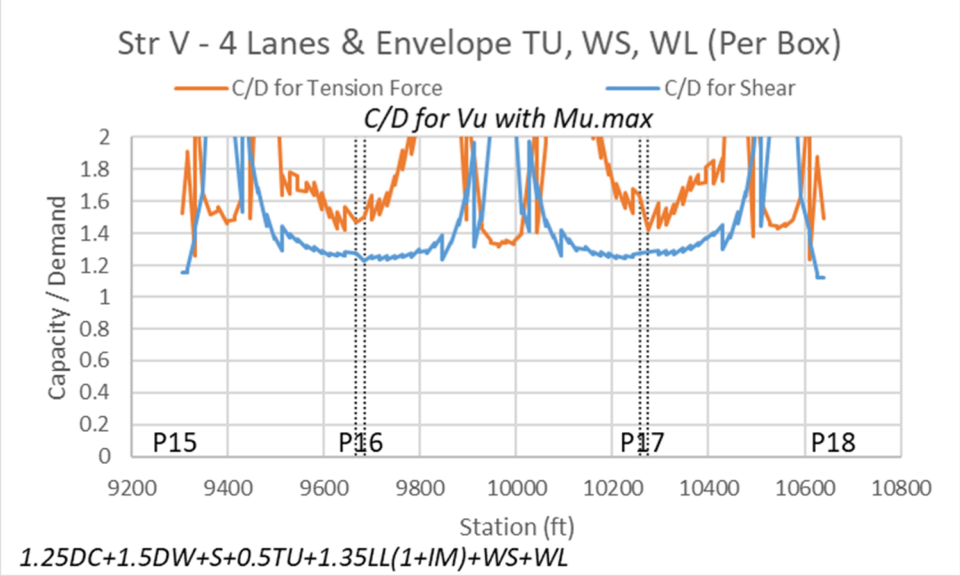
Strength V Shear – Analysis with the Uncracked Model Demands

An envelope of north and south box load effects and uniform thermal (TU) cases was considered to determine the max/min shear and max/min moments. Live load axial forces were taken as those in equilibrium with max/min influence shear results for calculation of the factored axial force.

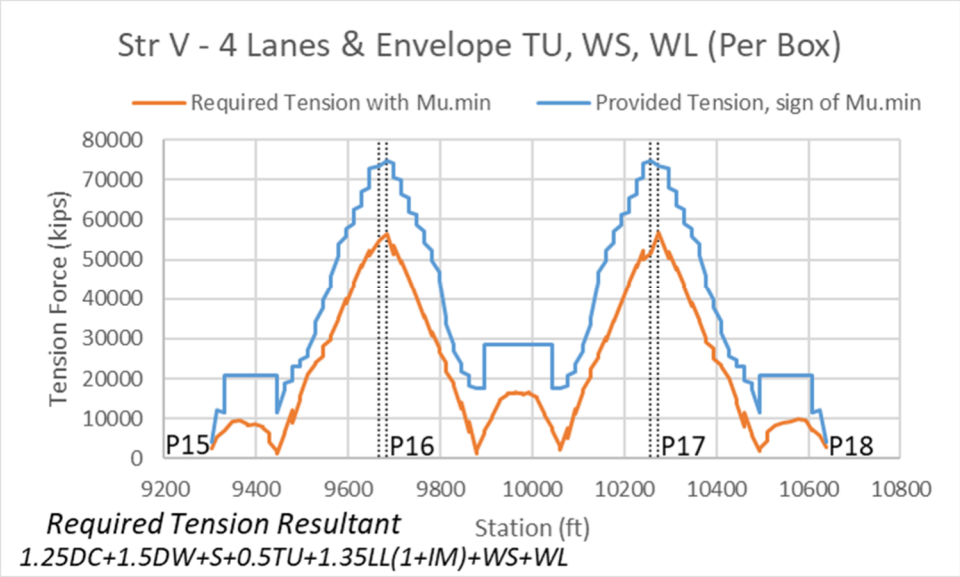




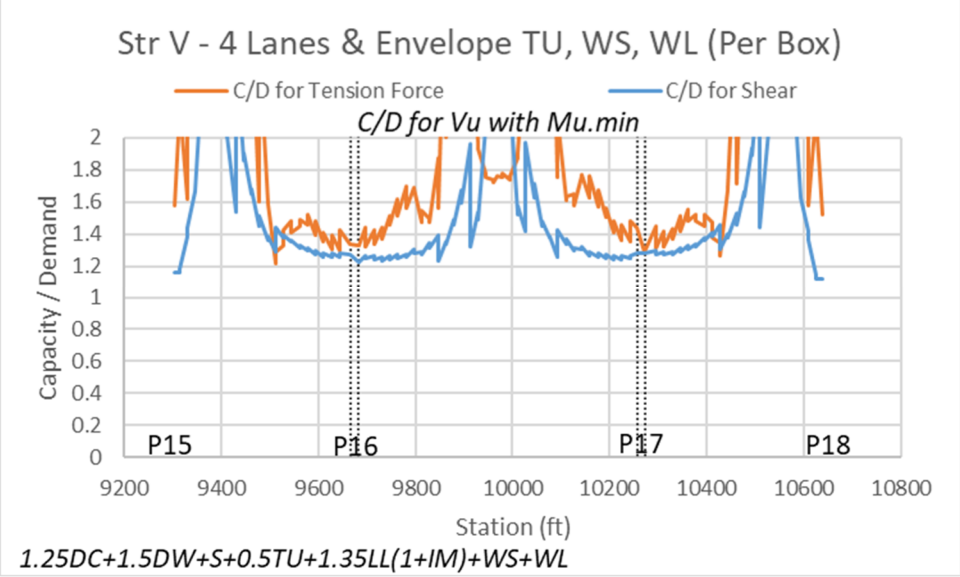
M.max = most positive



M.max = most positive



M.min = most negative



M.min = most negative

APPENDIX I

PRELIMINARY SEISMIC ASSESSMENT

WSHB Preliminary Seismic Assessment (October 15, 2020)

This document presents results from the preliminary seismic assessment of the West Seattle High-Rise Bridge (WSHB) in support of the Cost-Benefit Analysis (CBA). This work-in-progress document aims to define the seismic evaluation process, assumptions made pending finalized geotechnical recommendations, initial results relative to WSDOT BDM requirements for an Essential or Critical structure, and remaining work needed to complete the assessment. Special attention is given to capacity design requirements for the superstructure and foundations necessary to develop the intended column plastic hinging mechanism. Recommendations from a site-specific ground motion study included in the draft Geotechnical report dated 10/5/2020 were evaluated for comparison to the preliminary Site Class D spectra at both the 210yr FEE (Functional Evaluation Earthquake) and 975yr SEE (Safety Evaluation Earthquake) levels. The site-specific ground motion was found to amplify the demands in the period range of interest producing the governing results for the structure. In this document, highlights are used to draw attention to assumptions and items pending further investigation.

1.0 SEISMIC EVALUATION PROCESS

There are three distinct activities involved in the seismic evaluation process: linear demand analyses, nonlinear static pushover capacity analyses, and evaluation for capacity protection. An overview of each step is provided below as it relates to the WSHB seismic assessment.

A linear demand analyses was performed using response spectrum analysis (RSA) to define the deformation demands on the structure. All results discussed herein are relative to the site-specific ground motions for the 210yr FEE (Functional Evaluation Earthquake) and 975yr SEE (Safety Evaluation Earthquake) recommended by Shannon and Wilson. The site-specific ground motions were found to amplify the demands in the period range of interest producing the governing results relative to the preliminary Site Class D spectra. The approach spans were modeled to their first longitudinal expansion joint to include their effects on the WSHB; however, the assessment was limited to Piers 15, 16, 17, and 18 supporting the main structure. Both tension and compression models were considered to bound the effects of expansion joint and longitudinal seismic retainer behavior at Piers 15 and 18.

For the 975yr SEE, separate demand displacements were computed using 6x6 foundation spring matrices for liquefied and non-liquefied soil conditions consistent with the behavior at formation of a plastic mechanism. Cases including and excluding vertical acceleration effects were evaluated to determine the governing deformation demands on the piers and forces applicable to the superstructure assessment.

For the 210yr FEE, only non-liquefied soil conditions were considered, as will be discussed. Vertical acceleration effects were not considered in the FEE, since the analyses focuses on the potential for hinging in the columns.

A nonlinear static pushover capacity analysis (NLPO) was performed to define the displacement capacity of the structure and to determine the displacement ductility demands relative to WSDOT BDM limits for an Essential or Critical structure. This analysis included the nonlinear behavior of the column plastic hinges with expected material properties. A nonlinear soil spring

model was used to include the effects of foundation deformations on column plastic hinge rotational demands using the recommended liquefied and non-liquefied lateral soil parameters. This model was used to define the equivalent linear 6x6 foundation matrices used in RSA. Assumptions for pile axial stiffness were made prior to receiving recommendations for T-z and Q-z springs included in the draft Geotechnical report dated 10/5/2020. These assumptions are expected to have little impact on the results, as will be discussed.

Finally, an evaluation for capacity protection is performed to verify that the structure can develop the intended column plastic hinging mechanism. In this step, the strength of non-ductile elements is assessed for forces associated with flexural overstrength of the column plastic hinges. This was accomplished using the nonlinear static pushover (NLPO) model with overstrength hinge properties, thus a complete set of demands were obtained.

An assessment for longitudinal flexural overstrength demands indicated that the existing superstructure has vulnerabilities at the distressed Joints 14 and 38. The overstrength superstructure demands were then compared to the strength of the superstructure including the work-in-progress Phase II PT layout. The superstructure was also assessed for combinations of lateral and vertical seismic demands corresponding to the 100-30 directional combination rule.

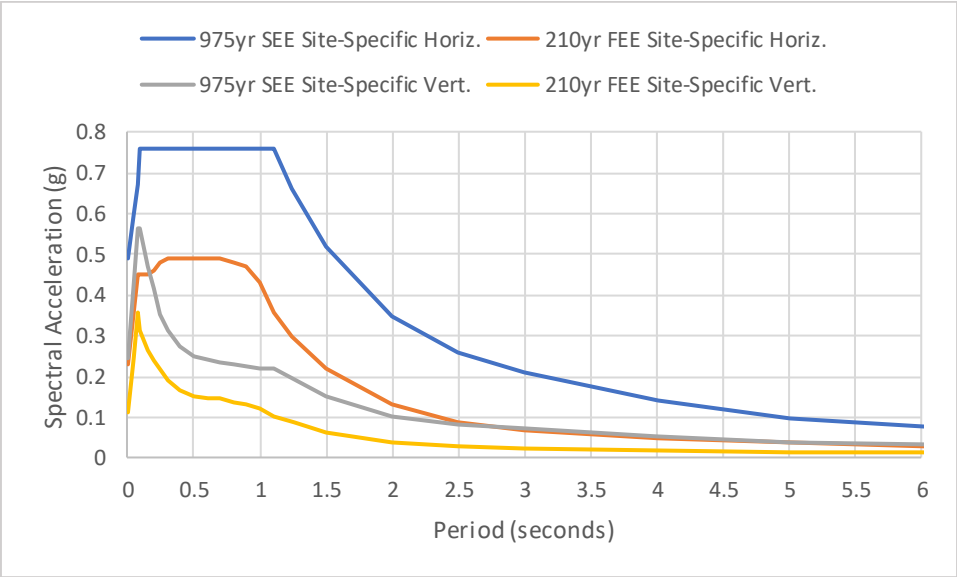
Foundation demands were evaluated using the combination of recommended lateral soil forces and overstrength demands. The lateral soil forces recommended by Shannon and Wilson represent pile loading required to maintain the code-based factor of safety for slope stability.

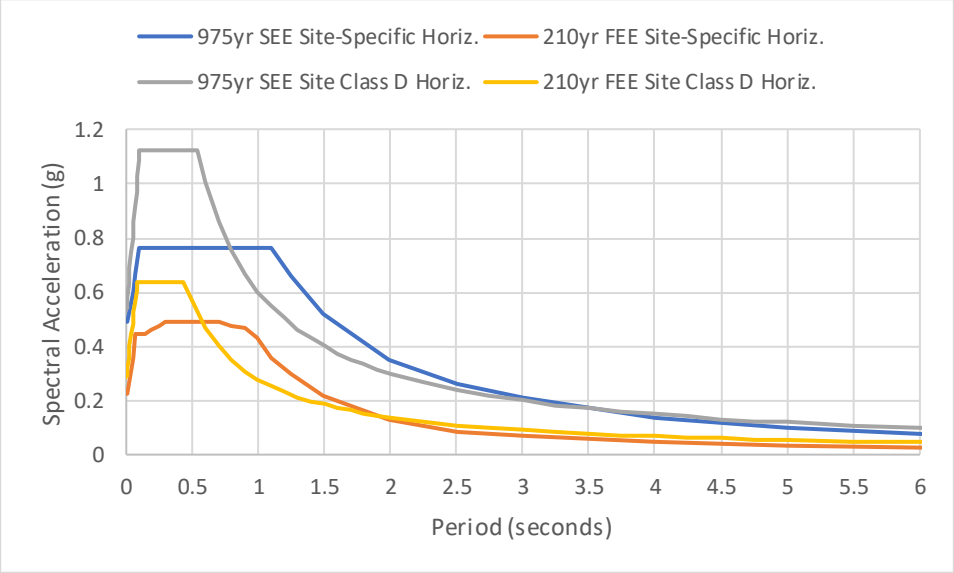
The following items still require a detailed assessment for overstrength demands: pile cap footing (piles and pile-head connections have been evaluated), column to footing joint, column to pier table joint, pier table diaphragm. The original design was stated to include an evaluation for capacity protection, but as the discussed superstructure seismic vulnerabilities show, additional verification is warranted. An initial conservative assessment for the pile cap footing demands was used to generate potential costs for footing strengthening among the retrofit alternatives.

2.0 PRELIMINARY SITE CLASS D AND SITE-SPECIFIC SPECTRA

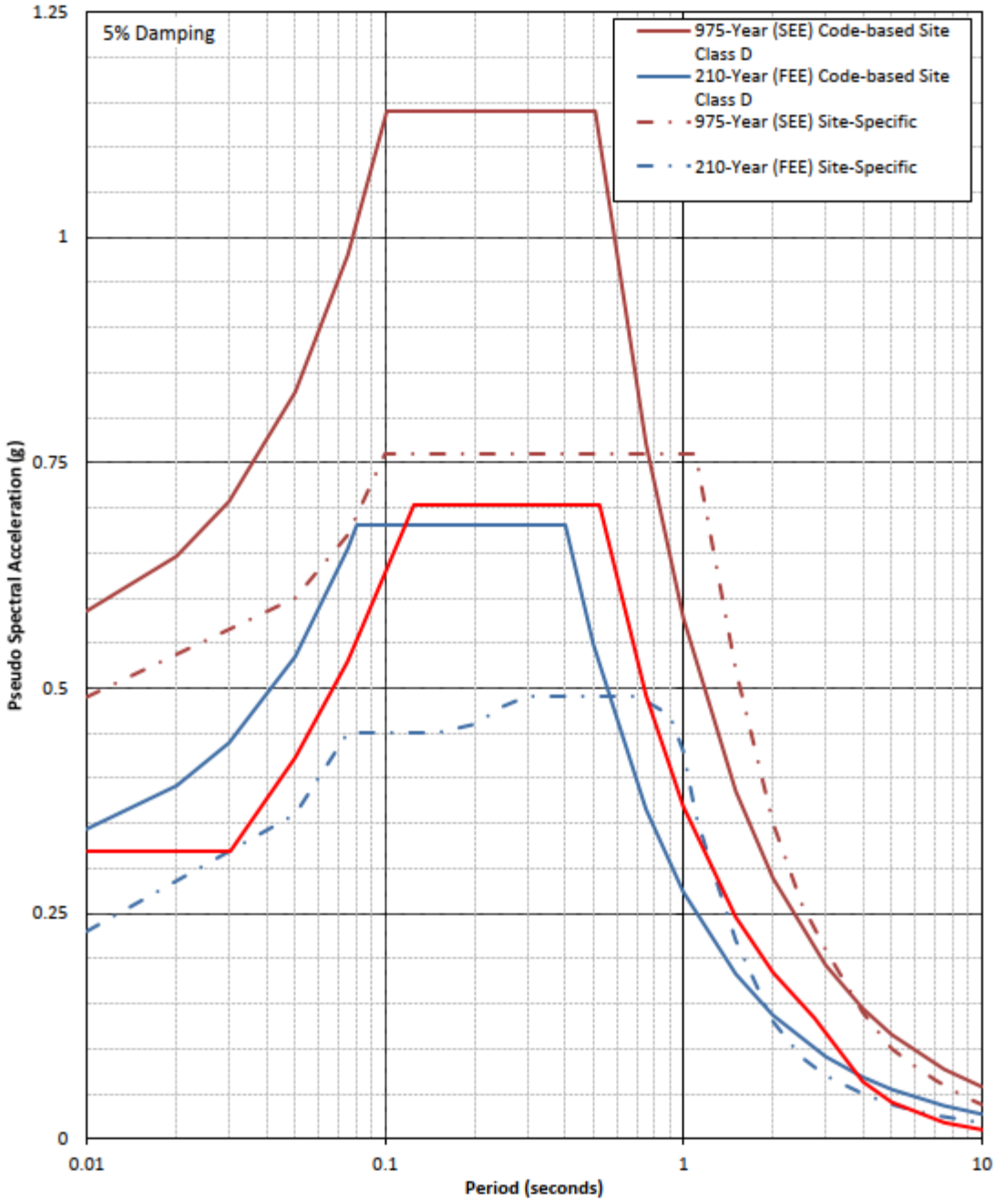
Assessment results discussed herein are relative to the site-specific 210yr FEE and 975yr SEE 5% damped horizontal and vertical spectra provided by Shannon and Wilson. The site-specific ground motions were found to amplify the demands in the period range of interest producing the governing results relative to the preliminary Site Class D spectra. The site-specific ground motion was noted to include modifications for basin effects, which increased demands beyond a period of 1 second. The fundamental longitudinal period of the structure is between 2.6 and 2.8 seconds depending on tension/compression model behavior, and 2.4 seconds for transverse.

The following site-specific response spectrum were considered in the assessment. The recommended vertical spectra were provided on 10/6/2020 separately from the draft Geotechnical report dated 10/5/2020.





WSB-Design Spectra.xlsm 10/6/2020



NOTES

FEE = functional evaluation earthquake; g = standard gravitational acceleration; SDOT = Seattle department of transportation; SEE = safety evaluation earthquake

SDOT
West Seattle High-Rise Bridge
Seattle, Washington

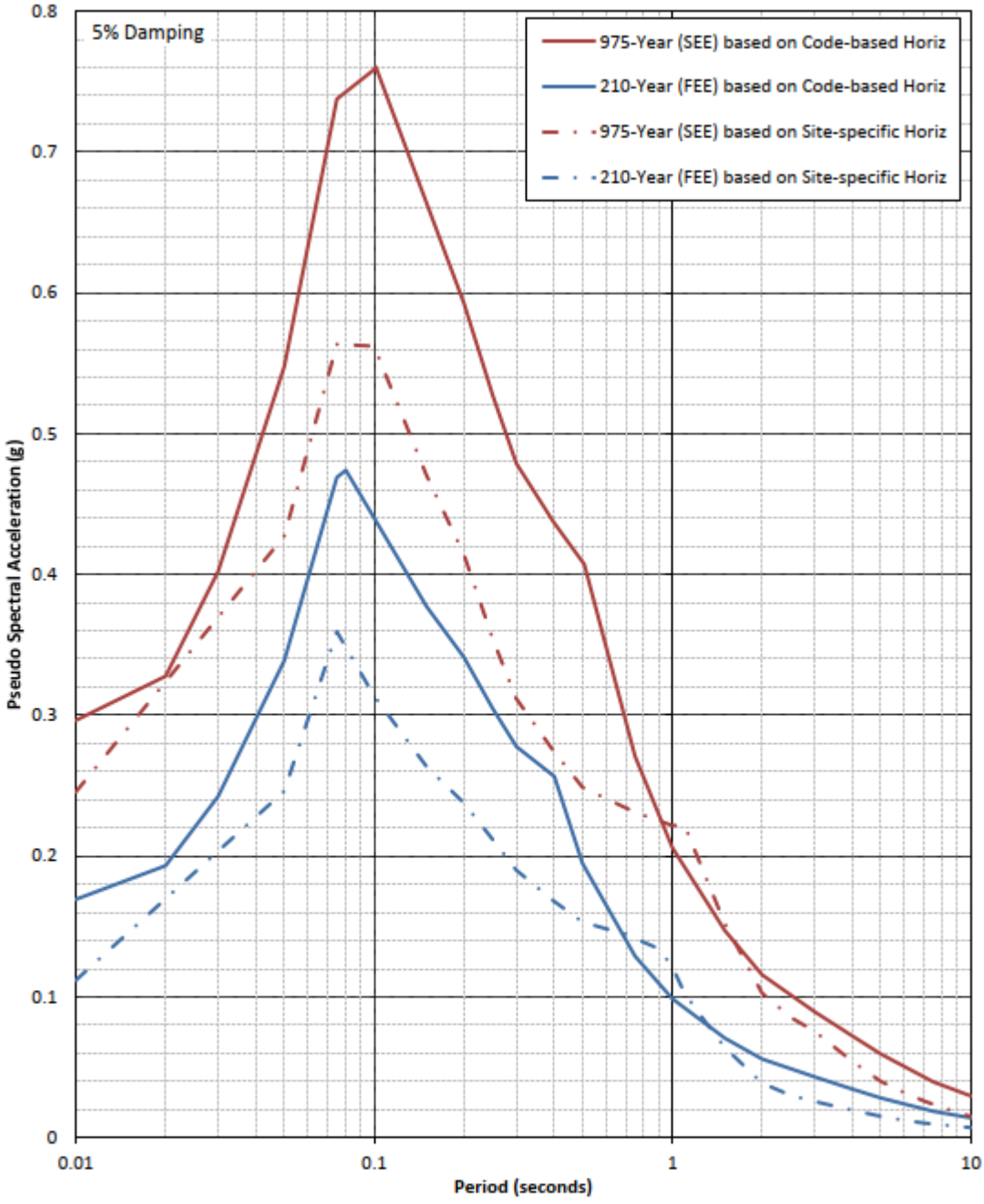
RECOMMENDED SITE-SPECIFIC HORIZONTAL DESIGN SPECTRA

October 2020 105492-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 3

WSB-Design Spectra.xlsm 10/6/2020



NOTES

FEE = functional evaluation earthquake; g = standard gravitational acceleration; SDOT = Seattle department of transportation; SEE = safety evaluation earthquake

We applied a Site Class D V/H ratio to the code-based ground surface horizontal design spectrum. The resulting vertical spectrum would correspond to a non-liquefied soil profile.

We applied a Site Class E V/H ratio to the site-specific ground surface horizontal design spectrum. The resulting vertical spectrum would correspond to an elevated pore pressure soil profile.

SDOT
West Seattle High-Rise Bridge
Seattle, Washington

RECOMMENDED VERTICAL DESIGN SPECTRA

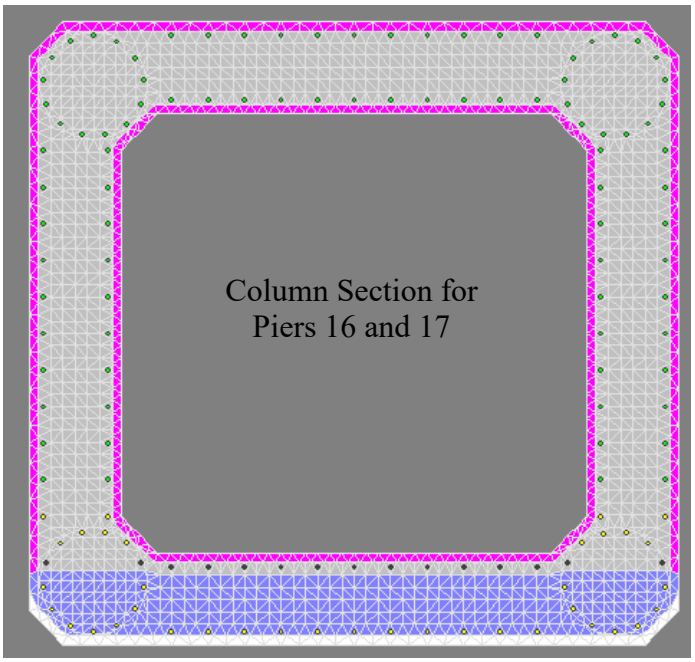
October 2020 105492-001

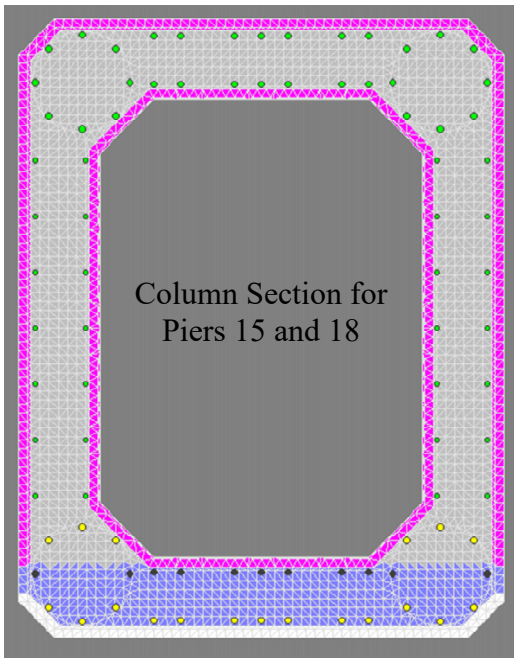
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG.

3.0 COLUMN MOMENT-CURVATURE ANALYSIS

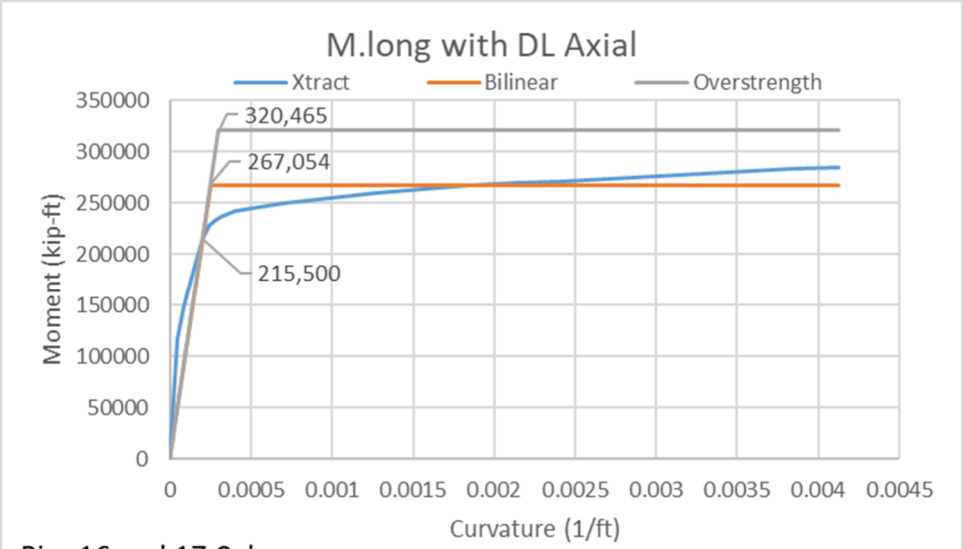
Moment-curvature analyses were performed for the columns of Piers 15-18 in Xtract. Expected material properties per the AASHTO Seismic Guide Specifications (SGS) were considered. Sample column cross sections appear in the following figures. Separate regions of confinement were considered for core concrete in the rectangular tied walls and corner boundary elements with circular hoops. SGS procedures were followed to bilinearize the moment-curvature response and compute the plastic moment capacity, Mp. The columns have modern confinement details in the end regions suitable for ductile response. The ultimate curvature was taken as first exceedance of the SGS reduced ultimate steel tensile strain or the Mander, Priestley, and Park (1988) ultimate concrete compressive strain. Expected material properties considered in the assessment are documented in the following section from the Xtract output.



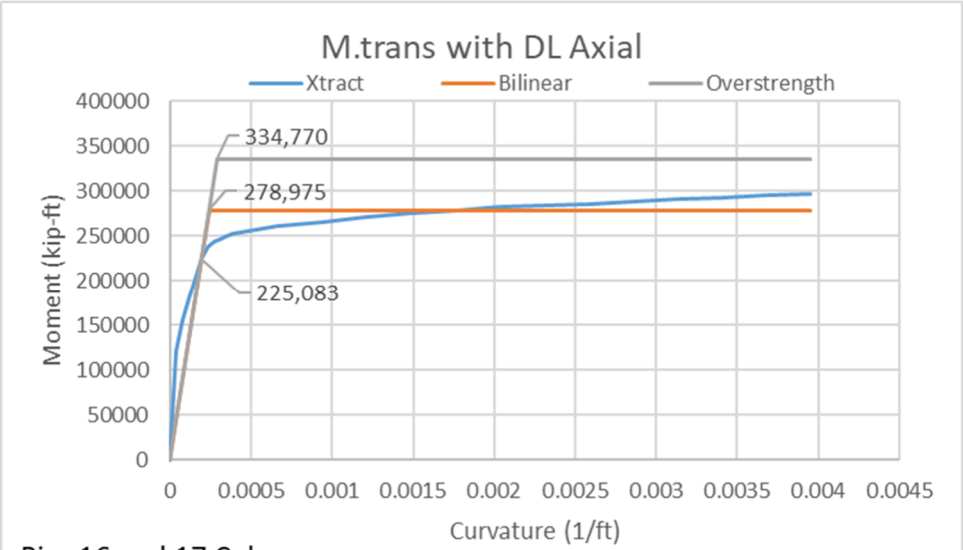


Sample moment-curvature analysis results at the dead load axial force for columns of Piers 15-18 appear in the following figures. Note that both a bilinear approximation using expected properties and overstrength per SGS is presented. The SGS overstrength factor of 1.2 is applicable due to the use of A706 Gr60 longitudinal reinforcement. It is unnecessarily conservative to consider the higher FHWA SRM overstrength factor of 1.4 due to bounded material properties with the A706 steel designation.

In the following, note M.long refers to longitudinal response and is actually bending about the transverse axis of the bridge and vise versa for M.trans. Moment-curvature results at the dead load axial force were used to establish effective cracked section properties in the demand analysis per the SGS.



Pier 16 and 17 Columns



Pier 16 and 17 Columns

Moment-Curvature Anlaysis Results for Piers 16 and 17 Columns

Dead Load Axial Force, P.dl			19060	kips		
f'ce			6.5	ksi		
Column Agross			21460	in^2		
P.dl / (f'ce*Agross)			0.137	value within the typical range		
Ece at f'ce			5008	ksi	721152	ksf
EIeff at P.dl for M.long			1069585070	kip-ft^2		
Ieff at P.dl for M.long			1483.16	ft^4	Ratio Ig.long	0.276
EIeff at P.dl for M.trans			1162860784	kip-ft^2		
Ieff at P.dl for M.trans			1612.50	ft^4	Ratio Ig.trans	0.279
	kip	kip-ft	1/ft	1/ft	1/ft	rad

P-Mp Column Interaction for Longituidnal Loads

P/(f'ce*Ag)	Axial, P	Mp.long	ϕy.long	ϕu.long	ϕp.long	θp.long
0.025	3487	157659	0.00024	0.00393	0.00369	0.02287
0.05	6975	182949	0.00024	0.00398	0.00374	0.02316
0.075	10462	207763	0.00024	0.00402	0.00378	0.02342
0.1	13949	232131	0.00024	0.00407	0.00382	0.02370
0.137	19060	267054	0.00025	0.00413	0.00388	0.02405
0.15	20924	279572	0.00025	0.00415	0.00390	0.02419
0.175	24411	302654	0.00026	0.00420	0.00394	0.02444
0.2	27898	325295	0.00026	0.00425	0.00399	0.02471
0.225	31385	347466	0.00027	0.00429	0.00403	0.02496
0.25	34873	368890	0.00025	0.00433	0.00408	0.02527
	kip	kip-ft	1/ft	1/ft	1/ft	rad

P-Mp Column Interaction for Transverse Loads

P/(f'ce*Ag)	Axial, P	Mp.trans	ϕy.trans	ϕu.trans	ϕp.trans	θp.trans
0.025	3487	164582	0.00023	0.00377	0.00353	0.02190
0.05	6975	191006	0.00023	0.00381	0.00358	0.02216
0.075	10462	216952	0.00023	0.00385	0.00362	0.02243
0.1	13949	242433	0.00023	0.00389	0.00366	0.02267
0.137	19060	278975	0.00024	0.00395	0.00371	0.02301
0.15	20924	292060	0.00024	0.00398	0.00373	0.02314
0.175	24411	316190	0.00025	0.00402	0.00377	0.02337
0.2	27898	339879	0.00025	0.00406	0.00381	0.02362
0.225	31385	363123	0.00026	0.00411	0.00385	0.02389
0.25	34873	385389	0.00024	0.00413	0.00389	0.02409

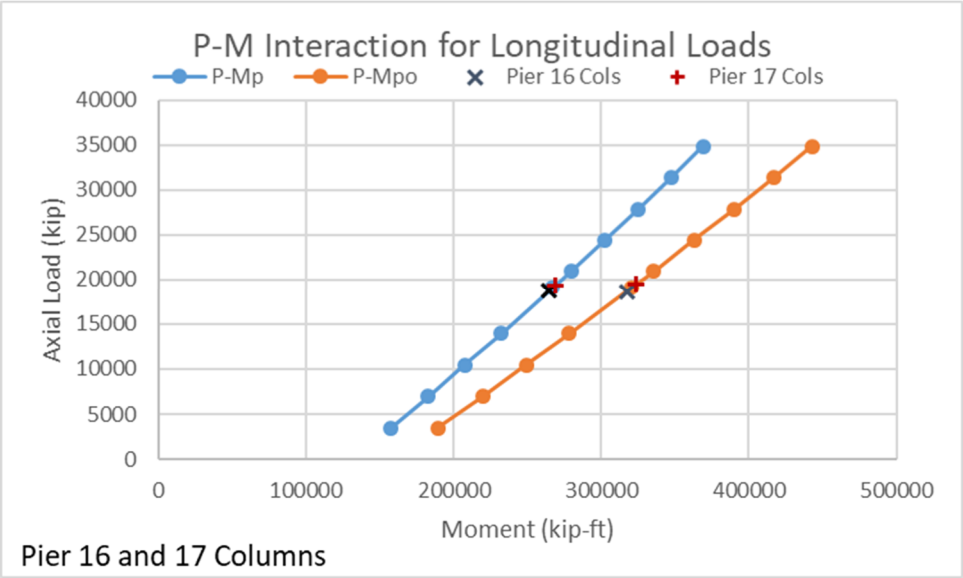
Plastic Hinge Length Calculation Columns of Piers 16 and 17

Both columns are in double curvature and form plastic hinges at top and bottom.

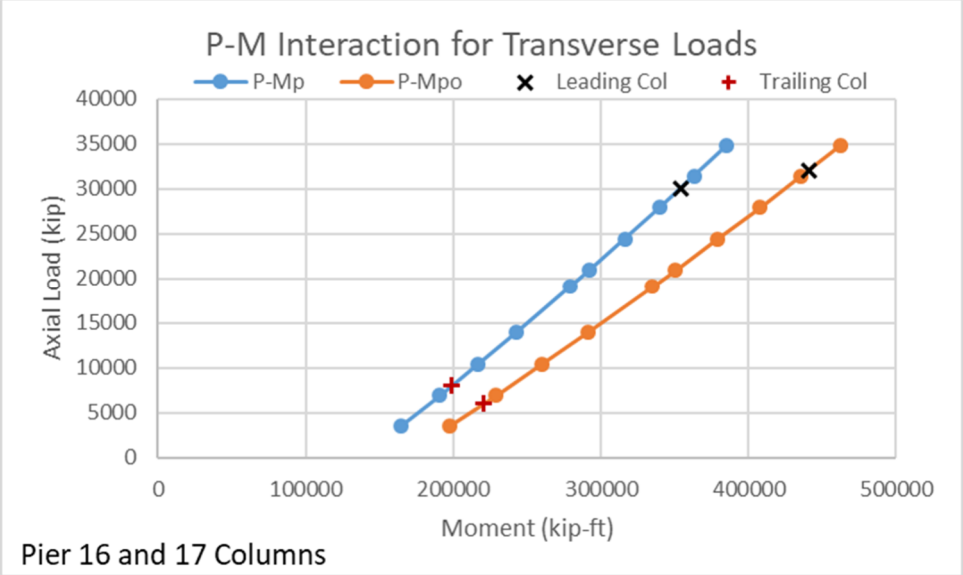
Point of contraflexure at midheight of the column, with the same Lp for Trans/Long.

Length	125	ft	1500	in
dbl #11	1.41	in		
fye A706	68	ksi		
Lsp=0.15(fye)dbl	14.4	in	strain penetration length	
Lc	750	in	length to point of contraflexure	
Lp=0.08*Lc+Lsp	74.4	in	plastic hinge length	
Lp.min=2*Lsp	28.8	in	does not control	
Therefore, Lp =	74.4	in	about both longitudinal and transverse axes	

The following provides the axial load versus moment interaction diagram at Mp and Mpo considered in the pushover analyses. Converged axial forces from the pushover results are shown. Note that the longitudinal P16 and P17 column forces are from pushover to the east, the values are switched during pushover to the west.

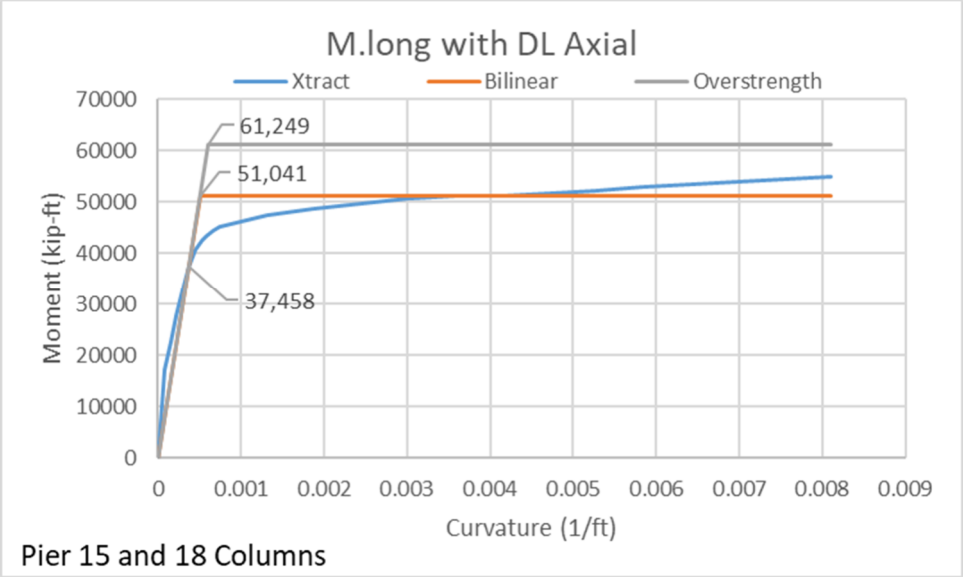


Pier 16 and 17 Columns

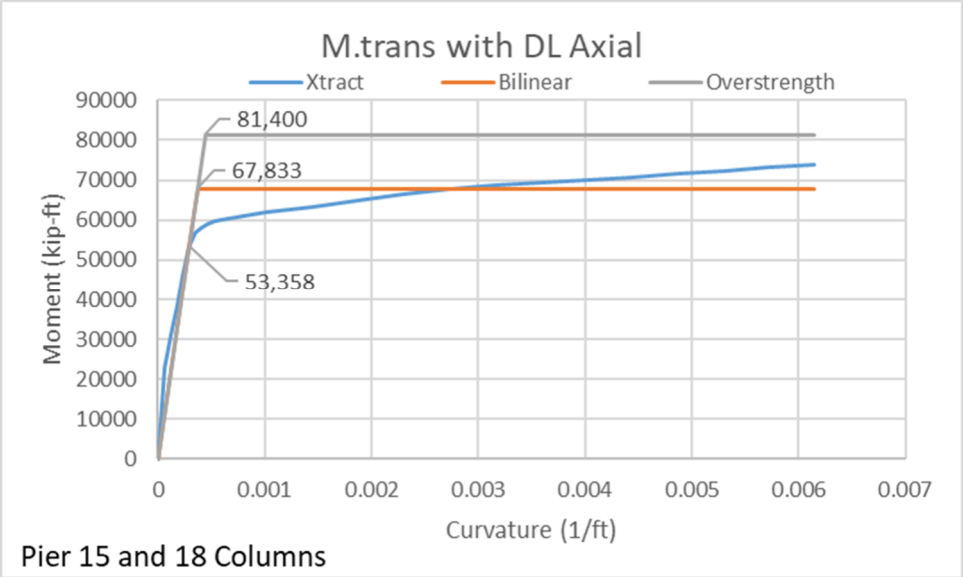


Pier 16 and 17 Columns

This process was repeated for the columns of Piers 15 and 18 as shown in the following.



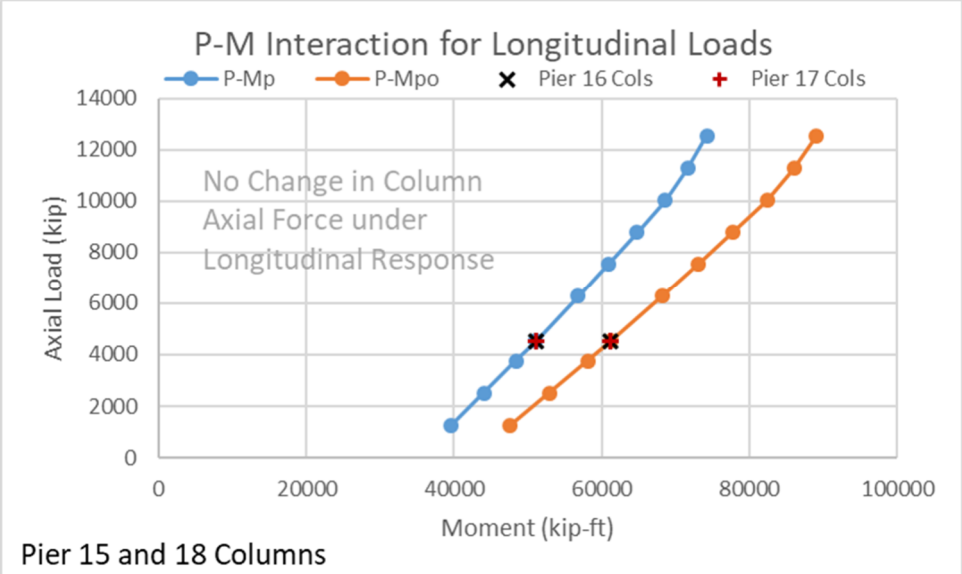
Pier 15 and 18 Columns



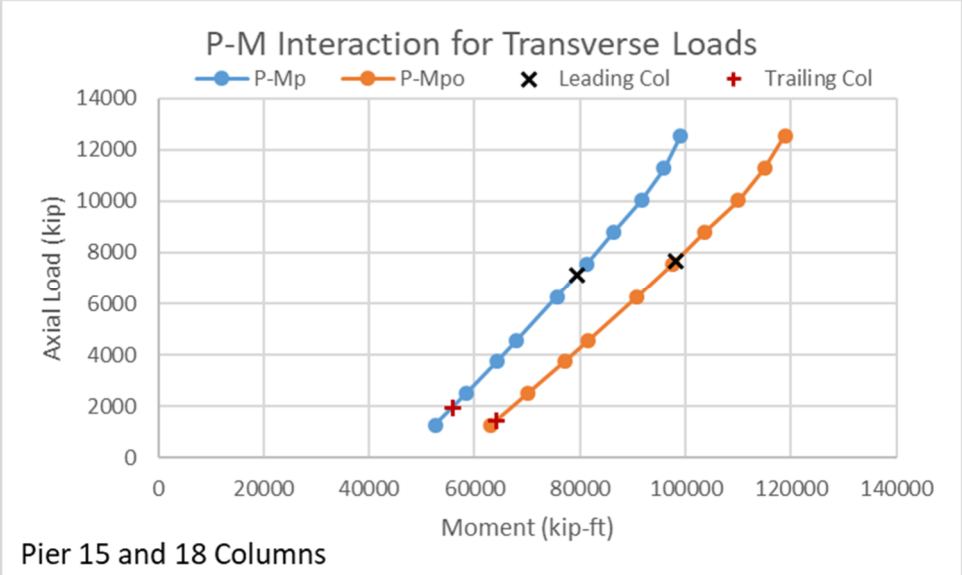
Pier 15 and 18 Columns

Moment-Curvature Anlaysis Results for Piers 15 and 18 Columns								
Dead Load Axial Force, P.dl			4535	kips				
f'ce			6.5	ksi				
Column Agross			7720	in^2				
P.dl / (f'ce*Agross)			0.090	value within the typical range				
Ece at f'ce			5008	ksi	721152	ksf		
EIeff at P.dl for M.long			100987634	kip-ft^2				
Ieff at P.dl for M.long			140.04	ft^4	Ratio Ig.long	0.258		
EIeff at P.dl for M.trans			182909411	kip-ft^2				
Ieff at P.dl for M.trans			253.64	ft^4	Ratio Ig.trans	0.303		
	kip	kip-ft	1/ft	1/ft	1/ft	rad	rad	
P-Mp Column Interaction for Longituidnal Loads						Pier 15	Pier 18	
P/(f'ce*Ag)	Axial, P	Mp.long	ϕy.long	ϕu.long	ϕp.long	θp.long	θp.long	
0.025	1255	39657	0.00051	0.00787	0.00735	0.08351	0.07998	
0.05	2509	44087	0.00051	0.00795	0.00745	0.08458	0.08101	
0.075	3764	48420	0.00051	0.00805	0.00754	0.08566	0.08204	
0.090	4535	51041	0.00051	0.00810	0.00760	0.08628	0.08263	
0.125	6273	56829	0.00051	0.00825	0.00774	0.08793	0.08422	
0.15	7527	60883	0.00051	0.00837	0.00785	0.08921	0.08544	
0.175	8782	64791	0.00052	0.00837	0.00785	0.08915	0.08538	
0.2	10036	68650	0.00052	0.00860	0.00808	0.09175	0.08787	
0.225	11291	71687	0.00052	0.00874	0.00822	0.09336	0.08941	
0.25	12545	74225	0.00049	0.00769	0.00720	0.08177	0.07832	
	kip	kip-ft	1/ft	1/ft	1/ft	rad	rad	
P-Mp Column Interaction for Transverse Loads						Pier 15	Pier 18	
P/(f'ce*Ag)	Axial, P	Mp.trans	ϕy.trans	ϕu.trans	ϕp.trans	θp.trans	θp.trans	
0.025	1255	52564	0.00036	0.00598	0.00562	0.03147	0.03012	
0.05	2509	58484	0.00036	0.00604	0.00568	0.03179	0.03043	
0.075	3764	64306	0.00037	0.00610	0.00574	0.03212	0.03074	
0.090	4535	67833	0.00037	0.00615	0.00577	0.03233	0.03094	
0.125	6273	75626	0.00038	0.00624	0.00586	0.03281	0.03141	
0.15	7527	81200	0.00039	0.00637	0.00599	0.03351	0.03208	
0.175	8782	86353	0.00039	0.00624	0.00585	0.03273	0.03133	
0.2	10036	91716	0.00040	0.00678	0.00638	0.03571	0.03418	
0.225	11291	95870	0.00039	0.00695	0.00657	0.03678	0.03520	
0.25	12545	99058	0.00036	0.00599	0.00563	0.03154	0.03018	
Plastic Hinge Length Calculation Columns of Piers 15 and 18								
Single curvature for longitudinal response and double curvature for transverse response.								
Slight difference in column heights between piers 15 and 18, thus there are four unique Lp.								
		Pier 15	**Pier 15**	**Pier 18**	**Pier 18**			
		Long. EQ	**Trans. EQ**	**Long. EQ**	**Trans. EQ**			
dbl steel		1.41	1.41	1.41	1.41	in		
fye A706		68	68	68	68	ksi		
Lsp=0.15(fye)dbl		14.4	14.4	14.4	14.4	in	strain penetration	
Lc		1524	660	1452	624	in	point of contraflexure	
		127	55	121	52	ft		
Lp=0.08*Lc+Lsp		136.3	67.2	130.5	64.3	in	plastic hinge length	
Lp.min=2*Lsp		28.8	28.8	28.8	28.8	in	does not control	
Therefore, Lp =		136.3	67.2	130.5	64.3	in		
		11.36	5.60	10.88	5.36	ft		

The following provides the axial load versus moment interaction diagram at Mp and Mpo considered in the pushover analyses. Converged axial forces from the pushover results are shown.



Pier 15 and 18 Columns



Pier 15 and 18 Columns

Moment-curvature analysis were also performed for the approach span columns to determine effective cracked section stiffness for the linear demand analysis.

3.1 EXPECTED MATERIAL PROPERTIES FROM XTRACT OUTPUT

3.1.1 Pier 16 and 17 Expected Material Properties

XTRACT Material Report

WSP

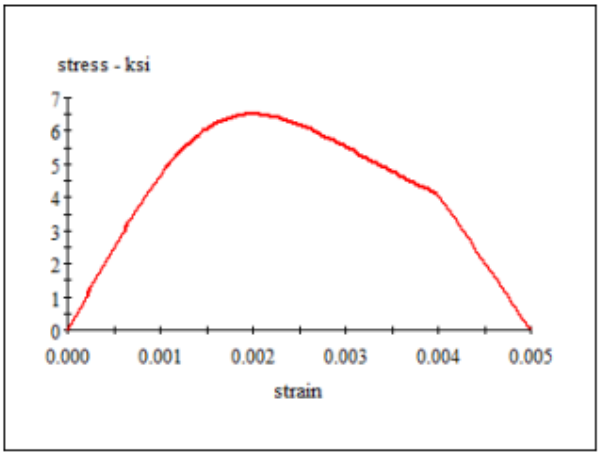
Material Name: Unconfined1
Material Type: Unconfined Concrete

Unconfined Cover Concrete Interior/Exterior

7/31/2020
WSHB
P16 and P17 Cols
Page __ of __

Input Parameters:

Tension Strength:	0 ksi
28 Day Strength:	6.500 ksi
Post Crushing Strength:	0 ksi
Tension Strain Capacity:	0 Ten
Spalling Strain:	5.000E-3 Comp
Failure Strain:	1.0000 Comp
Elastic Modulus:	5008 ksi
Secant Modulus:	3250 ksi



Model Details:

For Strain - $\varepsilon < 2 \cdot \varepsilon_t$ $fc = 0$

For Strain - $\varepsilon < 0$ $fc = \varepsilon \cdot Ec$

For Strain - $\varepsilon < \varepsilon_{cu}$ $fc = \frac{f'_c \cdot x \cdot r}{r - 1 + x^r}$

For Strain - $\varepsilon < \varepsilon_{sp}$ $fc = f_{cu} + (f_{cp} - f_{cu}) \cdot \frac{(\varepsilon - \varepsilon_{cu})}{(\varepsilon_{sp} - \varepsilon_{cu})}$

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

$r = \frac{Ec}{Ec - E_{sec}}$

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

ε = Concrete Strain

fc = Concrete Stress

Ec = Elastic Modulus

E_{sec} = Secant Modulus

ε_t = Tension Strain Capacity

ε_{cu} = Ultimate Concrete Strain

ε_{cc} = Strain at Peak Stress = .002

ε_{sp} = Spalling Strain

f'_c = 28 Day Compressive Strength

f_{cu} = Stress at ε_{cu}

f_{cp} = Post Spalling Strength

Material Color States:

- Tension strain after tension capacity
- Tension strain before tension capacity
- Initial state
- Compression before crushing strain
- Compression before end of spalling
- Compression after spalling

Reference:

Mander, J.B., Priestley, M. J. N., "Observed Stress-Strain Behavior of Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1827-1849

XTRACT Material Report

WSP

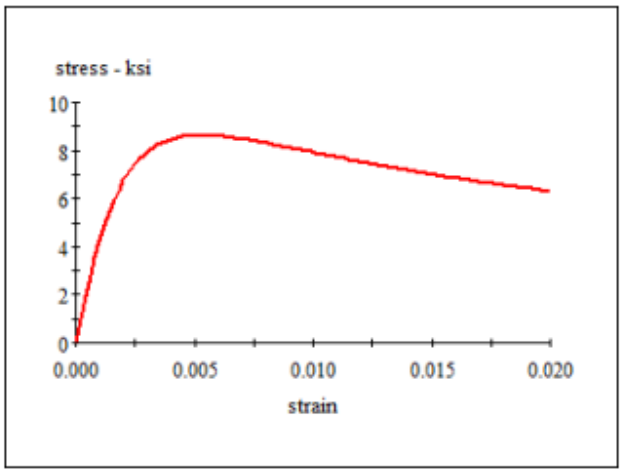
Material Name: Confined1
Material Type: Confined Concrete

Confined Concrete in the Tied Walls of the Cols

7/31/2020
WSHB
P16 and P17 Cols
Page __ of __

Input Parameters:

Tension Strength:	0 ksi
28 Day Strength:	6.500 ksi
Confined Concrete Strength:	8.632 ksi
Tension Strain Capacity:	0 Ten
Strain at Peak Stress:	5.280E-3
Crushing Strain:	20.00E-3 Comp
Elastic Modulus:	5008 ksi
Secant Modulus:	1635 ksi



Model Details:

For Strain - $\varepsilon < 2 \cdot \varepsilon_t$	$fc = 0$
For Strain - $\varepsilon < 0$	$fc = \varepsilon \cdot Ec$
For Strain - $\varepsilon < \varepsilon_{cu}$	$fc = \frac{f'_{cc} \cdot x \cdot r}{r - 1 + x^r}$

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

$$\varepsilon_{cc} = .002 \cdot \left[1 + 5 \cdot \left(\frac{f'_{cc}}{f'_c} - 1\right)\right]$$

$$r = \frac{Ec}{Ec - E_{sec}}$$

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

ε = Concrete Strain

fc = Concrete Stress

Ec = Elastic Modulus

ε_t = Tension Strain Capacity

ε_{cu} = Ultimate Concrete Strain

ε_{cc} = Strain at Peak Stress

f'_c = 28 Day Compressive Strength

f'_{cc} = Confined Concrete Strength

Material Color States:

- Tension strain after tension capacity
- Tension strain before tension capacity
- Initial state
- Compression before crushing strain

Reference:

Mander, J.B., Priestley, M. J. N., "Observed Stress-Strain Behavior of Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1827-1849

XTRACT Material Report

WSP

Material Name: Confined2

Material Type: Confined Concrete

Confined Concrete in the Hoop Boundary Elements

7/31/2020
WSHB
P16 and P17 Cols
Page __ of __

Input Parameters:

Tension Strength:	0 ksi
28 Day Strength:	6.500 ksi
Confined Concrete Strength:	8.686 ksi
Tension Strain Capacity:	0 Ten
Strain at Peak Stress:	5.363E-3
Crushing Strain:	19.85E-3 Comp
Elastic Modulus:	5008 ksi
Secant Modulus:	1620 ksi

Model Details:

For Strain - $\varepsilon < 2 \cdot \varepsilon_t$	$fc = 0$
For Strain - $\varepsilon < 0$	$fc = \varepsilon \cdot Ec$
For Strain - $\varepsilon < \varepsilon_{cu}$	$fc = \frac{f'_{cc} \cdot x \cdot r}{r - 1 + x^r}$

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

$$\varepsilon_{cc} = .002 \cdot \left[1 + 5 \cdot \left(\frac{f'_{cc}}{f'_c} - 1\right)\right]$$

$$r = \frac{Ec}{Ec - E_{sec}}$$

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

ε = Concrete Strain

fc = Concrete Stress

Ec = Elastic Modulus

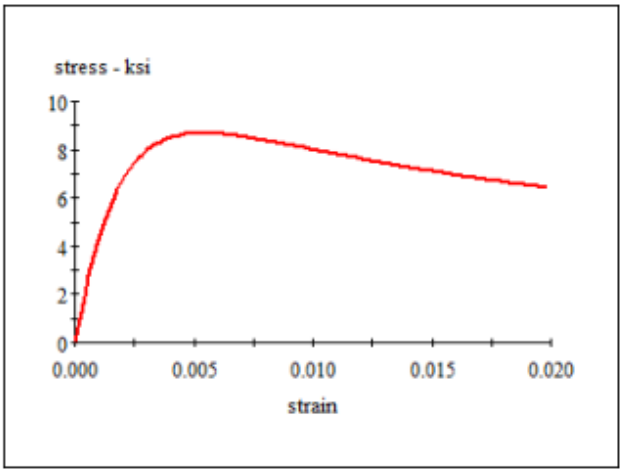
ε_t = Tension Strain Capacity

ε_{cu} = Ultimate Concrete Strain

ε_{cc} = Strain at Peak Stress

f'_c = 28 Day Compressive Strength

f'_{cc} = Confined Concrete Strength



Material Color States:

- Tension strain after tension capacity
- Tension strain before tension capacity
- Initial state
- Compression before crushing strain

Reference:

Mander, J.B., Priestley, M. J. N., "Observed Stress-Strain Behavior of Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1827-1849

XTRACT Material Report

WSP

Material Name: Steel1

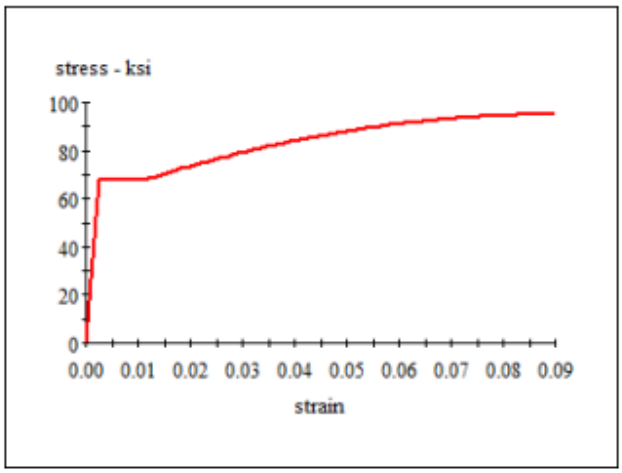
Material Type: Strain Hardening Steel

A706 #11 Longitudinal Reinforcement

7/31/2020
WSHB
P16 and P17 Cols
Page __ of __

Input Parameters:

Yield Stress:	68.00 ksi
Fracture Stress:	95.00 ksi
Yield Strain:	2.345E-3
Strain at Strain Hardening:	11.50E-3
Failure Strain:	90.00E-3
Elastic Modulus:	29.00E+3 ksi
Additional Information:	Symetric Tension and Comp.



Model Details:

For Strain - $\varepsilon < \varepsilon_y$ $fs = E \cdot \varepsilon$

For Strain - $\varepsilon < \varepsilon_{sh}$ $fs = f_y$

For Strain - $\varepsilon < \varepsilon_{su}$ $fs = f_u - (f_u - f_y) \cdot \left(\frac{\varepsilon_{su} - \varepsilon}{\varepsilon_{su} - \varepsilon_{sh}} \right)^2$

ε = Steel Strain

fs = Steel Stress

f_y = Yield Stress

f_u = Fracture Stress

ε_y = Yield Strain

ε_{sh} = Strain at Strain Hardening

ε_{su} = Failure Strain

E = Elastic Modulus

Material Color States:

- Tension force after onset of strain hardening
- Tension force after yield
- Initial state
- Compression force after yield
- Compression force after onset of strain hardening

3.1.2 Pier 15 and 18 Expected Material Properties

XTRACT Material Report

WSP

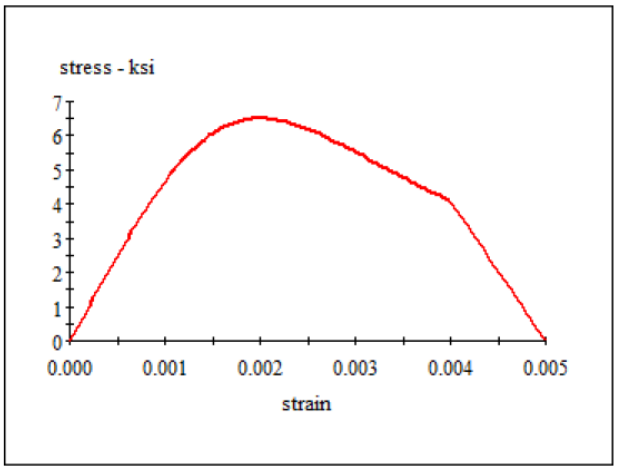
8/14/2020
WSHB
P15 and P18 Columns
Page __ of __

Material Name: Unconfined1
Material Type: Unconfined Concrete

Unconfined Cover Concrete Interior/Exterior

Input Parameters:

Tension Strength:	0 ksi
28 Day Strength:	6.500 ksi
Post Crushing Strength:	0 ksi
Tension Strain Capacity:	0 Ten
Spalling Strain:	5.000E-3 Comp
Failure Strain:	1.0000 Comp
Elastic Modulus:	5008 ksi
Secant Modulus:	3250 ksi



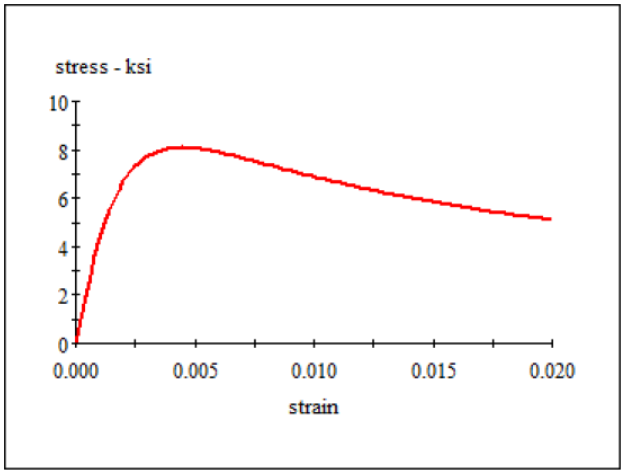
Material Name: Confined1
Material Type: Confined Concrete

Confined Concrete in the Tied Walls of the Cols

8/14/2020
WSHB
P15 and P18 Columns
Page __ of __

Input Parameters:

Tension Strength:	0 ksi
28 Day Strength:	6.500 ksi
Confined Concrete Strength:	8.081 ksi
Tension Strain Capacity:	0 Ten
Strain at Peak Stress:	4.432E-3
Crushing Strain:	20.00E-3 Comp
Elastic Modulus:	5008 ksi
Secant Modulus:	1823 ksi

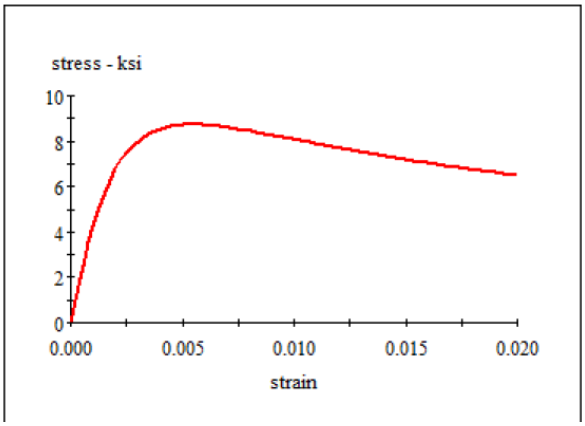


Material Name: Confined2
Material Type: Confined Concrete

Confined Concrete in the Hoop Boundary Elements

Input Parameters:

Tension Strength:	0 ksi
28 Day Strength:	6.500 ksi
Confined Concrete Strength:	8.725 ksi
Tension Strain Capacity:	0 Ten
Strain at Peak Stress:	5.423E-3
Crushing Strain:	20.00E-3 Comp
Elastic Modulus:	5008 ksi
Secant Modulus:	1609 ksi

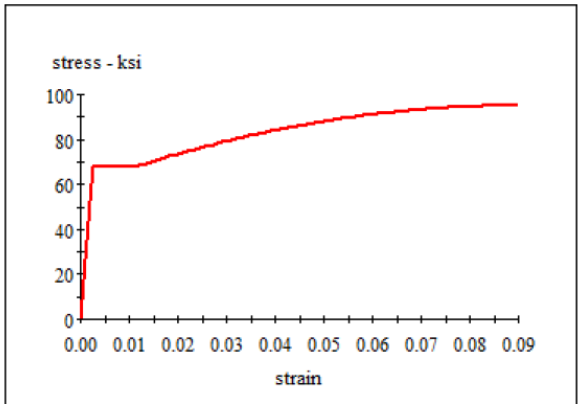


Material Name: Steel1
Material Type: Strain Hardening Steel

A706 #11 Longitudinal Reinforcement

Input Parameters:

Yield Stress:	68.00 ksi
Fracture Stress:	95.00 ksi
Yield Strain:	2.345E-3
Strain at Strain Hardening:	11.50E-3
Failure Strain:	90.00E-3
Elastic Modulus:	29.00E+3 ksi
Additional Information:	Symetric Tension and Comp.

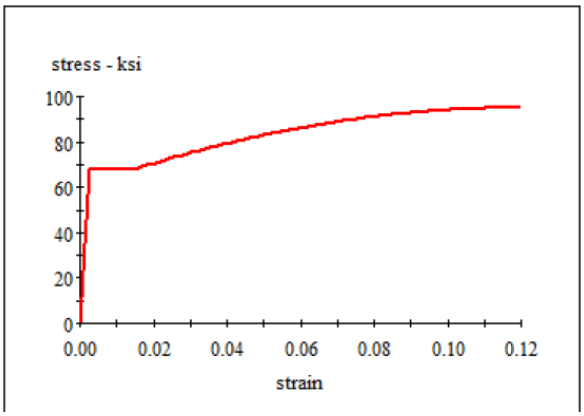


Material Name: Steel2
Material Type: Strain Hardening Steel

A706 #8 Longitudinal Reinforcement

Input Parameters:

Yield Stress:	68.00 ksi
Fracture Stress:	95.00 ksi
Yield Strain:	2.345E-3
Strain at Strain Hardening:	15.00E-3
Failure Strain:	.1200
Elastic Modulus:	29.00E+3 ksi
Additional Information:	Symetric Tension and Comp.



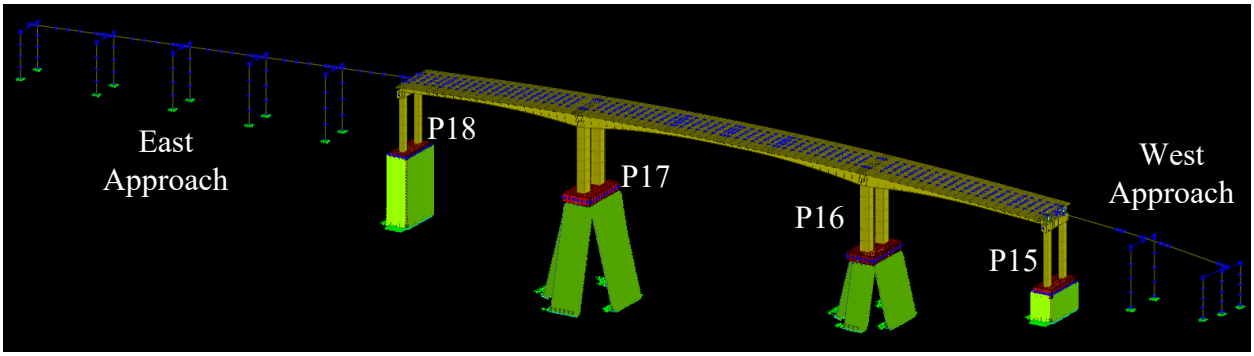
4.0 STRUCTURAL MODELS

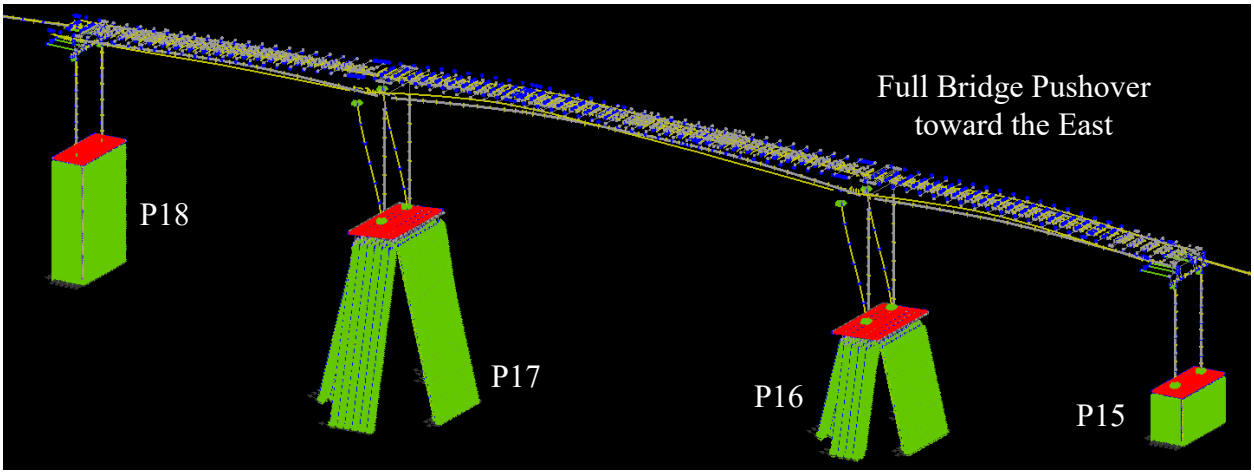
4.1 NONLINEAR STATIC PUSHOVER MODEL

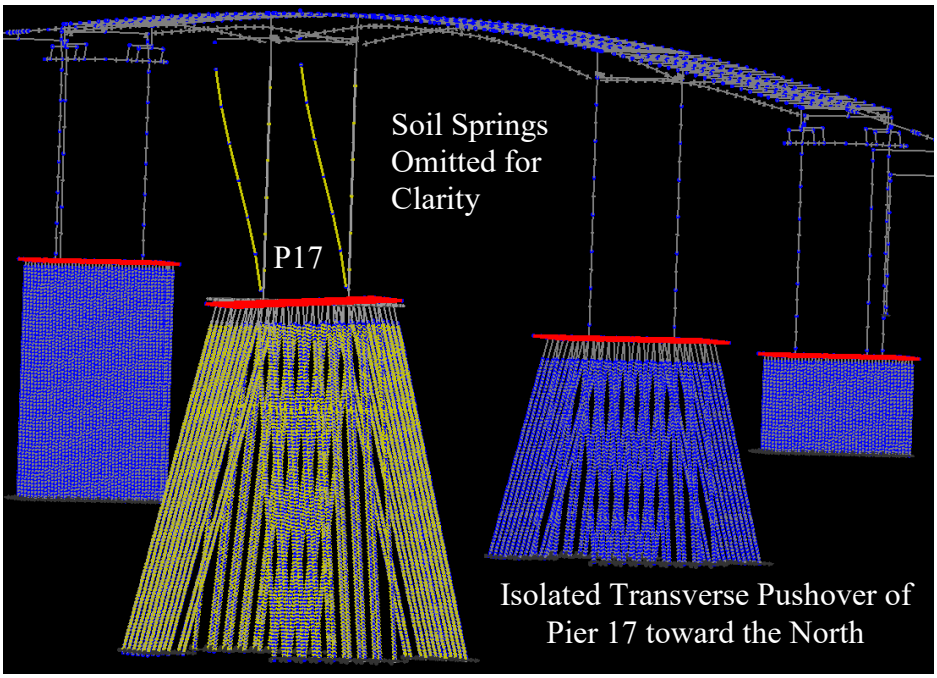
A structural model of the bridge was created to perform nonlinear static pushover analyses. Separate models were used for evaluation of displacement capacity using expected material properties and for evaluation of overstrength demands including the recommended lateral soil forces. The evaluations for the column displacement capacity did not include the lateral soil forces, as the results are intended to provide a comparison to displacement demands from linear RSA that cannot include the directional-dependent behavior. Foundation deformations under the recommended lateral soil forces are small relative to the foundation deformations under the plastic hinging mechanism from the structure above, thus this assumption is not viewed to have a significant impact on the results. These lateral soil forces were included in the assessment of force demands on the pile foundations.

Recommended lateral soil parameters for Piers 15-18 were converted into P-y springs and placed on the piles at 2ft intervals extending from the soffit of the concrete seal to the average pile tip. Both liquified and non-liquified soil properties were considered, corresponding to geotechnical recommendations. Each spring location contained a transverse and a longitudinal spring that had the same tension and compression behavior to represent the stiffness of the soil in each loading direction. Assumptions for pile axial stiffness were made prior to receiving recommendations for T-z and Q-z springs included in the draft Geotechnical report dated 10/5/2020. An axial stiffness of 2AE/L was assumed for each pile to account for skin friction along its length. This preliminary assumption was confirmed with the Geotech, but the intention is to use the recommended T-z and Q-z springs in future analyses validate the behavior.

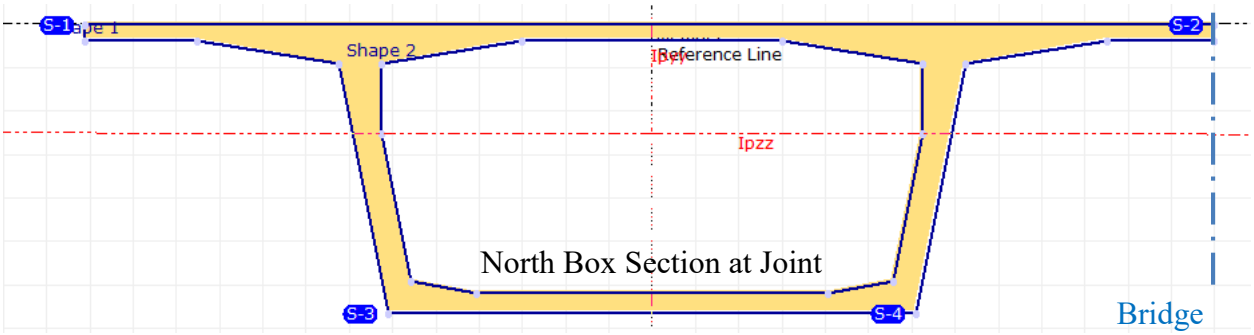
The structural model is shown below. Exaggerated deformations corresponding to a longitudinal pushover analysis of the entire bridge between Piers 15 and 18 toward the East are shown below. Isolated transverse pushover models of Piers 15-18 and longitudinal pushover models for Piers 15 and 18 were also considered. An overview of the pushover model cases is presented later. A snapshot of the transverse pushover of Pier 17 shows how staged construction was used to create sub-models for isolated pushover analyses without including the rest of the structure. Rigid links for the pier table diaphragm are not shown in the deformed shape but are present to transfer forces and equilibrate the column hinge moments.

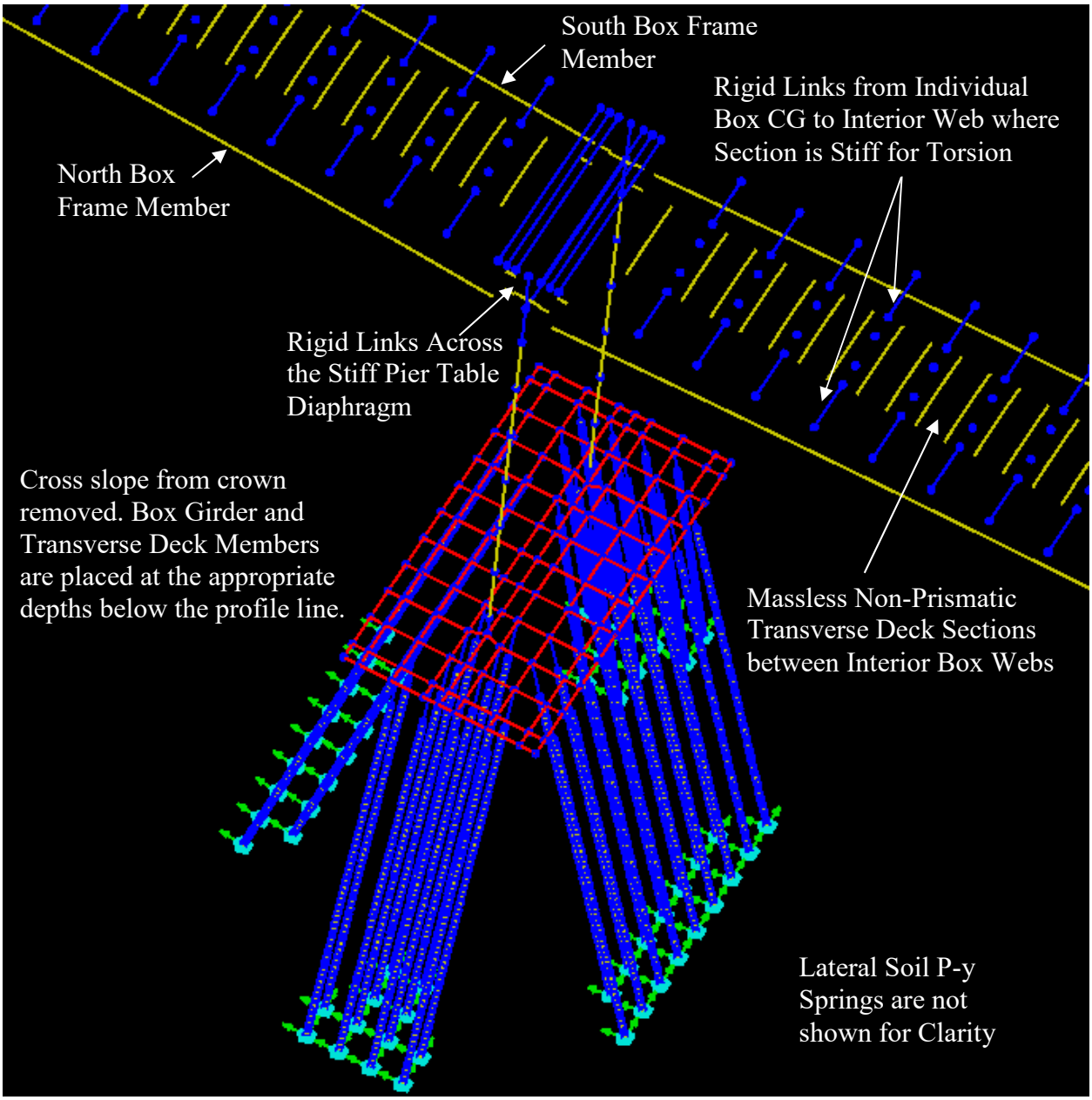






Both the nonlinear static pushover and linear RSA models considered the two-box modeling approach for the superstructure. The two-box model has individual frame members for the north and south boxes linked by members that recreate the transverse stiffness of the deck between the interior box webs. A sample north box cross section at Joint 38 is provided in the following figure. Sections for the South box were mirrored about the profile line (located at stress point S-2 below). All sources of DC and DW dead loads were added to the two-box model such that the correct distribution of weight and stiffness is achieved for seismic analysis. The additional transverse members were massless. These effects were verified in a one-to-one comparison between the single frame and two-box models using a static solution for DC+DW loading (fictitious load case that doesn't account for cantilevered construction effects). Evaluations for superstructure demands used only the seismic component of the pushover results due to forces in the plastic hinges. These loads were superimposed over the DC+DW+PS superstructure load effects from the time-dependent model. This approach is valid since the superstructure must remain elastic to satisfy capacity design requirements.

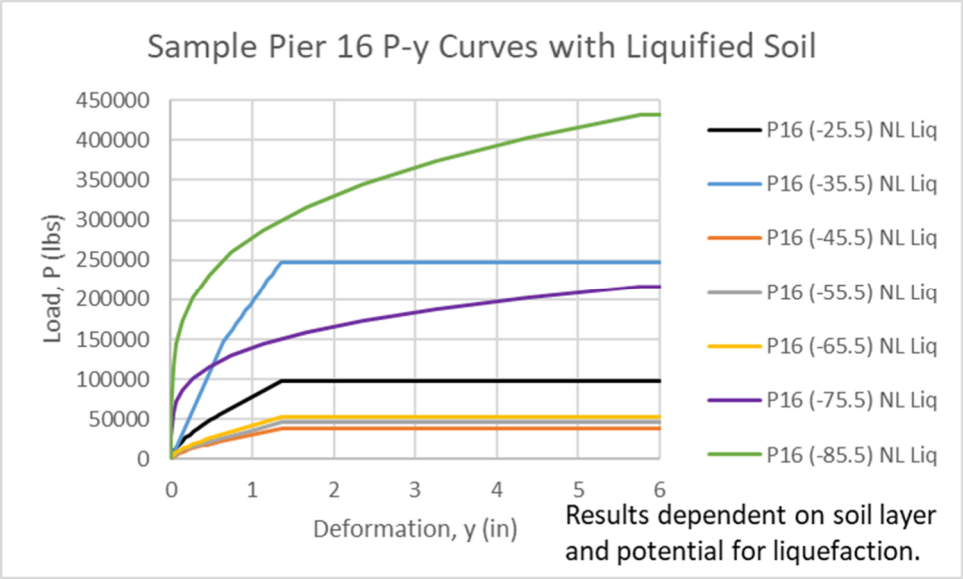

S-1
Shape 1
Shape 2
Reference Line
Ipzz
S-2
S-3
S-4
North Box Section at Joint
Bridge


South Box Frame Member
Rigid Links from Individual Box CG to Interior Web where Section is Stiff for Torsion
North Box Frame Member
Rigid Links Across the Stiff Pier Table Diaphragm
Cross slope from crown removed. Box Girder and Transverse Deck Members are placed at the appropriate depths below the profile line.
Massless Non-Prismatic Transverse Deck Sections between Interior Box Webs
Lateral Soil P-y Springs are not shown for Clarity

4.1.1 Lateral Soil P-y Springs

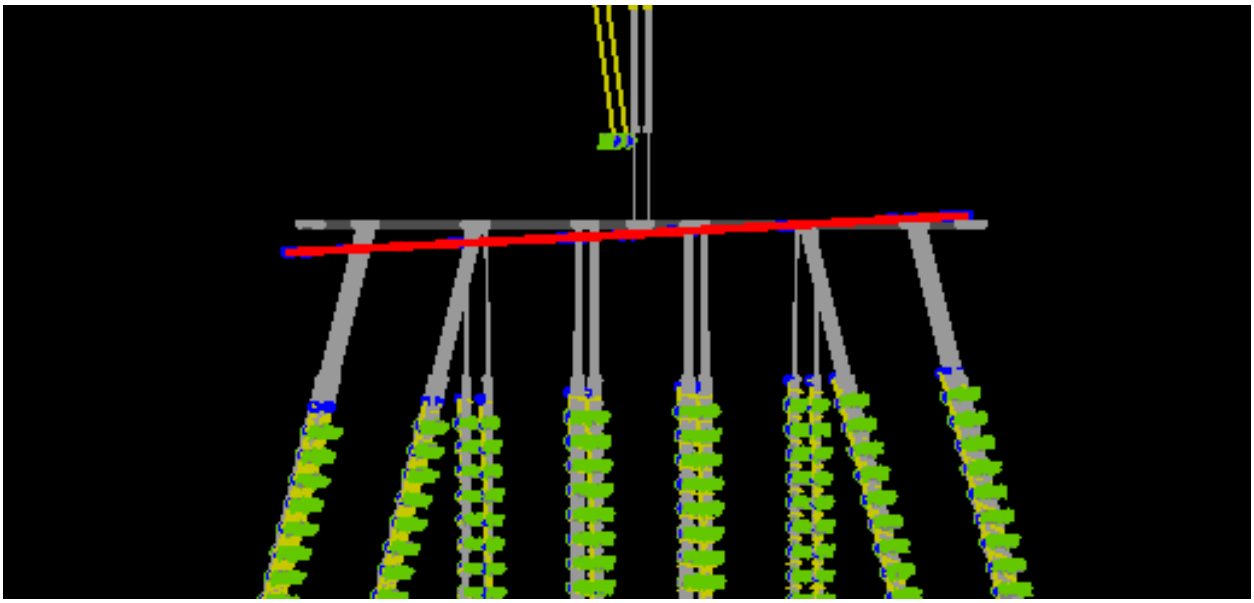
P-y springs were developed using the recommended non-liquified and separately liquefied lateral soil parameters provided by Shannon and Wilson. A sample calculation of the P-y spring properties for the 36" diameter piles from the LPile output with the recommended liquified soil properties appear below. Note that springs were placed at 2ft intervals along the entire length of the pile, thus the results needed to be modified for the tributary pile length.

Pier 16 P-y Curves with Liquifaction					P16	NL Liq		
Tributary Length		24	in					
Pile Head Elevation		-23.5	ft					
Depth below Head		2	ft		Depth below Head		4	ft
P-y Spring Elevation		-25.5	ft		P-y Spring Elevation		-27.5	ft
		P16 (-25.5) NL Liq					P16 (-27.5) NL Liq	
y(in)	p(lbs/in.)	y(in)	p(lbs)		y(in)	p(lbs/in.)	y(in)	p(lbs)
-24	-4060.82	-24	-97459.7		-24	-4280.74	-24	-102738
-1.62	-4056.77	-1.62	-97362.4		-1.62	-4276.46	-1.62	-102635
-1.35	-4052.72	-1.35	-97265.2		-1.35	-4272.19	-1.35	-102533
-0.975	-3177.7	-0.975	-76264.7		-0.975	-3349.79	-0.975	-80394.9
-0.6	-2302.68	-0.6	-55264.3		-0.6	-2427.38	-0.6	-58257.1
-0.54741	-2177.76	-0.54741	-52266.2		-0.54734	-2295.53	-0.54734	-55092.8
-0.49481	-2048.03	-0.49481	-49152.8		-0.49469	-2158.61	-0.49469	-51806.7
-0.44222	-1912.78	-0.44222	-45906.7		-0.44203	-2015.84	-0.44203	-48380.2
-0.38962	-1771.05	-0.38962	-42505.2		-0.38937	-1866.23	-0.38937	-44789.4
-0.33703	-1621.59	-0.33703	-38918.1		-0.33672	-1708.43	-0.33672	-41002.4
-0.28444	-1462.65	-0.28444	-35103.6		-0.28406	-1540.62	-0.28406	-36974.8
-0.23184	-1291.68	-0.23184	-31000.3		-0.2314	-1360.06	-0.2314	-32641.4
-0.17925	-1104.64	-0.17925	-26511.2		-0.17874	-1162.47	-0.17874	-27899.2
-0.12665	-894.358	-0.12665	-21464.6		-0.12609	-940.225	-0.12609	-22565.4
-0.07406	-645.398	-0.07406	-15489.6		-0.07343	-676.826	-0.07343	-16243.8
-0.02147	-303.973	-0.02147	-7295.34		-0.02077	-314.107	-0.02077	-7538.57
0	0	0	0		0	0	0	0
0.02147	303.9726	0.02147	7295.342		0.02077	314.1072	0.02077	7538.572
0.07406	645.398	0.07406	15489.55		0.07343	676.8264	0.07343	16243.83
0.12665	894.3583	0.12665	21464.6		0.12609	940.2249	0.12609	22565.4
0.17925	1104.635	0.17925	26511.24		0.17874	1162.466	0.17874	27899.18
0.23184	1291.68	0.23184	31000.33		0.2314	1360.057	0.2314	32641.36
0.28444	1462.651	0.28444	35103.62		0.28406	1540.615	0.28406	36974.76
0.33703	1621.587	0.33703	38918.1		0.33672	1708.433	0.33672	41002.4
0.38962	1771.048	0.38962	42505.16		0.38937	1866.225	0.38937	44789.41
0.44222	1912.778	0.44222	45906.68		0.44203	2015.841	0.44203	48380.18
0.49481	2048.035	0.49481	49152.83		0.49469	2158.612	0.49469	51806.68
0.54741	2177.759	0.54741	52266.21		0.54734	2295.535	0.54734	55092.84
0.6	2302.68	0.6	55264.31		0.6	2427.381	0.6	58257.15
0.975	3177.698	0.975	76264.75		0.975	3349.786	0.975	80394.86
1.35	4052.716	1.35	97265.18		1.35	4272.191	1.35	102532.6
1.62	4056.769	1.62	97362.45		1.62	4276.463	1.62	102635.1
24	4060.821	24	97459.71		24	4280.735	24	102737.6



For the batter piles, the springs were placed orthogonal to the axial direction of the pile in both the longitudinal and transverse directions, providing only lateral resistance. T-z and Q-z soil springs representing the pile axial interaction with the soil included in the draft Geotechnical report dated 10/5/2020 will be used in future evaluations.

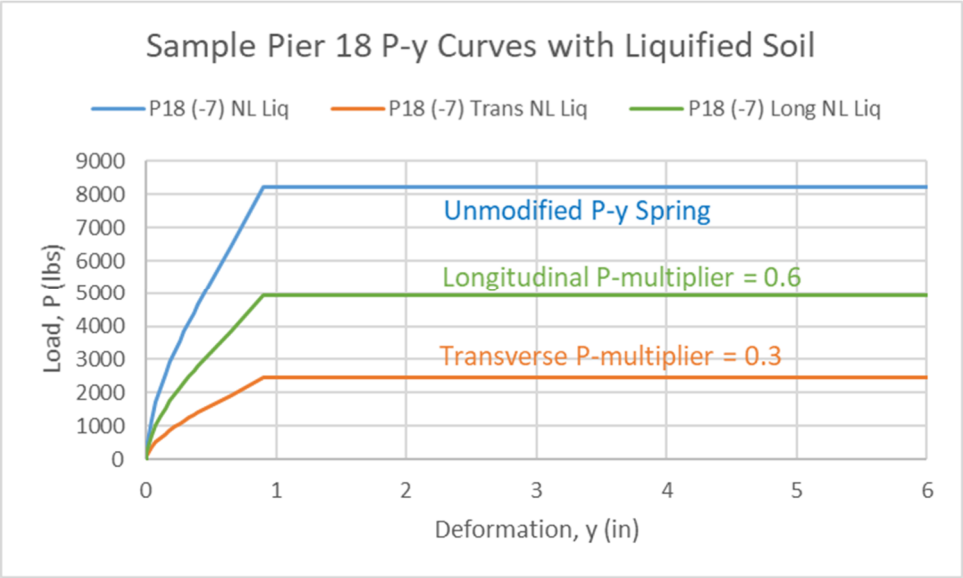
The following provides a brief example of the exaggerated foundation deformations from the model.



Sample P-y curves for plumb piles of Pier 18 with group factor reductions appear below.

Pier 18 P-y Curves with Liquifaction					P18	NL Liq	
Tributary Length		24	in			Trans NL Liq	
Pile Head Elevation		-5	ft			Long NL Liq	
Depth below Head		2	ft				
P-y Spring Elevation		-7	ft	Trans Pm	0.3	Long Pm	0.6
		P18 (-7) NL Liq		P18 (-7) Trans NL Liq		P18 (-7) Long NL Liq	
y(in)	p(lbs/in.)	y(in)	p(lbs)	y(in)	p(lbs)	y(in)	p(lbs)
-24	-342.573	-24	-8221.74	-24	-2466.52	-24	-4933.05
-1.08	-342.231	-1.08	-8213.54	-1.08	-2464.06	-1.08	-4928.12
-0.9	-341.889	-0.9	-8205.33	-0.9	-2461.6	-0.9	-4923.2
-0.65	-268.072	-0.65	-6433.73	-0.65	-1930.12	-0.65	-3860.24
-0.4	-194.255	-0.4	-4662.12	-0.4	-1398.64	-0.4	-2797.27
-0.36409	-183.458	-0.36409	-4403	-0.36409	-1320.9	-0.36409	-2641.8
-0.32819	-172.235	-0.32819	-4133.64	-0.32819	-1240.09	-0.32819	-2480.19
-0.29228	-160.519	-0.29228	-3852.45	-0.29228	-1155.74	-0.29228	-2311.47
-0.25637	-148.223	-0.25637	-3557.35	-0.25637	-1067.2	-0.25637	-2134.41
-0.22047	-135.23	-0.22047	-3245.52	-0.22047	-973.656	-0.22047	-1947.31
-0.18456	-121.376	-0.18456	-2913.02	-0.18456	-873.907	-0.18456	-1747.81
-0.14865	-106.415	-0.14865	-2553.97	-0.14865	-766.19	-0.14865	-1532.38
-0.11275	-89.9501	-0.11275	-2158.8	-0.11275	-647.641	-0.11275	-1295.28
-0.07684	-71.2462	-0.07684	-1709.91	-0.07684	-512.972	-0.07684	-1025.94
-0.04093	-48.5816	-0.04093	-1165.96	-0.04093	-349.788	-0.04093	-699.576
-0.00503	-13.5763	-0.00503	-325.832	-0.00503	-97.7496	-0.00503	-195.499
0	0	0	0	0	0	0	0
0.00503	13.57633	0.00503	325.8319	0.00503	97.74958	0.00503	195.4992
0.04093	48.58164	0.04093	1165.959	0.04093	349.7878	0.04093	699.5756
0.07684	71.24616	0.07684	1709.908	0.07684	512.9724	0.07684	1025.945
0.11275	89.9501	0.11275	2158.802	0.11275	647.6407	0.11275	1295.281
0.14865	106.4153	0.14865	2553.966	0.14865	766.1898	0.14865	1532.38
0.18456	121.376	0.18456	2913.024	0.18456	873.9071	0.18456	1747.814
0.22047	135.2301	0.22047	3245.521	0.22047	973.6564	0.22047	1947.313
0.25637	148.2229	0.25637	3557.348	0.25637	1067.205	0.25637	2134.409
0.29228	160.5189	0.29228	3852.453	0.29228	1155.736	0.29228	2311.472
0.32819	172.2351	0.32819	4133.642	0.32819	1240.093	0.32819	2480.185
0.36409	183.4583	0.36409	4402.999	0.36409	1320.9	0.36409	2641.799
0.4	194.2551	0.4	4662.121	0.4	1398.636	0.4	2797.273
0.65	268.072	0.65	6433.728	0.65	1930.118	0.65	3860.237
0.9	341.8889	0.9	8205.334	0.9	2461.6	0.9	4923.2
1.08	342.2308	1.08	8213.539	1.08	2464.062	1.08	4928.124
24	342.5727	24	8221.745	24	2466.523	24	4933.047

Piers 15 and 18 contained hollow 24" prestressed concrete plumb piles with dowelled connections into the footing. P-y springs were developed based on the LPile output and then reduced by the associated group factor P-multiplier in the longitudinal and transverse directions, as shown in the following sample output. Sample calculations appear on the previous page.

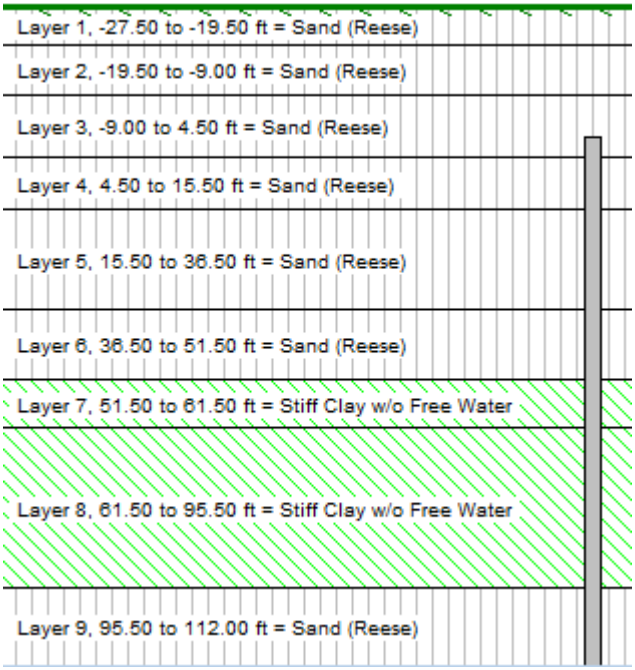


The following represents a summary of the input liquified soil properties for Pier 16. The pile head is at the soffit of the concrete seal at elevation -23.5 ft. Note that LPile soil layering elevations are relative to the pile head, with a positive depth pointing toward the pile tip. Unique P-y springs were also considered using the recommended non-liquefied soil properties.

Summary of Input Soil Properties

Pier 16 LPile

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand	-27.5000	125.0000	--	32.0000	--	25.0000
	(Reese, et al.)	-19.5000	125.0000	--	32.0000	--	25.0000
2	Sand	-19.5000	48.0000	--	8.0000	--	5.0000
	(Reese, et al.)	-9.0000	48.0000	--	8.0000	--	5.0000
3	Sand	-9.0000	58.0000	--	20.0000	--	40.0000
	(Reese, et al.)	4.5000	58.0000	--	20.0000	--	40.0000
4	Sand	4.5000	48.0000	--	26.0000	--	20.0000
	(Reese, et al.)	15.5000	48.0000	--	26.0000	--	20.0000
5	Sand	15.5000	58.0000	--	10.0000	--	20.0000
	(Reese, et al.)	36.5000	58.0000	--	10.0000	--	20.0000
6	Sand	36.5000	58.0000	--	10.0000	--	25.0000
	(Reese, et al.)	51.5000	58.0000	--	10.0000	--	25.0000
7	Stiff Clay	51.5000	58.0000	4000.	--	0.00400	--
	w/o Free Water	61.5000	58.0000	4000.	--	0.00400	--
8	Stiff Clay	61.5000	58.0000	8000.	--	0.00400	--
	w/o Free Water	95.5000	58.0000	8000.	--	0.00400	--
9	Sand	95.5000	63.0000	--	39.0000	--	125.0000
	(Reese, et al.)	112.0000	63.0000	--	39.0000	--	125.0000



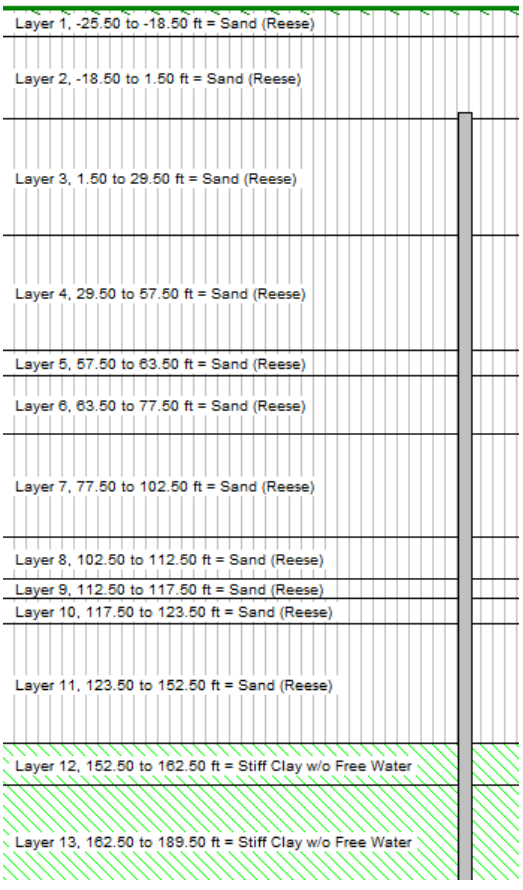
Pier 16 Soil Layering from LPile

The following represents input for liquified soil properties for Pier 17, with a -23.5 ft pile head elevation.

Pier 17 LPile

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand	-25.5000	115.0000	--	8.0000	--	20.0000
	(Reese, et al.)	-18.5000	115.0000	--	8.0000	--	20.0000
2	Sand	-18.5000	53.0000	--	8.0000	--	15.0000
	(Reese, et al.)	1.5000	53.0000	--	8.0000	--	15.0000
3	Sand	1.5000	63.0000	--	36.0000	--	85.0000
	(Reese, et al.)	29.5000	63.0000	--	36.0000	--	85.0000
4	Sand	29.5000	63.0000	--	38.0000	--	115.0000
	(Reese, et al.)	57.5000	63.0000	--	38.0000	--	115.0000
5	Sand	57.5000	53.0000	--	27.0000	--	35.0000
	(Reese, et al.)	63.5000	53.0000	--	27.0000	--	35.0000
6	Sand	63.5000	63.0000	--	32.0000	--	95.0000
	(Reese, et al.)	77.5000	63.0000	--	32.0000	--	95.0000
7	Sand	77.5000	53.0000	--	34.0000	--	110.0000
	(Reese, et al.)	102.5000	53.0000	--	34.0000	--	110.0000
8	Sand	102.5000	63.0000	--	39.0000	--	125.0000
	(Reese, et al.)	112.5000	63.0000	--	39.0000	--	125.0000
9	Sand	112.5000	43.0000	--	24.0000	--	5.0000
	(Reese, et al.)	117.5000	43.0000	--	24.0000	--	5.0000
10	Sand	117.5000	48.0000	--	39.0000	--	125.0000
	(Reese, et al.)	123.5000	48.0000	--	39.0000	--	125.0000
11	Sand	123.5000	63.0000	--	39.0000	--	125.0000
	(Reese, et al.)	152.5000	63.0000	--	39.0000	--	125.0000
12	Stiff Clay	152.5000	53.0000	8000.	--	0.00400	--
	w/o Free Water	162.5000	53.0000	8000.	--	0.00400	--
13	Stiff Clay	162.5000	58.0000	12000.	--	0.00400	--
	w/o Free Water	189.5000	58.0000	12000.	--	0.00400	--



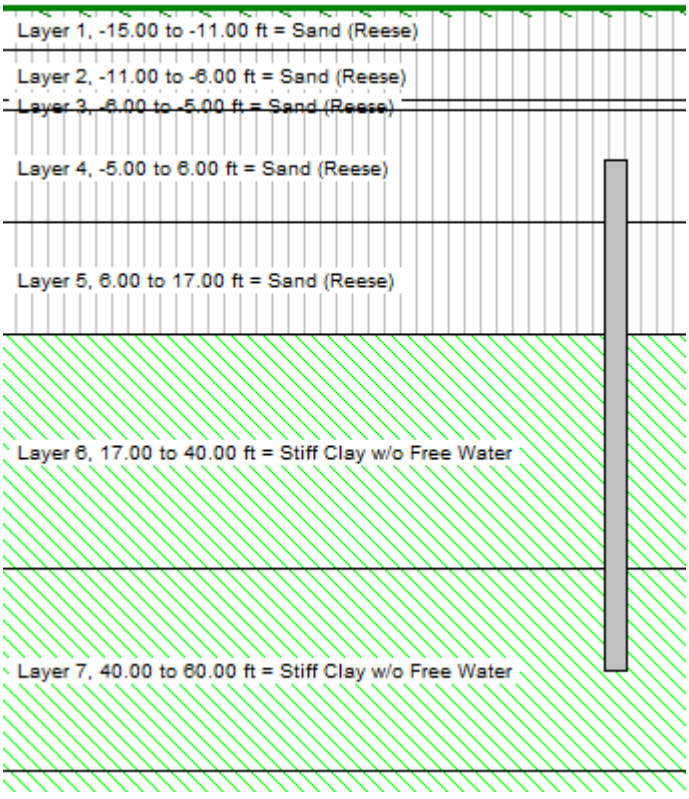
Pier 17 Soil Layering from LPile

The following represents input for liquified soil properties for Pier 15, with a pile head elevation of -11ft.

Summary of Input Soil Properties

Pier 15 LPile

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand	-15.0000	120.0000	--	31.0000	--	75.0000
	(Reese, et al.)	-11.0000	120.0000	--	31.0000	--	75.0000
2	Sand	-11.0000	120.0000	--	32.0000	--	90.0000
	(Reese, et al.)	-6.0000	120.0000	--	32.0000	--	90.0000
3	Sand	-6.0000	58.0000	--	32.0000	--	60.0000
	(Reese, et al.)	-5.0000	58.0000	--	32.0000	--	60.0000
4	Sand	-5.0000	68.0000	--	30.0000	--	50.0000
	(Reese, et al.)	6.0000	68.0000	--	30.0000	--	50.0000
5	Sand	6.0000	58.0000	--	12.0000	--	25.0000
	(Reese, et al.)	17.0000	58.0000	--	12.0000	--	25.0000
6	Stiff Clay	17.0000	53.0000	4000.	--	0.00400	--
	w/o Free Water	40.0000	53.0000	4000.	--	0.00400	--
7	Stiff Clay	40.0000	53.0000	6000.	--	0.00400	--
	w/o Free Water	60.0000	53.0000	6000.	--	0.00400	--
8	Stiff Clay	60.0000	53.0000	8000.	--	0.00400	--
	w/o Free Water	109.0000	53.0000	8000.	--	0.00400	--
9	Sand	109.0000	63.0000	--	39.0000	--	125.0000
	(Reese, et al.)	155.0000	63.0000	--	39.0000	--	125.0000



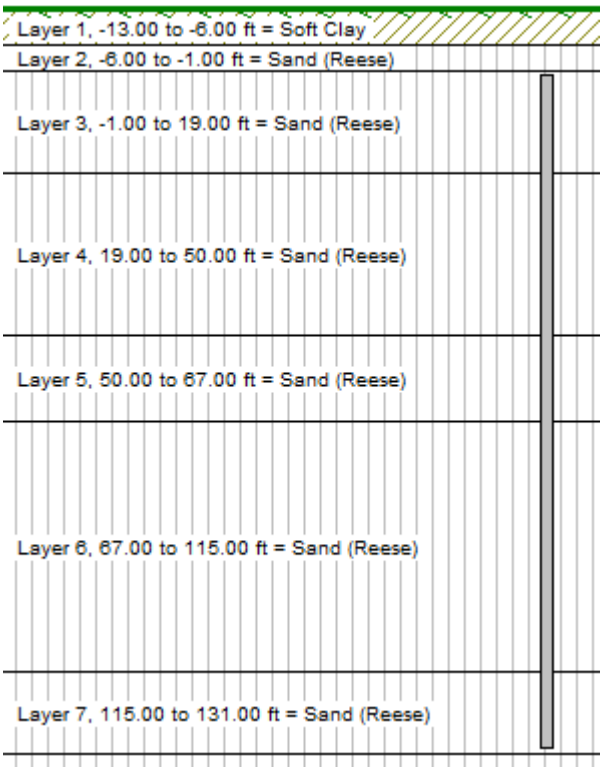
Pier 15 Soil Layering from LPile

The following represents input for liquified soil properties for Pier 18, with a pile head elevation of -5ft.

Summary of Input Soil Properties

Pier 18 LPile

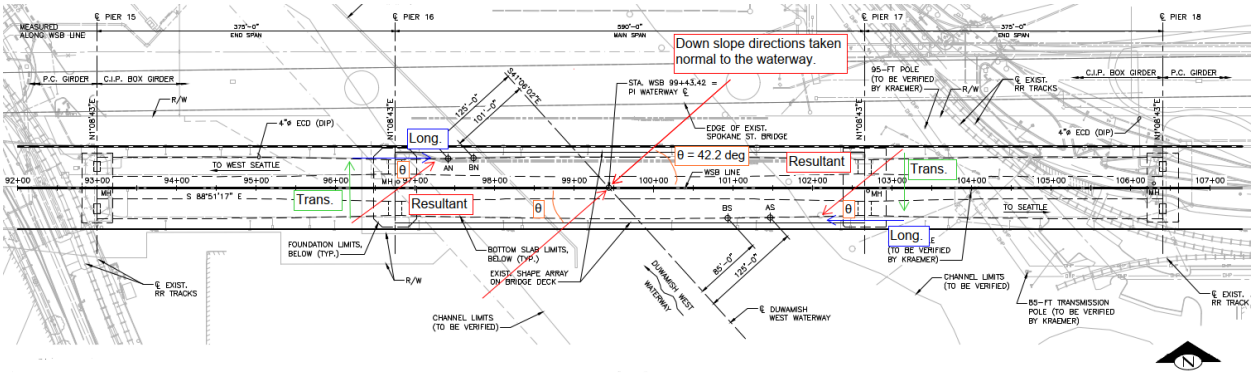
Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Soft Clay	-13.0000	90.0000	400.0000	--	0.02000	--
		-6.0000	90.0000	400.0000	--	0.02000	--
2	Sand (Reese, et al.)	-6.0000	120.0000	--	8.0000	--	25.0000
		-1.0000	120.0000	--	8.0000	--	25.0000
3	Sand (Reese, et al.)	-1.0000	58.0000	--	8.0000	--	15.0000
		19.0000	58.0000	--	8.0000	--	15.0000
4	Sand (Reese, et al.)	19.0000	58.0000	--	22.0000	--	50.0000
		50.0000	58.0000	--	22.0000	--	50.0000
5	Sand (Reese, et al.)	50.0000	58.0000	--	5.0000	--	10.0000
		67.0000	58.0000	--	5.0000	--	10.0000
6	Sand (Reese, et al.)	67.0000	58.0000	--	33.0000	--	65.0000
		115.0000	58.0000	--	33.0000	--	65.0000
7	Sand (Reese, et al.)	115.0000	53.0000	--	29.0000	--	65.0000
		131.0000	53.0000	--	29.0000	--	65.0000
8	Sand (Reese, et al.)	131.0000	63.0000	--	39.0000	--	120.0000
		170.0000	63.0000	--	39.0000	--	120.0000
9	Sand (Reese, et al.)	170.0000	53.0000	--	28.0000	--	45.0000
		176.0000	53.0000	--	28.0000	--	45.0000
10	Sand (Reese, et al.)	176.0000	68.0000	--	39.0000	--	125.0000
		180.0000	68.0000	--	39.0000	--	125.0000
11	Stiff Clay w/o Free Water	180.0000	63.0000	12000.	--	0.00400	--
		217.5000	63.0000	12000.	--	0.00400	--

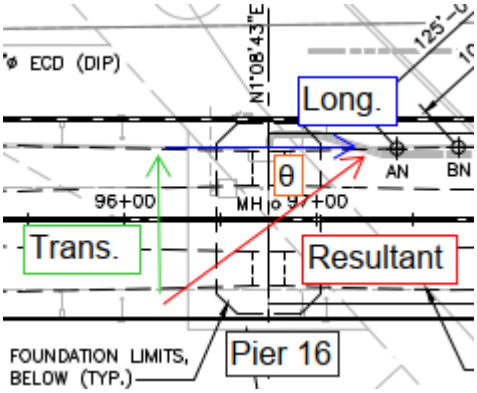


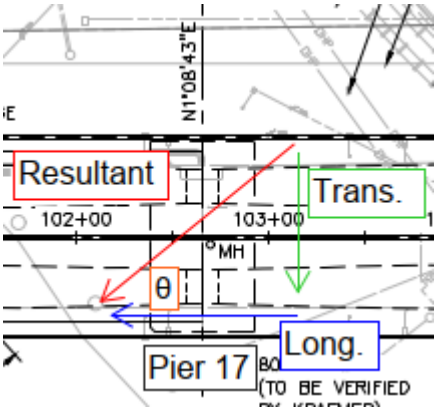
Pier 18 Soil Layering from LPile

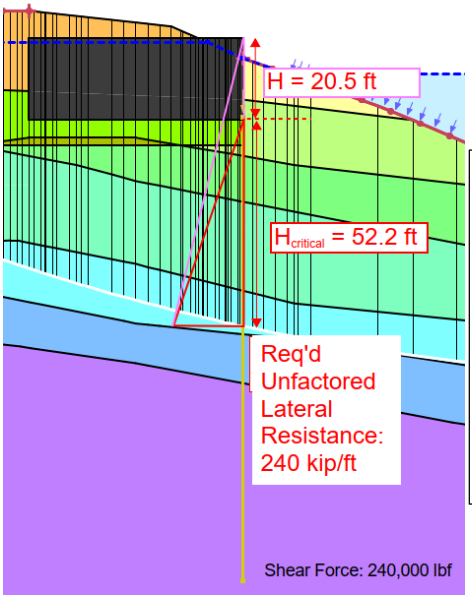
4.1.2 Modeling Lateral Soil Forces

The following downslope directions and orientations for the longitudinal and transverse components of the lateral soil forces were considered. The angle θ = 42.2 deg represents the angle between the longitudinal axis and a line taken normal to the centerline of the Duwamish waterway, assumed to represent the downslope direction following Geotechnical recommendations.

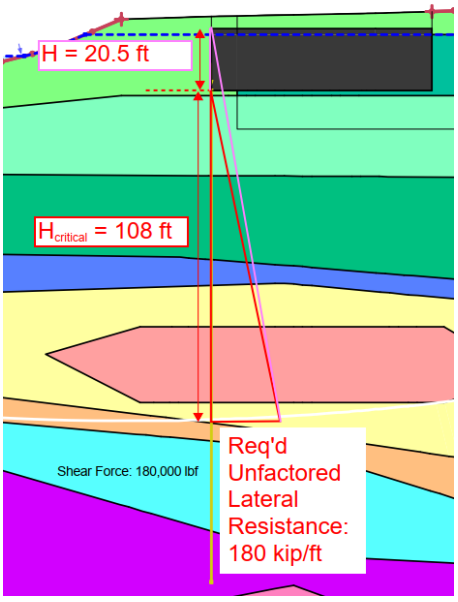








Geotech lateral soil forces to maintain the required factor of safety for slope stability.



The lateral soil forces provided by Shannon and Wilson represent pile loading required to maintain the code-based factor of safety for slope stability. The lateral soil forces were interpreted as resultant forces per foot width needed to restrain the soil from movement and were applied concurrently with the liquified soil properties following geotechnical recommendations. The resultant forces were converted to distributed pile loads as described in the following table. The lateral pile forces were then converted into longitudinal and transverse load components to orient the resultant force in the down slope direction.

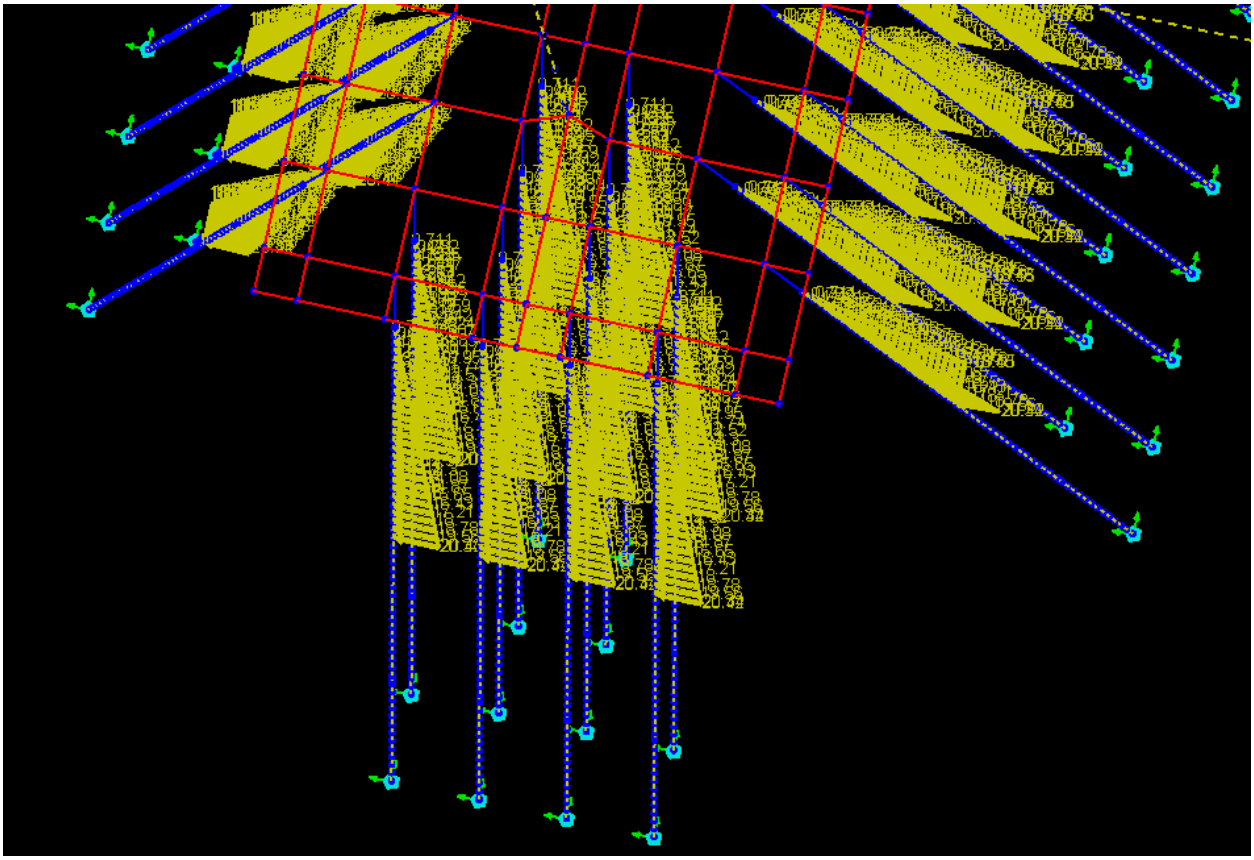
Lateral Loads	Pier 16	Pier 17		
H.critical	52.2	108	ft	below seal soffit
Resultant	240	180	kip/ft	per foot width
Pile Dia	3	3	ft	considered as the width
Resultant	720	540	kips	each pile
Top Load	0	0	klf	at the seal soffit
Base Load	27.59	10	klf	at depth H.critical
Soffit Seal	-23.5	-23.5	ft	elevation pile head
H.critical	-75.7	-131.5	ft	elevation base of loading
D.head	-52.2	-108	ft	depth below pile head

Lateral Load Oriengation		
θ.LS.long	42.2	deg
LS.trans	0.67	sin(θ.LS)
LS.long	0.74	cos(θ.LS)

Later, it was determined that the Geotech had a more simplified recommendation for applying the lateral soil forces. It was recommended to apply the resultant forces on a per foot width basis, where width was taken from the pile cap dimension. The loading applied to the individual piles based on their width exceeded this more simplified recommendation, so this more conservative interpretation of the loading was maintained. The following calculations provide a comparison of the applied loading, using a diagonal dimension of the 99 ft x 54 ft footing as 113 ft.

- Pier 16 with 56 Piles
  - Combined Pile Force = 56 (3 ft) (240 kip/ft) = 40320 kip
  - Geotech Recommendation = 113 ft (240 kip/ft) = 27120 kip
  - Conservatism Factor = 40320 / 27120 = 1.5
    - Conservative evaluation maintained; future work can revise the loading.
- Pier 17 with 60 Piles
  - Combined Pile Force = 60 (3 ft) (180 kip/ft) = 32400 kip
  - Geotech Recommendation = 113 ft (180 kip/ft) = 20340 kip
  - Conservatism Factor = 32400 / 20340 = 1.6
    - Conservative evaluation maintained; future work can revise the loading.

The following diagram depicts the longitudinal and transverse pile loading applied at Pier 16 oriented in the down slope direction. For the batter piles, the loads were assigned such that they were orthogonal to the axial direction of the pile in both the longitudinal and transverse directions. So far, only the more conservative red triangular distribution of loading over the height H.critical has been evaluated. The pink triangular distribution over a height (H + H.critical) represents a second load case including the height of the footing and concrete seal that has the same resultant forces (240 klf for Pier 16 and 180 klf for Pier 17) as the loading evaluated. This case is not expected to significantly alter the results.



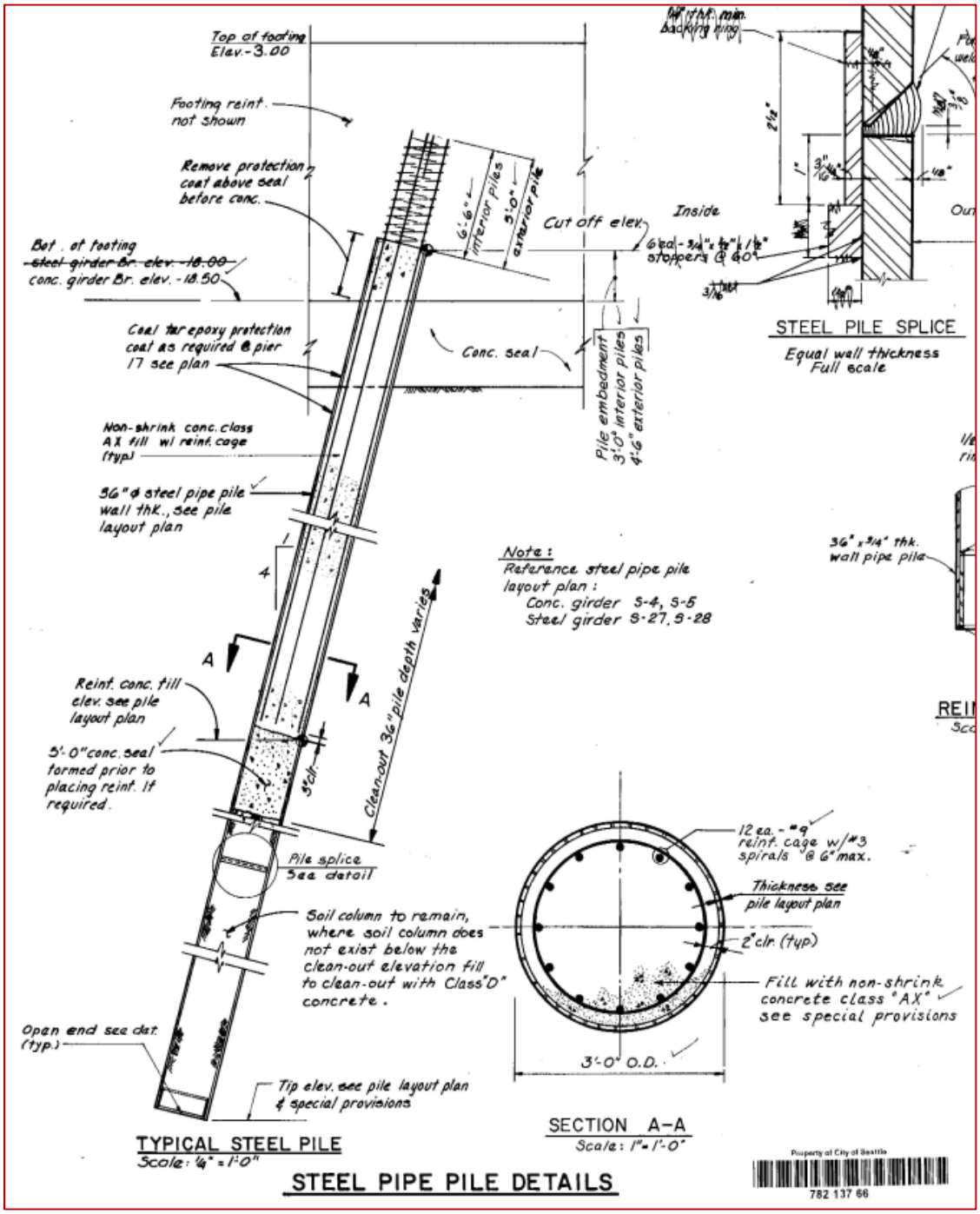
4.1.3 Overburden and Buoyant Footing Weights

The structural model considered the pile cap footings as a rigid weightless material with applied area loads oriented in the gravity direction to recreate the weight of overburden and the buoyant foundation. The effective weight of soils and elevations were determined from Geotechnical recommendations for lateral soil parameters. The following represents the applied loading at each foundation. The unit weight of the pile material was adjusted for submerged conditions. The considered pile geometry and properties are discussed in the following section.

	Pier 15	Pier 16	Pier 17	Pier 18	
Overburden	2682	3958	2727	1453	kip
Buoyant Footing/Seal	3046	10593	11213	4179	kip
Combined Weight	5728	14551	13940	5632	kip

4.1.4 Pile Geometry and Details for the RC Filled Portion

Details for the 36” diameter steel pipe piles from the as-built plans are shown below. The upper region of the pile acts as a reinforced concrete filled steel tube (RCFST), with consideration for lack of axial development of the pile wall material near the foundation soffit. Elevations for the RCFST to open pile transition and transition between 1” and 0.75” wall thickness were obtained from the as-built plans. The maximum pile moments occur within the RCFST region. Special attention was given to the effective material properties and moment-curvature response.



Concrete Filled Steel Tube Pile Properties using WSDOT BDM Section 7.10.2

$Es := 29000ksi$

$f'ce := 1.3 \cdot (4ksi) = 5.2 \cdot ksi$

$Ece := 120000 \cdot (1.0) \cdot (0.155)^{2.0} \cdot (5.2)^{0.33} \cdot ksi = 4967.361 \cdot ksi$

$do := 36in$

$t := 1in$

$di := do - 2 \cdot t = 34 \cdot in$

$As := \frac{\pi \cdot do^2}{4} - \frac{\pi \cdot di^2}{4} = 109.956 \cdot in^2$

$Is := \frac{\pi \cdot do^4}{64} - \frac{\pi \cdot di^4}{64} = 16850.718 \cdot in^4$

$Ac := \frac{\pi \cdot di^2}{4} = 907.92 \cdot in^2$

$Ic := \frac{\pi \cdot di^4}{64} = 65597.24 \cdot in^4$

The piles have a large range of axial forces, set P/Po using the dead load axial.

$P := 1000kip$

For now, assume the pipe piles are A252 Gr3 with fy = 45 ksi.

Sensitivity to Pile Designation Investigated Later

$fye := 1.1 \cdot (45ksi) = 49.5 \cdot ksi$

$Po := 0.95 \cdot f'ce \cdot Ac + fye \cdot As = 9927.935 \cdot kip$

$C' := 0.15 + \frac{P}{Po} + \frac{As}{As + Ac} = 0.359$ which is less than the upper bound of 0.9

$EI_{eff} := Es \cdot Is + C' \cdot Ece \cdot Ic = 4.205 \times 10^6 \cdot kip \cdot ft^2$

$I_{ratio} := \frac{EI_{eff}}{Es \cdot Is} = 1.239$ used as a property modifier in the structural model

$A_{ratio} := \frac{As + \frac{Ece}{Es} \cdot Ac}{As} = 2.414$ used as a property modifier in the structural model

Confinement Properties using Mander's Model with Steel Shell

Transverse volumetric steel ratio computed for an equivalent 1in length of steel shell.

$Asp := t \cdot 1in = 1 \cdot in^2$

$sh := 1in$

$\rho s := \frac{4 \cdot Asp}{di \cdot sh} = 0.118$ transverse volumetric steel ratio

$Ke := 1.0$ confinement effectiveness coefficient for a steel shell

$fyh := 45ksi$ for the steel shell

$f'ce = 5.2 \cdot ksi$ for the concrete fill

$f1 := 0.5 \cdot Ke \cdot \rho s \cdot fyh = 2.647 \cdot ksi$ effective lateral confinement pressure from the steel shell

Exceeds the recommended max effective lateral confinement stress for Mander's model.

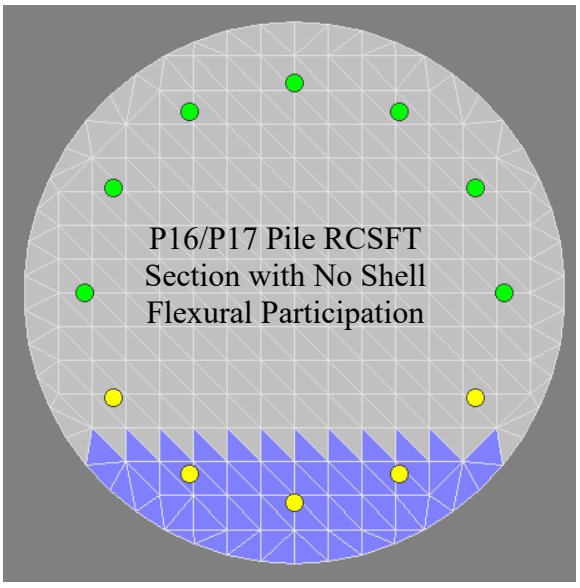
$f1_{use} := 0.3 \cdot f'ce = 1.56 \cdot ksi$ values beyond this point are known to be inaccurate

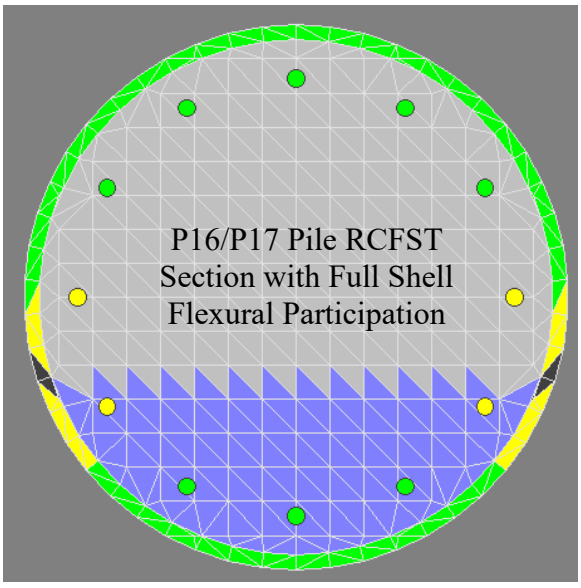
$f'cc := f'ce \cdot \left(-1.254 + 2.254 \cdot \sqrt{1 + \frac{7.94 \cdot f1_{use}}{f'ce}} - 2 \cdot \frac{f1_{use}}{f'ce} \right) = 11.914 \cdot ksi$ ultimate concrete compressive strength

$\varepsilon sm := 0.1$ very conservative assumption for pile material strain at maximum stress

$\varepsilon cu := 0.004 + \frac{1.4 \cdot \rho s \cdot fyh \cdot \varepsilon sm}{f'cc} = 0.066$ very high due to steel shell confinement, use recommended max of 0.02 from AASHTO SGS

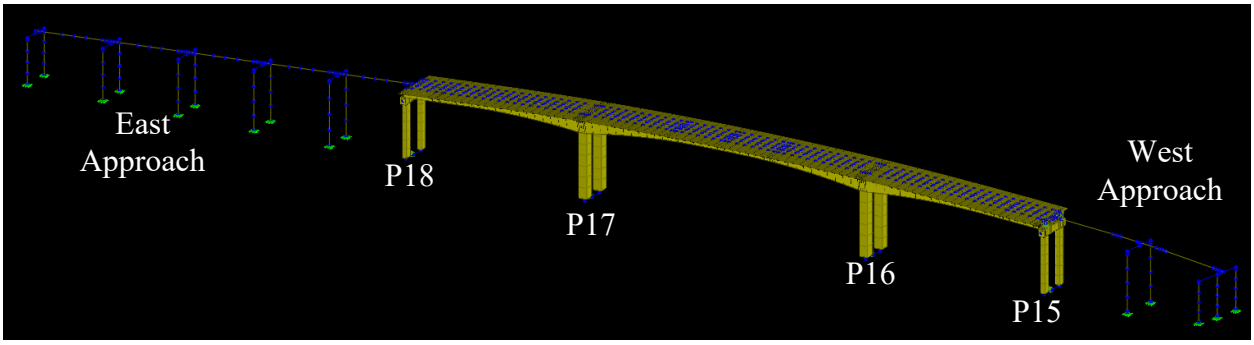
$\varepsilon cu_{use} := 0.02$

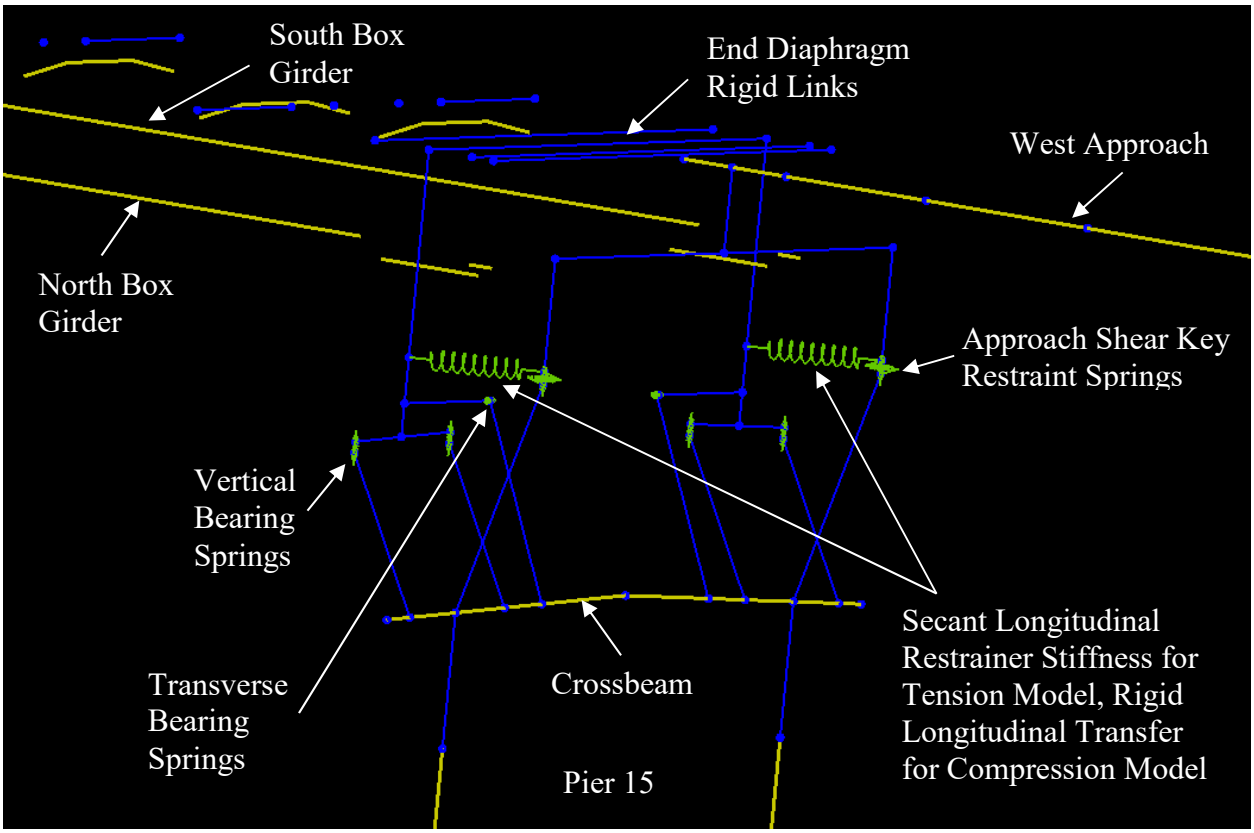




4.2 LINEAR RESPONSE SPECTRUM DEMAND ANALYSIS MODEL

The east and west approach frames were modeled to their first expansion joint to include their effect on the WSHB and Piers 15-18 of interest using tension and compression models in the linear RSA demand analysis. The tension model includes a secant stiffness longitudinal spring at the centroid of the seismic restrainers attributable to each box girder. The process for converging this spring stiffness will be discussed. In the compression model, the spring is made rigid to allow for longitudinal force transfer between the end diaphragms of the box girder and approach span. An overview of the considered analysis cases is presented later. Effective section properties for the columns were determined using moment-curvature analysis. The two-box model was considered, with substitutions for linear secant stiffness 6x6 foundation springs. Procedures used to determine the stiffness matrix terms are discussed in the following section.





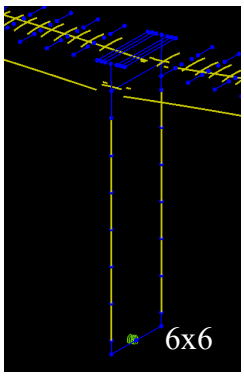
4.2.1 6x6 Foundation Stiffness Matrices

The following process was used to define the 6x6 foundation matrices for linear RSA demand analyses. First, forces at the centroid of the pile-cap foundation were determined corresponding to development of the plastic mechanism with expected material properties. Sub-models of the pile foundation including the lateral P-y soil springs were created with the recommended liquefied and separately non-liquefied soil properties. Procedures from WSDOT BDM Appendix 7-B1 were used to determine the individual stiffness matrix terms including the off-diagonal terms. Each stiffness matrix was then verified to provide an accurate estimate for the pile cap footing centroid deformations in the nonlinear soil spring model under a similar set of demands.

Results from the RSA demand and NLSPO capacity analyses indicated that Piers 15-18 all formed a plastic mechanism at the demand displacement, except for Piers 15 and 18 under longitudinal response which remained elastic due to increased flexibility without moment restraint at the top of the column. The difference between use of a 6x6 spring consistent with behavior at a plastic mechanism at these locations was viewed to be small, due in part to lower forces being attracted under the increased flexibility, proximity of the base moment to formation of a mechanism, and general magnitude of foundation deformations relative to the elastic flexibility of the columns.

Sample 6x6 foundation spring input is shown in the following table. The springs themselves were placed the centroid of the pile cap and connected to the bases of the column using rigid links as shown below.

Translation X (lb/in)	Translation Y (lb/in)	Translation Z (lb/in)	Rotation X (lb/rad)	Rotation Y (lb/rad)	Rotation Z (lb/rad)
5.8014e7	0.0000	0.0000	0.0000	-3.2909e9	0.0000
	5.7118e7	0.0000	-5.7935e8	0.0000	0.0000
		2.6509e8	0.0000	0.0000	0.0000
			2.3185e13	0.0000	0.0000
				1.1580e13	0.0000
					1.0715e11



The following sample calculations step through the WSDOT BDM Appendix 7-B1 process used to determine the stiffness matrix terms. Verifications and a comparison to the use of only the diagonal stiffness matrix terms are provided.

6x6 Spring Stiffness Creation Sheet

P16 Non-Liquified Soil Spring Model at Longitudinal and Transverse Expected Mechanism Forces

All results are specific to the "WSB Zero" Displacement UCS for nodal and spring deformations.

Equilibrium results from the pushover analysis at a full plastic mechanism:

$Fx := 8560kip$ shear along the longitudinal axis

$Fy := 8869kip$ shear along the transverse axis

$Fz := -55646kip$ vertical axial force

$Mx := -1175480kip \cdot ft$ moment about the longitudinal axis

$My := 600423kip \cdot ft$ moment about the transverse axis

$Mz := 1000kip \cdot ft$ torsional moment (not from pushover, but just to get stiffness)

Longitudinal LC1: (restrain y-disp and x-rot transverse behavior)

Apply the lateral load of $Fx = 8560 \cdot kip$

and restrain the footing CG against y-axis rotation using a grounded stiff spring.

$\Delta x := 0.0952272154390812in$

$My_{ind} := 74109.578125kip \cdot ft$ from the grounded spring force

$Vx_{app} := Fx = 8560 \cdot kip$

Longitudinal LC2: (restrain y-disp and x-rot transverse behavior)

Apply the moment load of $My = 600423 \cdot kip \cdot ft$

and restrain the footing CG against x-axis displacement using a grounded stiff spring.

$My_{app} := My = 600423 \cdot kip \cdot ft$

$Vx_{ind} := 5262.69482421875kip$ from the grounded spring force

$\theta y := 0.000563525594770908 \cdot rad$

Transverse LC1: (restrain x-disp and y-rot longitudinal behavior)

Apply the lateral load of $Fy = 8869 \cdot kip$

and restrain the footing CG against x-axis rotation using a grounded stiff spring.

$\Delta y := 0.0967024862766266in$

$Mx_{ind} := -26028.349609375kip \cdot ft$ from the grounded spring force

$Vy_{app} := Fy = 8869 \cdot kip$

Transverse LC2: (restrain x-disp and y-rot longitudinal behavior)

Apply the moment load of $Mx = -1175480 \cdot kip \cdot ft$

and restrain the footing CG against y-axis displacement using a grounded stiff spring.

$Mx_{app} := Mx = -1175480 \cdot kip \cdot ft$

$Vy_{ind} := 1927.83776855469 kip$ from the grounded spring force

$\theta x := -0.000596870435401797 \cdot rad$

Vertical LC: (restrain x/y-disp and x/y-rot long/trans behavior)

Apply $Pz_{app} := Fz = -55646 \cdot kip$ and observe $\Delta z := -0.209300167858601 in$

Torsional LC: (restrain x/y-disp and x/y-rot long/trans behavior)

Apply $Mz_{app} := Mz = 1000 \cdot kip \cdot ft$ and observe $\theta z := 0.000099164477433078 \cdot rad$

Where the linear spring constants or k values are defined as follows:

$$k11 := \frac{Vx_{app}}{\Delta x} = 89890.26887 \cdot \frac{kip}{in}$$ longitudinal lateral stiffness

$$k22 := \frac{Pz_{app}}{\Delta z} = 265866.96308 \cdot \frac{kip}{in}$$ vertical or axial stiffness

$$k33 := \frac{Vy_{app}}{\Delta y} = 91714.29134 \cdot \frac{kip}{in}$$ transverse lateral stiffness

$$k44 := \frac{Mx_{app}}{\theta x} = 23632867643.21705 \cdot \frac{kip \cdot in}{rad}$$ transverse bending or moment stiffness

$$k55 := \frac{Mz_{app}}{\theta z} = 121011074.83875 \cdot \frac{kip \cdot in}{rad}$$ torsional stiffness

$$k66 := \frac{-My_{app}}{-\theta y} = 12785712072.10048 \cdot \frac{kip \cdot in}{rad}$$ longitudinal bending or moment stiffness

$$k34 := \frac{Vy_{ind}}{\theta x} = -3229909.96741 \cdot \frac{kip}{rad}$$ transverse lateral cross-couple term

$$k16 := \frac{Vx_{ind}}{-\theta y} = -9338874.53037 \cdot \frac{kip}{rad}$$ longitudinal lateral cross-couple term

$$k43 := \frac{Mx_{ind}}{\Delta y} = -3229908.63357 \cdot \frac{kip \cdot in}{in}$$ transverse moment cross-couple term

$$k61 := \frac{-My_{ind}}{\Delta x} = -9338873.69697 \cdot \frac{kip \cdot in}{in}$$ longitudinal moment cross-couple term

Note that k34 and k16 should have opposite signs, it was found that k34 should be negative.

Since off diagonal terms must be equal, use the following averages:

$$k16_{avg} := \frac{k16 + k61}{2} = -9338874.11367 \cdot \frac{kip}{rad} \quad k61_{avg} := k16_{avg}$$

$$k34_{avg} := \frac{k34 + k43}{2} = -3229909.30049 \cdot \frac{kip}{rad} \quad k43_{avg} := k34_{avg}$$

note that the original off diagonal terms are nearly equivalent

$$k := \begin{bmatrix} k11 \cdot \left(\frac{kip}{in}\right)^{-1} & 0 & 0 & 0 & k16_{avg} \cdot \left(\frac{kip}{rad}\right)^{-1} & 0 \\ 0 & k33 \cdot \left(\frac{kip}{in}\right)^{-1} & 0 & -k34_{avg} \cdot \left(\frac{kip}{rad}\right)^{-1} & 0 & 0 \\ 0 & 0 & k22 \cdot \left(\frac{kip}{in}\right)^{-1} & 0 & 0 & 0 \\ 0 & -k43_{avg} \cdot \left(\frac{kip}{rad}\right)^{-1} & 0 & k44 \cdot \left(\frac{kip \cdot in}{rad}\right)^{-1} & 0 & 0 \\ k61_{avg} \cdot \left(\frac{kip}{rad}\right)^{-1} & 0 & 0 & 0 & k66 \cdot \left(\frac{kip \cdot in}{rad}\right)^{-1} & 0 \\ 0 & 0 & 0 & 0 & 0 & k55 \cdot \left(\frac{kip \cdot in}{rad}\right)^{-1} \end{bmatrix}$$

The following represent the model deformations under the applied loading (kip and inch units).

$$d_{long} := \begin{pmatrix} 0.166465111076832 \\ 0.000417466930230148 \\ -0.211500830948353 \\ 6.81746783470771E\text{-}08 \\ 0.000597291684243828 \\ 5.42423822480487E\text{-}06 \end{pmatrix} \quad F_{long} := \begin{pmatrix} \frac{Fx}{kip} \\ 0 \\ \frac{Fz}{kip} \\ 0 \\ \frac{My}{kip \cdot in} \\ 0 \end{pmatrix} \quad d_{long.exp} := k^{-1} \cdot F_{long} = \begin{pmatrix} 0.1664 \\ 0 \\ -0.2093 \\ 0 \\ 0.00069 \\ 0 \end{pmatrix}$$

$$d_{trans} := \begin{pmatrix} 0.000375784336938523 \\ 0.118886165320873 \\ -0.209984131157398 \\ -0.000397861876990646 \\ -5.15385352173325E\text{-}08 \\ 0.0000016986909940897 \end{pmatrix} \quad F_{trans} := \begin{pmatrix} 0 \\ \frac{Fy}{kip} \\ \frac{Fz}{kip} \\ \frac{Mx}{kip \cdot in} \\ 0 \\ 0 \end{pmatrix} \quad d_{trans.exp} := k^{-1} \cdot F_{trans} = \begin{pmatrix} 0 \\ 0.11829 \\ -0.2093 \\ -0.00061 \\ 0 \\ 0 \end{pmatrix}$$

Above, d.long and d.trans represent the CG foundation deformations from the pushover response at a full plastic mechanism. F.long and F.trans correspond to the mechanism forces at the CG of the foundation. The expected foundation deformations d.long.exp and d.trans.exp agree well with the observed displacements indicating that the 6x6 stiffness matrix is working as intended. Apply within the model and verify that the same behavior is obtained.

A matrix with only the diagonal terms is shown below for comparison purposes only.

$$k_d := \begin{bmatrix} k11\cdot\left(\frac{kip}{in}\right)^{-1} & 0 & 0 & 0 & 0 & 0 \\ 0 & k33\cdot\left(\frac{kip}{in}\right)^{-1} & 0 & 0 & 0 & 0 \\ 0 & 0 & k22\cdot\left(\frac{kip}{in}\right)^{-1} & 0 & 0 & 0 \\ 0 & 0 & 0 & k44\cdot\left(\frac{kip\cdot in}{rad}\right)^{-1} & 0 & 0 \\ 0 & 0 & 0 & 0 & k66\cdot\left(\frac{kip\cdot in}{rad}\right)^{-1} & 0 \\ 0 & 0 & 0 & 0 & 0 & k55\cdot\left(\frac{kip\cdot in}{rad}\right)^{-1} \end{bmatrix}$$

$$k_d^{-1}\cdot F_{long} = \begin{pmatrix} 0.09523 \\ 0 \\ -0.2093 \\ 0 \\ 0.00056 \\ 0 \end{pmatrix} \qquad k_d^{-1}\cdot F_{trans} = \begin{pmatrix} 0 \\ 0.0967 \\ -0.2093 \\ -0.0006 \\ 0 \\ 0 \end{pmatrix}$$

poorer comparisons to pushover deformations if only the diagonal terms are considered

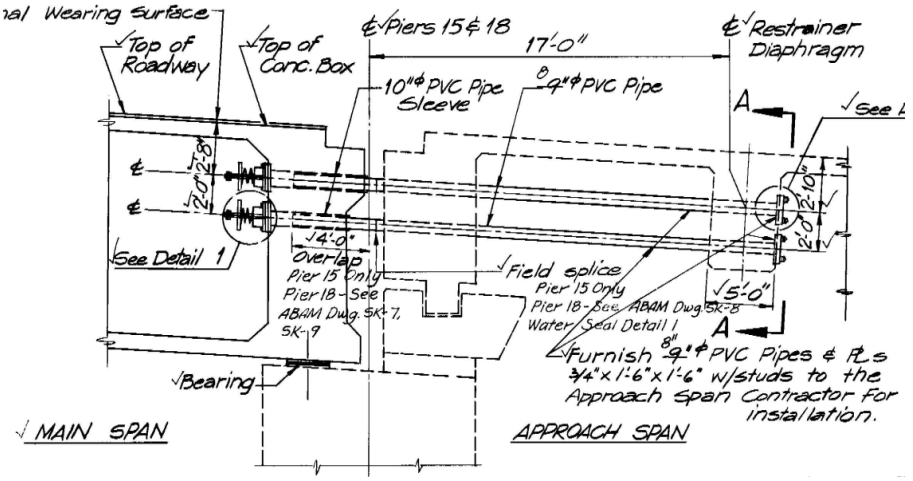
$$k_{input} := \begin{bmatrix} k11\cdot\left(\frac{lbf}{in}\right)^{-1} & 0 & 0 & 0 & k16_{avg}\cdot\left(\frac{lbf}{rad}\right)^{-1} & 0 \\ 0 & k33\cdot\left(\frac{lbf}{in}\right)^{-1} & 0 & -k34_{avg}\cdot\left(\frac{lbf}{rad}\right)^{-1} & 0 & 0 \\ 0 & 0 & k22\cdot\left(\frac{lbf}{in}\right)^{-1} & 0 & 0 & 0 \\ 0 & -k43_{avg}\cdot\left(\frac{lbf}{rad}\right)^{-1} & 0 & k44\cdot\left(\frac{lbf\cdot in}{rad}\right)^{-1} & 0 & 0 \\ k61_{avg}\cdot\left(\frac{lbf}{rad}\right)^{-1} & 0 & 0 & 0 & k66\cdot\left(\frac{lbf\cdot in}{rad}\right)^{-1} & 0 \\ 0 & 0 & 0 & 0 & 0 & k55\cdot\left(\frac{lbf\cdot in}{rad}\right)^{-1} \end{bmatrix}$$

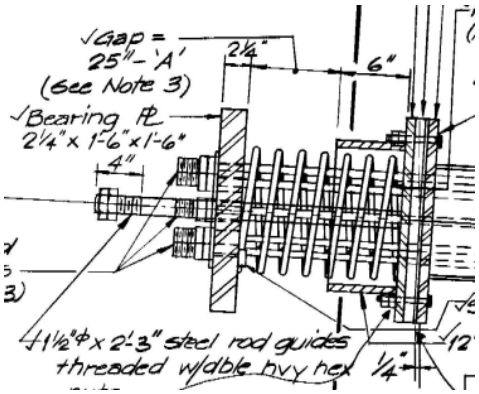
$$k_{input} = \begin{pmatrix} 89890268.87462 & 0 & 0 & 0 & -9338874113.6724 & 0 \\ 0 & 91714291.34334 & 0 & 3229909300.48808 & 0 & 0 \\ 0 & 0 & 265866963.07665 & 0 & 0 & 0 \\ 0 & 3229909300.48808 & 0 & 23632867643217.047 & 0 & 0 \\ -9338874113.6724 & 0 & 0 & 0 & 12785712072100.477 & 0 \\ 0 & 0 & 0 & 0 & 0 & 121011074838.75264 \end{pmatrix}$$

The above stiffness matrix was input into Larsa 4D for equivalent linear analysis.

4.2.2 Longitudinal Restrainer Force vs. Deflection Curve

The nonlinear relationship for restrainers attributable to a single box girder is the same at all four locations. The initial restrainer gap was sized for the mean annual temperature of 55ºF in the analysis. Each restrainer assembly has 9 (3/4" diameter) 6 x 9 galvanized steel cables per 782-138 Sheet 66. The 6 x 9 Classification cannot be found in vendor catalogs, therefore reference for the material properties was obtained for a 6 x 7 galvanized fiber core wire rope. These material properties include the restrainer cable construction stretch (due to core compression with helically laid wires), elastic modulus, proportional limit, and ultimate strength. Analysis results indicate that the seismic restrainers remain elastic, so the results are less sensitive to the assumption of 6 x 7 classification in lieu of 6 x 9.





JOINT DIMENSION A
(see Installation Notes)

Temperature	Conc. Girder		Steel
	Pier 15	Pier 18	Pier 15
40°F	16 ¼"	17 ¼"	18 ½"
55°F	15 ¼"	16"	17 ½"
70°F	14 ¼"	14 ¾"	16 ½"

The following calculations define the considered nonlinear relationship for the seismic restrainers. First the restrainer gap is closed, and the construction stretch is removed as the restrainer goes taught. The restrainer then undergoes linear elastic response to the proportional limit before experiencing a reduced stiffness to ultimate response.

Earthquake Longituidnal Restrainers at Piers 15 and 18

Each restrainer assembly has 9 (3/4" diameter) 6 x 9 Gavlanized Steel Cables per 782-138 Sheet 66
The 6 x 9 Classification cannot be found in vendor catalogs, therefore reference for the breaking force is obtained from the following.

6 x 7 Classification

6 x 7 Classification Wire Ropes give long service in operating conditions where ropes are dragged along the ground or over rollers. Larger sheaves and drums (than those used for more flexible constructions) are required to avoid breakage from fatigue. 6 x 7 Classification Ropes contain 6 strands with 3 through 14 wires, no more than 9 of which are outside wires.

In a 6 x 7 rope, the wires are larger than those of other constructions of the same diameter. Wires of a 6 x 7 are approximately twice the diameter of outer wires of a 6 x 25 FW rope. The large size of the outer wire gives the class excellent abrasion resistance—at great sacrifice of bendability and resistance to fatigue. The standard 6 x 7 rope construction is made with fiber core and right regular lay.

Characteristics 6 x 7 Rope

Excellent abrasion resistance; less bending fatigue resistance.

Typical Applications

Dragging and haulage in mines, inclined planes and tramways, sand lines.

Diameter (in.)	Bright / Fiber Core		Galvanized / Fiber Core		Diameter (in.)
	Approx. Wt. / Ft. (lbs.)	Minimum Breaking Force (Tons of 2,000 lbs.)	Approx. Wt. / Ft. (lbs.)	Minimum Breaking Force (Tons of 2,000 lbs.)	
3/16	0.056	1.5	–	–	3/16
1/4	0.094	2.64	0.094	2.38	1/4
5/16	0.15	4.1	0.15	3.69	5/16
3/8	0.21	5.86	0.21	5.27	3/8
7/16	0.29	7.93	0.29	7.14	7/16
1/2	0.38	10.3	0.38	9.27	1/2
9/16	0.48	13	0.48	11.7	9/16
5/8	0.59	15.9	0.59	14.3	5/8
3/4	0.84	22.7	0.84	20.4	3/4

Min Cable Breaking Force	20.4 tons	40.8 kips	for each cable	6x7 results in a lower breaking force than if the restrainer had been 6x19 IWRC
Number Cables	9			
Min Breaking (Tensile) Force	367.2 kips	for each restrainer contining nine cables		
Galvanized Strength Ratio	0.90	matches doccumentation of 10% strenght loss due to reduced area prior to galvanization		

Metallic Area/Elastic Stretch Calculator

Approximate Metallic Areas

Rope Core: Fiber Core

Rope Construction: 6x7

Rope Diameter (in.): 0.75 /

Approx. Area (sq. in.): 0.216

Check on Calculator: 6x19 (S) IWRC (1" dia) Calculated Area of 0.47 in^2 matches Doccumentation

Ungalvanized Approx. Area	0.216	in^2	
Galvanized Area Ratio	0.90	assume the 10% loss in strength corresponds to 10% loss in metalic area prior to galvanization	
Galvanized Approx. Area	0.194	in^2	
Area All Cables in Restrainer	1.747	in^2	

Constructional Stretch

When a load is applied to wire rope, the helically-laid wires and strands act in a constricting manner thereby compressing the core and bringing all the rope elements into closer contact. The result is a slight reduction in diameter and an accompanying lengthening of the rope.

Constructional stretch is influenced by the following factors:

1) type of core (fiber or steel),
2) rope construction (6x7, 6x25 FW, 6x41 WS, 8x19 S, etc.),
3) length of lay,
4) material.

Ropes with wire strand core (WSC) or independent wire rope core (IWRC) have less constructional stretch than those with fiber core (FC). The reason for this is the fact that the steel cannot compress as much as the fiber core.

Usually, constructional stretch will cease at an early stage in the rope's life. However, some fiber core ropes, if lightly loaded (as in the case of elevator ropes), may display a degree of constructional stretch over a considerable portion of their life.

A definite value for determining constructional stretch cannot be assigned since it is influenced by several factors. The following table gives some idea of the approximate stretch as a percentage of rope under load.

Rope Construction	Approx. Stretch*
6 strand FC	1/2% - 3/4%
6 strand IWRC	1/4 % - 1/2%
8 strand FC	3/4% - 1%

* Varies with the magnitude of the loading.

Elastic Stretch

Elastic stretch results from recoverable deformation of the metal itself. Here, again, a quantity cannot be precisely calculated. However, the following equation can provide a reasonable approximation for a good many situations.

$$\text{Changes in length (ft)} = \frac{\text{Change in load (lb) x Length (ft)}}{\text{Area (inches}^2\text{) x Modulus of Elasticity (psi)}}$$

The modulus of elasticity is given below.

Approximate Modulus of Elasticity (lbs. per square Inch)

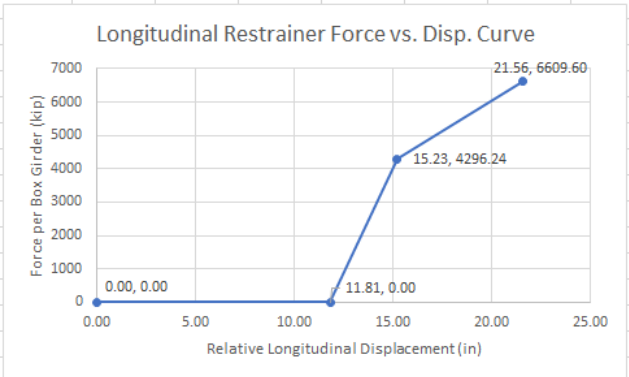
Rope Classification	Zero through 20% Loading	21 to 65% Loading*
6 x 7 with fiber core	11,700,000	13,000,000
6 x 19 with fiber core	10,800,000	12,000,000
6 x 37 with fiber core	9,900,000	11,000,000
8 x 19 with fiber core	8,100,000	9,000,000
6 x 19 with IWRC	13,500,000	15,000,000
6 x 37 with IWRC	12,600,000	14,000,000

* Applicable to new rope, i.e., not previously loaded.

Therefore, there is an initial strain due to construction stretch and an elastic stretch which uses the Approximate Modulus of Elasticity				
Construction Stretch Strain	0.0075	initial strain due to loading		
Approx. Modulus of Elasticity	13000	ksi	noted to apply to 65% of the minimum breaking (tensile) strength	
Therefore, consider 65% of the minimum breaking (tensile) strength as the proportional limit (yield) of the restrainers.				
Proportional (Yield) Limit	238.68	kips	for each restrainer contining nine cables	
Average Longitudinal Restrainer Gap from the Plans				
For the purposes of determining the restrainer stiffness in the Tension Model, use the average Restrainer Gap at 55 deg from 782-138 Sheets 66 and 97				
From Sheet 66, Gap = 25 in - "A" Dimension from Sheet 99				
Avg "A" Dimension P15/P18	15.625	in	at 55 deg, which is approximately the average annual temperature for Seattle from the TU and TG Study	
Average Restrainer Gap	9.375	in	average displacement needed to engage the longitudinal seismic restrainers	
Longitudinal Restrainer Length from the Plans				
Length Pier 15/18 CL to Diaph	234	in	19.5	ft
(Diaphragm + Restrainer) Gap	25	in		
Box Girder End Diaphragm	66	in	5.5	ft
Total Restrainer Length	325	in	27.08	ft
Calculate the Force-Displacement Curve for the Longitudinal Restrainers				
Number Restrainers per Box	18	pe box	from 782-138 Sheet 59	
Restrainer Yield Force per Box	4296.24	kips		
Restrainer Ult. Force per Box	6609.6	kips		
Gap to Engage Restrainer	9.375	in		
Total Length of Restrainer	325	in		
Construction Stretch Strain	0.0075		initial strain due to loading	
Initial Construction Stretch	2.4375	in	to tighten steel wire due to load	
Gap to Engage + Const. Stretch	11.8125	in	average longitudinal displacement before loading the restrainers	
Approx. Elastic Modulus	13000	ksi		
Restrainer Steel Area per Box	31.45	in^2		
Elasitc Strain at Yield	0.010509		at the proportional limit (yield) defined by 65% of the minimum breaking force	
Elastic Stretch at Yield	3.415509	in	at the proportional limit (yield)	
Assumed Ult Tensile Strain	0.03		based on limited available information for IPS steel wire	
Ultimate Tensile Stretch	9.75	in		

Construct the Force-Displacement Curve for the Longitudinal Restrainers (per box)

Disp. (in)	Force (kips)	Cable Behavior
0.00	0	initial
11.81	0	taught
15.23	4296	yield
21.56	6610	ultimate



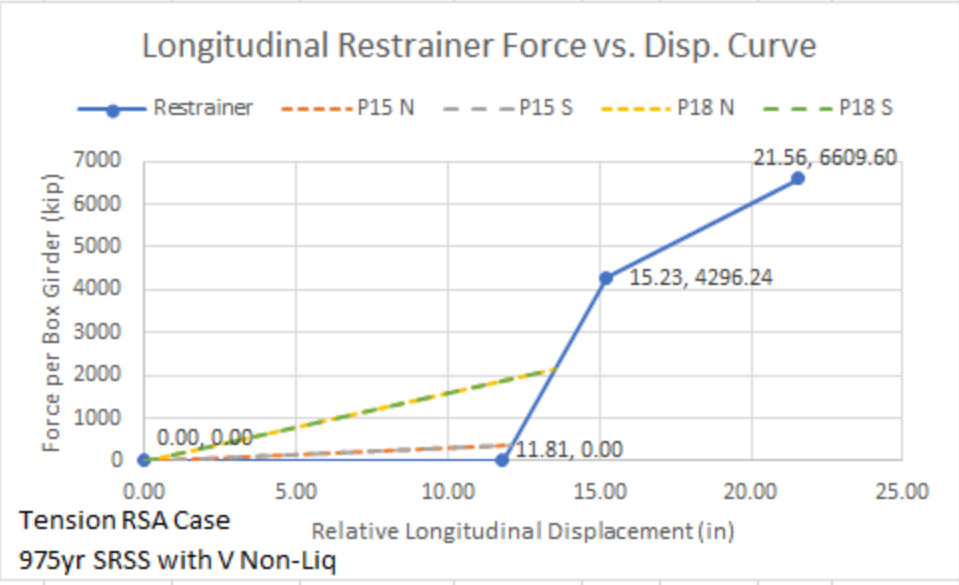
Apply longitudinal restrainers at P15 and P18 North South girder locations between end diaphragms.
Therefore, there are four restrainers that require secant stiffness convergence in a typical tension model RSA run.
Unique convergence is required for each RSA case.

The restrainer group CG is centered laterally along the box and is located 9 ft below the roadway surface per 782-138 Sheet 58.

Results indicate the seismic restrainers remain elastic in the 975yr SEE, as shown by the sample convergence process and resulting secant stiffness springs. Note that a unique convergence was obtained in each tension model case, including those with low longitudinal displacements which did not engage the seismic restrainers.

Iterate until covnergence is achieved, defined by Δ.ratio (assumed/predicted) within 15%

Iteration #1 Input			Interpolate	(lbf/in)		**Iteration #1 Results**	
Location	Spring	Δ iter (in)	Force (kip)	K secant	Δ.ratio	Spring	Δx (in)
P15 North	283	11.925	142	11867	0.77	283	15.44
P15 South	284	11.925	142	11867	0.73	284	16.30
P18 North	285	13.5	2123	157233	1.09	285	12.39
P18 South	286	13.5	2123	157233	1.07	286	12.58
Iteration #2 Input			Interpolate	(lbf/in)		**Iteration #2 Results**	
Location	Spring	Δ iter (in)	Force (kip)	K secant	Δ.ratio	Spring	Δx (in)
P15 North	283	12.1	362	29887	1.02	283	11.91
P15 South	284	12.1	362	29887	0.94	284	12.86
P18 North	285	13.55	2186	161294	1.25	285	10.86
P18 South	286	13.55	2186	161294	1.21	286	11.15
Iteration #3 Results			Interpolate	(lbf/in)		**Iteration #3 Results**	
Location	Spring	Δ iter (in)	Force (kip)	K secant	Δ.ratio	Spring	Δx (in)
P15 North	283	12.1	362	29887	1.02	283	11.89
P15 South	284	12.1	362	29887	0.99	284	12.19
P18 North	285	13.52	2148	158861	1.05	285	12.90
P18 South	286	13.52	2148	158861	1.03	286	13.11



4.2.3 Mode Shapes and Cumulative Mass Participation

Unique mode shapes and modal periods are obtained based on tension/compression model behavior and whether liquefied or non-liquefied soil properties were considered. An adequate number of modes was considered to obtain 90% cumulative mass participation in the two lateral and vertical directions. Since the pile cap foundation mass was included, this required consideration of 400 modes to reach the target vertical mass participation, although inclusion of this mass has little effect on the overall results. Analysis times were fairly unsensitive to the number of modes considered. The following sample results are presented for a tension and compression model case with liquefied soil properties and 6x6 foundation spring matrices consistent with development of the expected plastic mechanism.

Model with Basic Piles (Tension Model 975yr SRSS with V Liquified Soil Properties)					
Mode Shape	**Period (s)**	**Trans Mass**	**Long Mass**	**Vert Mass**	
M1: f = 0.35, t = 2.8467	2.847	0.0	59.7	0.0	Fundamental Longitudinal
M2: f = 0.42, t = 2.3976	2.398	24.8	59.7	0.0	Fundamental Transverse
M3: f = 0.45, t = 2.2445	2.245	25.2	67.2	0.0	
M4: f = 0.45, t = 2.2273	2.227	33.9	71.2	0.0	
M5: f = 0.47, t = 2.1227	2.123	76.8	71.4	0.0	
M6: f = 0.49, t = 2.0541	2.054	76.8	76.3	0.0	
M7: f = 0.58, t = 1.7331	1.733	76.8	76.3	0.9	
M8: f = 0.59, t = 1.6971	1.697	76.8	76.3	0.9	
M9: f = 0.79, t = 1.2627	1.263	80.3	76.5	0.9	
M10: f = 0.93, t = 1.0711	1.071	80.3	79.8	0.9	
M11: f = 1.13, t = 0.8863	0.886	80.3	79.8	0.9	
M12: f = 1.16, t = 0.8612	0.861	80.3	79.8	0.9	
M13: f = 1.2, t = 0.8346	0.835	80.3	79.8	21.9	Fundamental Vertical
M14: f = 1.41, t = 0.7102	0.710	80.4	79.8	22.0	
M15: f = 1.42, t = 0.7023	0.702	80.4	79.8	22.3	
M16: f = 1.55, t = 0.6460	0.646	80.4	79.8	22.3	
M17: f = 1.78, t = 0.5627	0.563	80.4	80.0	22.3	
M18: f = 1.93, t = 0.5174	0.517	80.4	80.0	23.2	
M19: f = 1.98, t = 0.5048	0.505	80.4	80.1	23.3	
M20: f = 2.06, t = 0.4853	0.485	80.4	80.1	23.6	
M21: f = 2.11, t = 0.4744	0.474	80.4	80.1	23.6	
M22: f = 2.12, t = 0.4718	0.472	80.4	80.1	23.6	
M23: f = 2.13, t = 0.4700	0.470	80.4	80.1	24.4	
M24: f = 2.2, t = 0.4538	0.454	80.4	80.1	24.4	
M25: f = 2.22, t = 0.4496	0.450	80.4	80.1	24.4	
M26: f = 2.42, t = 0.4136	0.414	80.4	80.1	24.4	
...	...	...	...	...	
M400: f = 47.38, t = 0.0211	0.021	96.2	99.1	89.7	Mass Participation Target

Model with Basic Piles (Compression Model 975yr SRSS with V Liquified Soil Properties)					
Mode Shape	Period (s)	Trans Mass	Long Mass	Vert Mass	
M1: f = 0.39, t = 2.5749	2.575	0.0	76.3	0.0	Fundamental Longitudinal
M2: f = 0.42, t = 2.3945	2.394	30.9	76.4	0.0	Fundamental Transverse
M3: f = 0.48, t = 2.1001	2.100	69.4	76.4	0.0	
M4: f = 0.55, t = 1.8260	1.826	78.5	76.4	0.0	
M5: f = 0.57, t = 1.7402	1.740	78.5	76.4	0.9	
M6: f = 0.68, t = 1.4608	1.461	79.8	76.5	0.9	
M7: f = 0.93, t = 1.0754	1.075	79.8	79.8	0.9	
M8: f = 0.96, t = 1.0423	1.042	80.3	79.8	0.9	
M9: f = 1.09, t = 0.9169	0.917	80.3	79.8	7.1	Fundamental Vertical
M10: f = 1.16, t = 0.8655	0.866	80.3	79.8	17.7	Fundamental Vertical
M11: f = 1.33, t = 0.7536	0.754	80.3	79.8	19.1	
M12: f = 1.38, t = 0.7222	0.722	80.3	79.9	19.1	
M13: f = 1.49, t = 0.6719	0.672	80.3	79.9	19.9	
M14: f = 1.73, t = 0.5770	0.577	80.3	80.0	19.9	
M15: f = 1.8, t = 0.5557	0.556	80.3	80.1	19.9	
M16: f = 1.85, t = 0.5391	0.539	80.4	80.1	19.9	
M17: f = 1.9, t = 0.5257	0.526	80.4	80.1	19.9	
M18: f = 1.99, t = 0.5024	0.502	80.4	80.1	20.0	
M19: f = 2.07, t = 0.4833	0.483	80.4	80.1	20.1	
M20: f = 2.11, t = 0.4742	0.474	80.4	80.1	20.1	
M21: f = 2.16, t = 0.4631	0.463	80.4	80.1	20.1	
M22: f = 2.2, t = 0.4536	0.454	80.4	80.1	20.1	
M23: f = 2.21, t = 0.4521	0.452	80.4	80.1	22.3	
M24: f = 2.44, t = 0.4106	0.411	80.4	80.2	22.8	
M25: f = 2.47, t = 0.4052	0.405	80.4	80.2	22.8	
...	...	...	...	...	
M400: f = 49.29, t = 0.0203	0.020	95.5	99.4	89.2	Mass Participation Target

4.2.4 Modal and Directional Combination Rules

Modal combination rules are approximations introduced in combining the peaks of all modal responses determined using the earthquake response spectra. The Complete Quadratic Combination (CQC) modal combination rule was used in all analyses. The horizontal spectra were oriented in the longitudinal and transverse directions of the WSHB. Note that due to the curved geometry of the west approach, this does not correspond to the local longitudinal axes of each bent. Both the square root of the sum of the squares (SRSS) and 100-30 directional combination rules were used to determine the peak deformation demands, both longitudinal and transverse, for each pier. The following section provides a complete list of RSA cases considered and their results.

5.0 DISPLACEMENT DEMANDS

As discussed, both tension and compression models were considered to bound the effects of expansion joint and restrainer behavior at Piers 15 and 18. The following RSA cases were considered in the evaluation of displacement demands for the 975yr SEE and 210yr FEE site-specific ground motions. Evaluations for the 975yr SEE included both liquefied and non-liquefied soil and considered cases both including and excluding vertical accelerations. For the 210yr FEE, only non-liquefied soil conditions were considered. Based on the comparison of displacement demands and capacities including both liquefied and non-liquefied properties in the SEE, this assumption is not expected to significantly impact the results for the FEE event where the effects of potential liquefaction would be reduced.

5.1 975YR SEE DISPLACEMENT DEMANDS WITH LIQUEFIED SOIL

Cases 10, 12, and 14 were conservatively evaluated with a compression model as well since longitudinal displacement may engage pounding between end diaphragms at Pier 15 and 18 under some thermal conditions. No restrainer stiffness was considered for cases 9, 11, and 13, since the longitudinal displacements were not large enough to close to restrainer gap.

Case #	Hazard	Directional Combination	Modeled Foundation?	Liquified?	Tension or Comp Model?	Converged Restrainer?
1	975	$\sqrt{(L^2+T^2)}$	6x6	Liquified	Tension	Yes
2	975	$\sqrt{(L^2+T^2)}$	6x6	Liquified	Compression	Rigid Transfer
3	975	$\sqrt{(L^2+T^2+V^2)}$	6x6	Liquified	Tension	Yes
4	975	$\sqrt{(L^2+T^2+V^2)}$	6x6	Liquified	Compression	Rigid Transfer
5	975	1.0L+0.3T	6x6	Liquified	Tension	Yes
6	975	1.0L+0.3T	6x6	Liquified	Compression	Rigid Transfer
7	975	1.0L+0.3T+0.3V	6x6	Liquified	Tension	Yes
8	975	1.0L+0.3T+0.3V	6x6	Liquified	Compression	Rigid Transfer
9	975	0.3L+1.0T	6x6	Liquified	Tension	No, Open Gap
10	975	0.3L+1.0T	6x6	Liquified	Compression	Rigid Transfer
11	975	0.3L+1.0T+0.3V	6x6	Liquified	Tension	No, Open Gap
12	975	0.3L+1.0T+0.3V	6x6	Liquified	Compression	Rigid Transfer
13	975	0.3L+0.3T+1.0V	6x6	Liquified	Tension	No, Open Gap
14	975	0.3L+0.3T+1.0V	6x6	Liquified	Compression	Rigid Transfer

In the table below, North and South reference the column locations rather than the directions of the displacement demand. The governing results for each pier are tabulated considering all RSA cases and column displacements.

Deformation Demands for 975yr Cases with Liquefied Soil Conditions (inches at the top of the column, i.e. the superstructure soffit or crossbeam)																
	Pier 15				Pier 16				Pier 17				Pier 18			
	Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse	
Case #	North	South	North	South	North	South	North	South	North	South	North	South	North	South	North	South
1	11.39	12.65	18.71	18.64	12.83	12.75	13.69	13.71	12.87	12.95	11.05	11.05	19.85	20.31	13.63	13.61
2	13.56	13.81	11.36	11.35	14.77	14.94	12.68	12.68	14.97	15.21	13.2	13.19	13.77	14.18	14.69	14.68
3	11.14	12.88	19.13	19.04	12.86	12.7	13.45	13.46	12.7	12.8	10.9	10.9	20.69	20.99	14.29	14.27
4	13.33	13.61	11.08	11.07	14.57	14.76	12.61	12.6	14.69	14.98	13.22	13.21	13.55	14.05	14.45	14.44
5	11.16	11.96	5.46	5.42	12.4	12.38	3.86	3.86	12.48	12.47	3.02	3.02	20.1	20.26	3.66	3.65
6	13.36	13.53	3.53	3.53	14.44	14.52	3.8	3.8	14.86	14.92	3.79	3.79	13.55	13.67	3.98	3.97
7	11.35	12.27	5.46	5.41	12.49	12.41	3.74	3.75	12.49	12.48	3.18	3.18	20.03	20.19	4.36	4.36
8	13.65	14.8	3.29	3.46	14.8	14.89	3.46	3.46	14.87	14.98	3.47	3.47	13.76	13.93	4.11	4.1
9	5.97	5.98	18.88	18.83	4.69	4.92	14.28	14.28	4.67	5.13	10.94	10.93	5.58	5.61	13.97	13.97
10	4.31	4.46	11.35	11.33	4.69	4.89	12.53	12.53	4.68	5.06	13.09	13.08	4.38	4.95	15.15	15.13
11	5.72	5.96	17.52	17.46	4.49	4.72	13.32	13.33	4.32	4.92	11.38	11.36	5.53	5.56	17.25	17.25
12	4.48	4.66	11.43	11.42	4.74	4.91	12.75	12.74	4.83	5.2	13.25	13.25	4.51	5.06	14.94	14.92
13	4.52	4.74	5.3	5.28	4.28	4.33	4.02	4.03	4.23	4.31	3.54	3.54	5.28	5.31	4.4	4.4
14	4.54	4.62	3.44	3.43	4.64	4.71	3.83	3.83	4.61	4.73	3.87	3.87	4.45	4.62	4.31	4.3
	Pier 15				Pier 16				Pier 17				Pier 18			
Direction	Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse	
Column	North	South	North	South	North	South	North	South	North	South	North	South	North	South	North	South
Maximum	13.65	14.8	19.13	19.04	14.8	14.94	14.28	14.28	14.97	15.21	13.25	13.25	20.69	20.99	17.25	17.25
Maximum	14.8		19.13		14.94		14.28		15.21		13.25		20.99		17.25	

5.2 975YR SEE DISPLACEMENT DEMANDS WITH NON-LIQUEFIED SOIL

Cases 10, 12, and 14 were conservatively evaluated with a compression model as well since longitudinal displacement may engage pounding between end diaphragms at Pier 15 and 18 under some thermal conditions. No restrainer stiffness was considered for cases 9, 11, and 13, since the longitudinal displacements were not large enough to close to restrainer gap.

Case #	Hazard	Directional Combination	Modeled Foundation?	Liquified?	Tension or Comp Model?	Converged Restrainer?
1	975	$\sqrt{(L^2+T^2)}$	6x6	Non-Liq	Tension	Yes
2	975	$\sqrt{(L^2+T^2)}$	6x6	Non-Liq	Compression	Rigid Transfer
3	975	$\sqrt{(L^2+T^2+V^2)}$	6x6	Non-Liq	Tension	Yes
4	975	$\sqrt{(L^2+T^2+V^2)}$	6x6	Non-Liq	Compression	Rigid Transfer
5	975	1.0L+0.3T	6x6	Non-Liq	Tension	Yes
6	975	1.0L+0.3T	6x6	Non-Liq	Compression	Rigid Transfer
7	975	1.0L+0.3T+0.3V	6x6	Non-Liq	Tension	Yes
8	975	1.0L+0.3T+0.3V	6x6	Non-Liq	Compression	Rigid Transfer
9	975	0.3L+1.0T	6x6	Non-Liq	Tension	No, Open Gap
10	975	0.3L+1.0T	6x6	Non-Liq	Compression	Rigid Transfer
11	975	0.3L+1.0T+0.3V	6x6	Non-Liq	Tension	No, Open Gap
12	975	0.3L+1.0T+0.3V	6x6	Non-Liq	Compression	Rigid Transfer
13	975	0.3L+0.3T+1.0V	6x6	Non-Liq	Tension	No, Open Gap
14	975	0.3L+0.3T+1.0V	6x6	Non-Liq	Compression	Rigid Transfer

In the table below, North and South reference the column locations rather than the directions of the displacement demand. The governing results for each pier are tabulated considering all RSA cases and column displacements.

Deformation Demands for 975yr Cases with Non-Liquefied Soil Conditions (inches at the top of the column, i.e. the superstructure soffit or crossbeam)																
	Pier 15				Pier 16				Pier 17				Pier 18			
	Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse	
Case #	North	South	North	South	North	South	North	South	North	South	North	South	North	South	North	South
1	11.12	12.48	16.91	16.84	12.56	12.48	13.57	13.58	12.56	12.53	11.74	11.74	20.98	21.33	13.26	13.25
2	13.55	13.8	11.11	11.1	14.75	14.93	12.49	12.48	15.02	15.3	13.04	13.03	13.77	14.23	14.45	14.44
3	12.41	13.44	16.01	15.94	12.35	12.27	13.45	13.45	12.54	12.49	11.45	11.45	20.93	21.28	12.88	12.87
4	13.67	13.93	11.23	11.21	14.84	14.99	12.66	12.65	15.18	15.44	13.06	13.05	13.88	14.32	14.22	14.2
5	11.82	12.41	4.75	4.72	12.19	12.12	3.39	3.39	12.39	12.41	2.98	2.98	20.56	20.66	4.45	4.45
6	13.28	13.47	3.44	3.43	14.43	14.53	3.59	3.59	14.65	14.77	3.52	3.52	13.4	13.59	3.87	3.86
7	11.84	12.51	5.15	5.11	12.38	12.21	3.61	3.62	12.38	12.41	2.89	2.89	20.4	20.49	3.88	3.88
8	13.59	13.76	3.4	3.4	14.65	14.73	3.64	3.63	14.95	15.02	3.64	3.64	13.72	13.85	3.97	3.97
9	5.11	5.48	17.51	17.5	4.83	4.93	13.36	13.36	4.61	4.96	11.61	11.6	5.33	5.4	14.2	14.19
10	4.35	4.51	11.47	11.46	4.74	4.9	12.85	12.84	4.76	5.11	12.98	12.97	4.4	4.94	14.57	14.55
11	5.15	5.53	17.52	17.47	4.87	4.97	13.37	13.37	4.65	5	11.61	11.6	5.38	5.44	14.21	14.2
12	4.75	4.87	11.41	11.4	5.06	5.19	12.65	12.64	5.09	5.4	13	13	4.77	5.24	14.57	14.55
13	4.15	4.54	5.29	5.27	4.47	4.5	4.22	4.22	4.46	4.54	3.41	3.41	5.32	5.35	3.53	3.53
14	4.49	4.58	3.4	3.39	4.6	4.66	3.8	3.8	4.57	4.67	3.93	3.93	4.4	4.56	4.28	4.28
	Pier 15				Pier 16				Pier 17				Pier 18			
Direction	Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse	
Column	North	South	North	South	North	South	North	South	North	South	North	South	North	South	North	South
Maximum	13.67	13.93	17.52	17.5	14.84	14.99	13.57	13.58	15.18	15.44	13.06	13.05	20.98	21.33	14.57	14.55
Maximum	13.93		17.52		14.99		13.58		15.44		13.06		21.33		14.57	

5.3 210YR FEE DISPLACEMENT DEMANDS WITH NON-LIQUEFIED SOIL

Cases 10, 12, and 14 are not applicable since longitudinal deformation demands did not close the gap in the end diaphragms at Piers 15 and 18. Tension model cases did not include the restrainer stiffness, since none of the 210yr displacement demands were found to close the restrainer gap.

Case #	Hazard	Directional Combination	Modeled Foundation?	Liquified?	Tension or Comp Model?	Converged Restrainer?
1	210	$\sqrt{(L^2+T^2)}$	6x6	Non-Liq	Tension	No, Open Gap
2	210	$\sqrt{(L^2+T^2)}$	6x6	Non-Liq	Compression	Pounding Ends
3	210	$\sqrt{(L^2+T^2+V^2)}$	6x6	Non-Liq	Tension	No, Open Gap
4	210	$\sqrt{(L^2+T^2+V^2)}$	6x6	Non-Liq	Compression	Pounding Ends
5	210	1.0L+0.3T	6x6	Non-Liq	Tension	No, Open Gap
6	210	1.0L+0.3T	6x6	Non-Liq	Compression	Pounding Ends
7	210	1.0L+0.3T+0.3V	6x6	Non-Liq	Tension	No, Open Gap
8	210	1.0L+0.3T+0.3V	6x6	Non-Liq	Compression	Pounding Ends
9	210	0.3L+1.0T	6x6	Non-Liq	Tension	No, Open Gap
10	210	0.3L+1.0T	6x6	Non-Liq	Compression	Not Applicable
11	210	0.3L+1.0T+0.3V	6x6	Non-Liq	Tension	No, Open Gap
12	210	0.3L+1.0T+0.3V	6x6	Non-Liq	Compression	Not Applicable
13	210	0.3L+0.3T+1.0V	6x6	Non-Liq	Tension	No, Open Gap
14	210	0.3L+0.3T+1.0V	6x6	Non-Liq	Compression	Not Applicable

In the table below, North and South reference the column locations rather than the directions of the displacement demand. The governing results for each pier are tabulated considering all RSA cases and column displacements.

Deformation Demands for 210yr Cases with Non-Liquefied Soil Conditions (inches at the top of the column, i.e. the superstructure soffit or crossbeam)																
	Pier 15				Pier 16				Pier 17				Pier 18			
	Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse	
Case #	North	South	North	South	North	South	North	South	North	South	North	South	North	South	North	South
1	5	5.44	5.99	5.97	4.99	4.98	4.82	4.82	4.93	5.01	4.08	4.08	5.73	5.77	4.43	4.43
2	4.68	4.76	4.25	4.24	5.1	5.17	4.7	4.7	5.16	5.27	4.69	4.69	4.73	4.91	5.11	5.1
3	4.86	5.2	6.67	6.64	4.99	5	5.04	5.05	5.06	5.18	3.96	3.96	5.92	5.94	4.62	4.62
4	4.65	4.74	4.36	4.35	5.06	5.13	4.77	4.77	5.12	5.22	4.82	4.82	4.68	4.87	5.13	5.12
5	4.84	5.11	1.6	1.59	4.9	4.87	1.25	1.25	4.9	4.9	1.19	1.19	5.6	5.62	1.4	1.4
6	4.68	4.76	4.25	4.24	5.1	5.17	4.7	4.7	5.16	5.27	4.69	4.69	4.73	4.91	5.11	5.1
7	4.75	4.98	1.85	1.84	4.91	4.89	1.33	1.33	4.98	5.02	1.03	1.03	5.81	5.81	1.38	1.38
8	4.68	4.75	1.31	1.31	5.06	5.08	1.38	1.38	5.1	5.12	1.36	1.36	4.7	4.76	1.45	1.45
9	1.76	2.02	6.28	6.26	1.6	1.61	5.03	5.03	1.52	1.62	4.13	4.13	1.87	1.9	4.43	4.43
10																
11	1.84	1.96	6.85	6.83	1.63	1.66	5.24	5.24	1.64	1.77	4.13	4.12	1.95	1.96	4.66	4.65
12																
13	1.55	1.65	2.01	2	1.58	1.58	1.52	1.52	1.6	1.63	1.2	1.2	1.86	1.87	1.4	1.4
14																
	Pier 15				Pier 16				Pier 17				Pier 18			
Direction	Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse		Longituidnal		Transverse	
Column	North	South	North	South	North	South	North	South	North	South	North	South	North	South	North	South
Maximum	5	5.44	6.85	6.83	5.1	5.17	5.24	5.24	5.16	5.27	4.82	4.82	5.92	5.94	5.13	5.12
Maximum	5.44		6.85		5.17		5.24		5.27		4.82		5.94		5.13	

5.4 SUMMARY OF GOVERNING 975YR SEE AND 210YR FEE DEMANDS

The following table summarizes the 975yr SEE and 210yr FEE displacement demands corresponding to the recommended site-specific horizontal and vertical spectra provided by Shannon and Wilson. Results are compared with those from the preliminary Site Class D spectra to confirm that the site-specific ground motions govern. Note that the preliminary Site Class D displacement demands exceeded the site-specific results for the 210yr FEE, but this did not change to result since the piers remain elastic under both motions, as will be discussed. Demands from the site-specific evaluation supersede the preliminary Site Class D results.

Site-Specific	975yr SEE, Liquefied		975yr SEE, Non-Liquefied		210yr FEE, Non-Liquefied	
Pier	Δlong (in)	Δtrans (in)	Δlong (in)	Δtrans (in)	Δlong (in)	Δtrans (in)
15	14.8	19.13	13.93	17.52	5.44	6.85
16	14.94	14.28	14.99	13.58	5.17	5.24
17	15.21	13.25	15.44	13.06	5.27	4.82
18	20.99	17.25	21.33	14.57	5.94	5.13

Site Class D	975yr SEE, Liquefied		975yr SEE, Non-Liquefied		210yr FEE, Non-Liquefied	
Pier	Δlong (in)	Δtrans (in)	Δlong (in)	Δtrans (in)	Δlong (in)	Δtrans (in)
15	12.83	16.27	13.08	15.89	5.91	7.15
16	13.87	12.65	14.1	12.19	6.37	5.69
17	14.23	11.54	14.44	11.25	6.47	5.28
18	20.22	13.23	20.49	12.92	8.19	6.04

6.0 DUCTILITY DEMANDS

This section summarizes the results of the nonlinear static pushover analyses and provides a comparison relative to the RSA displacement demands. Of interest are the computed displacement ductility demands relative to the WSDOT BDM limits for an Essential or Critical structure summarized in the following table. Displacement ductility demands were computed using AASHTO SGS Equation C4.9-6, also shown below.

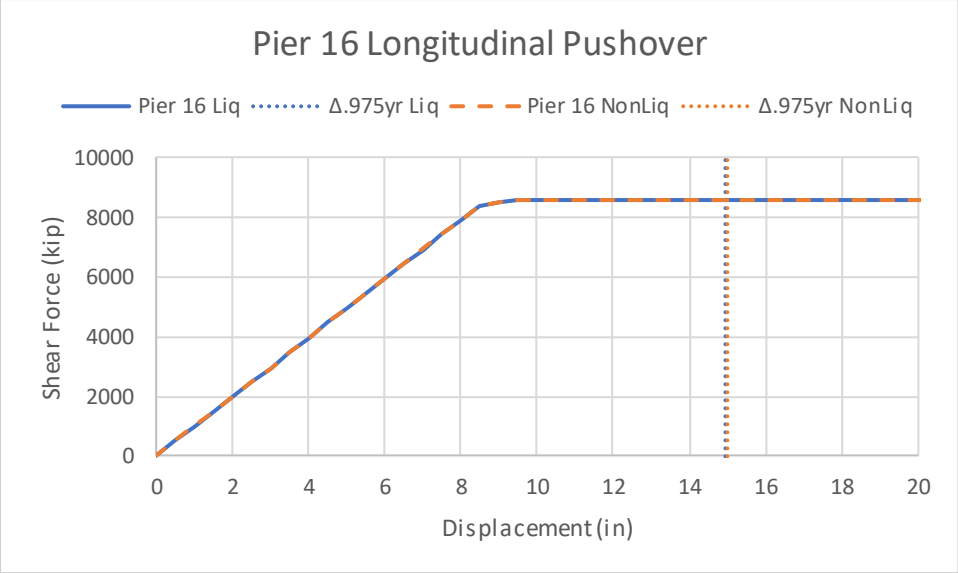
Table 4.1-2 Displacement Ductility Demand Values, μ_D

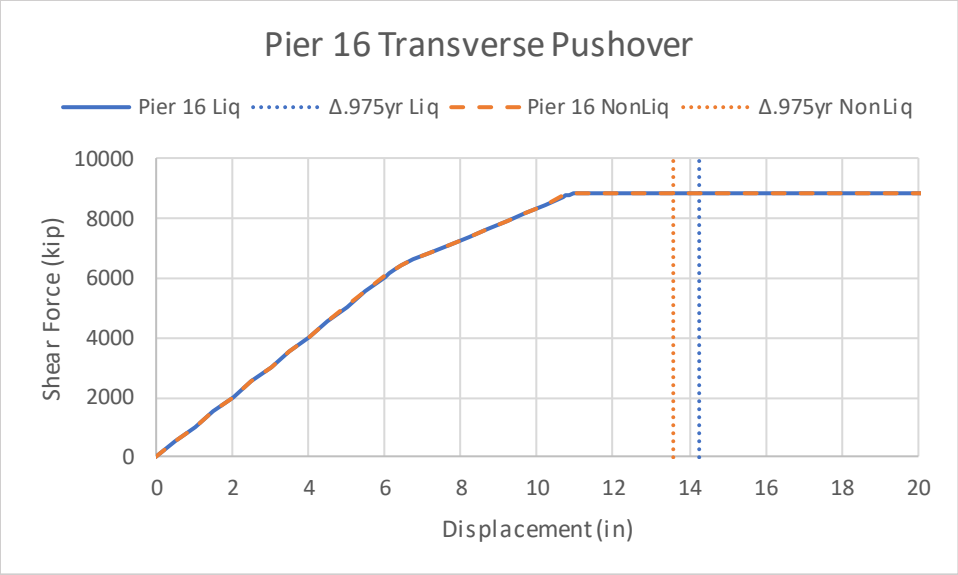
Seismic Critical Member	Displacement Ductility Demand Limits				
	Normal Bridges	Essential Bridges		Critical Bridges	
		SEE	FEE	SEE	FEE
Wall Type Pier in Weak Direction	5.0	2.5	1.5	1.5	1.0
Wall Type Pier in Strong Direction	1.0	1.0	1.0	1.0	1.0
Single Column Bent	5.0	2.5	1.5	1.5	1.0
Multiple Column Bent	6.0	3.5	2.0	1.5	1.0
Pile Column with Plastic Hinge at Top of Column	5.0	3.5	2.0	1.5	1.0
Pile Column with Plastic Hinge Below Ground	4.0	2.5	1.5	1.5	1.0
Superstructure	1.0	1.0	1.0	1.0	1.0

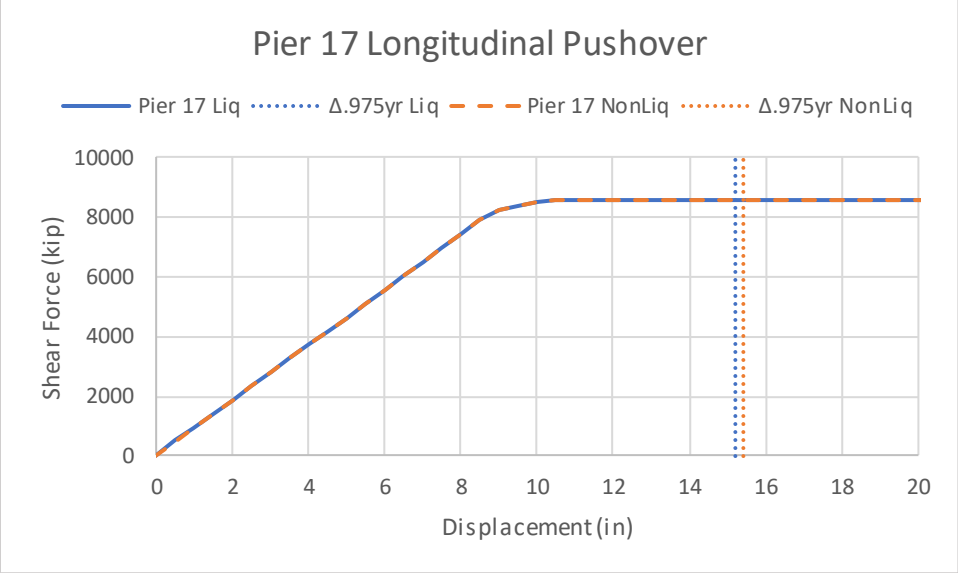
$$\mu_D = 1 + 3\left(\frac{\phi_{col}}{\phi_{yi}} - 1\right)\frac{L_p}{L}\left(1 - 0.5\frac{L_p}{L}\right) \qquad \text{(C4.9-6)}$$

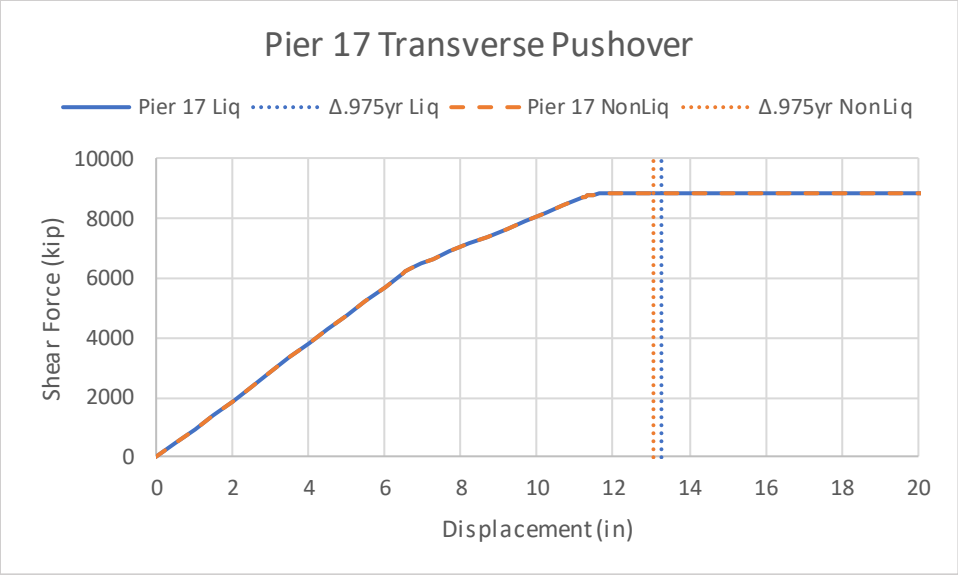
6.1 COMPARISON OF 975YR SEE DISPLACEMENT DEMANDS

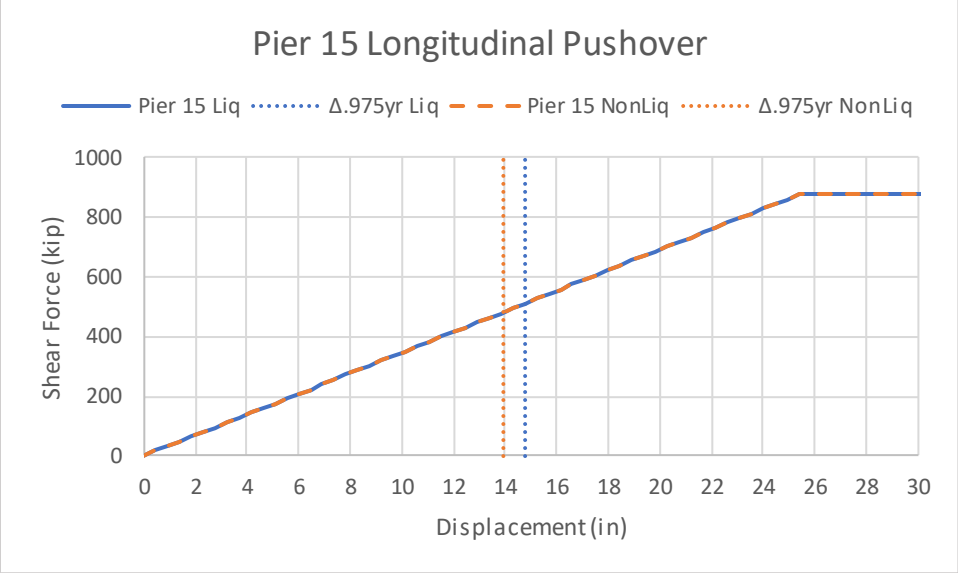
First, the nonlinear static pushover results are compared with the 975yr SEE displacement demands using both liquefied and non-liquefied soil properties to determine their impact on the overall behavior. Note that the results are truncated before reaching the displacement capacity of the piers due to the general low magnitude of the demands relative to the capacities. Note that the longitudinal pushover results for Piers 16 and 17 include only the seismic component of the column moments. Inclusion of the starting moments from the DC+DW+PS time-dependent analysis results that include creep and shrinkage will be evaluated subsequently when investigating the displacement at first hinge formation for comparison to the 210yr demands. These initial moments are no longer relevant once a plastic mechanism has formed, since forces are capped at the governing longitudinal displacement demands shown below.

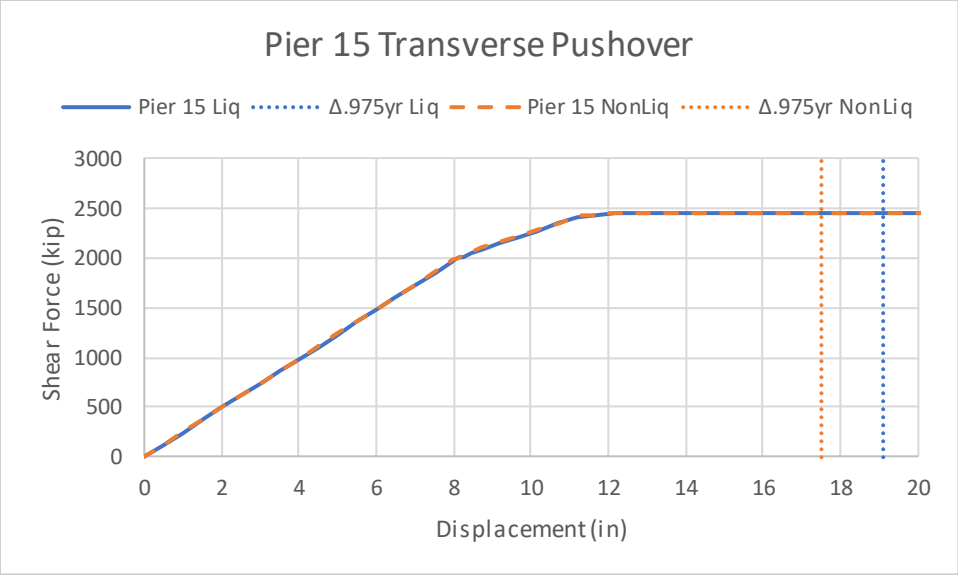


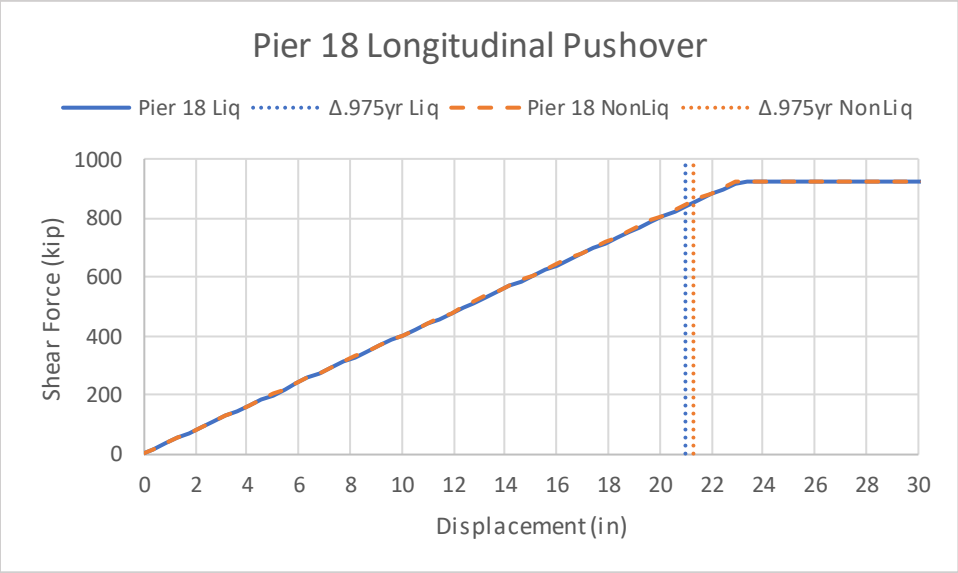

Pier 16 Transverse Pushover
Pier 16 Liq
Δ.975yr Liq
Pier 16 NonLiq
Δ.975yr NonLiq
Shear Force (kip)
10000
8000
6000
4000
2000
0
0
2
4
6
8
10
12
14
16
18
20
Displacement (in)

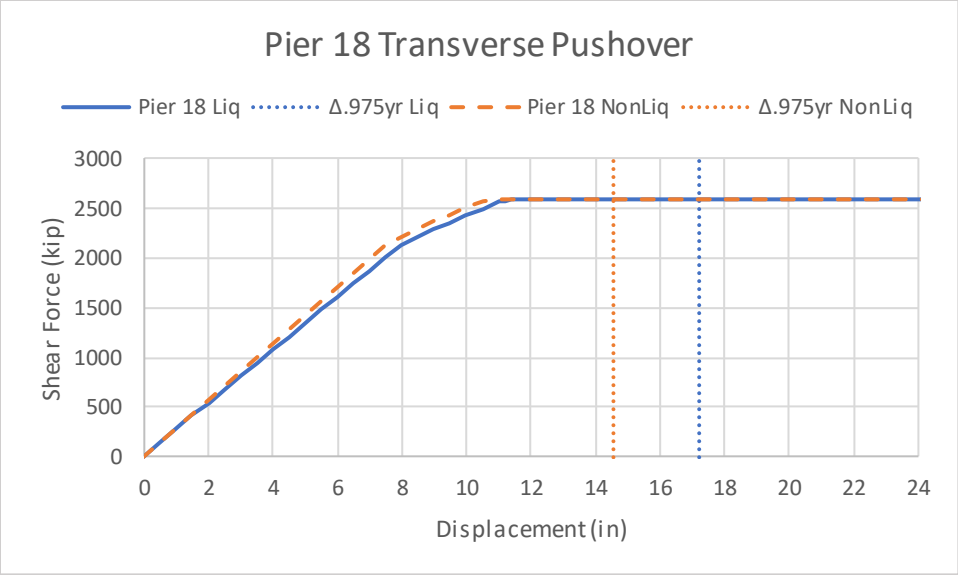

Pier 17 Longitudinal Pushover
Pier 17 Liq
Δ.975yr Liq
Pier 17 NonLiq
Δ.975yr NonLiq
Shear Force (kip)
10000
8000
6000
4000
2000
0
0
2
4
6
8
10
12
14
16
18
20
Displacement (in)


Pier 17 Transverse Pushover
Pier 17 Liq
Δ.975yr Liq
Pier 17 NonLiq
Δ.975yr NonLiq
Shear Force (kip)
10000
8000
6000
4000
2000
0
0
2
4
6
8
10
12
14
16
18
20
Displacement (in)


Pier 15 Longitudinal Pushover
Pier 15 Liq
Δ.975yr Liq
Pier 15 NonLiq
Δ.975yr NonLiq
Shear Force (kip)
1000
800
600
400
200
0
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Displacement (in)


Pier 15 Transverse Pushover
Pier 15 Liq
Δ.975yr Liq
Pier 15 NonLiq
Δ.975yr NonLiq
Shear Force (kip)
3000
2500
2000
1500
1000
500
0
0
2
4
6
8
10
12
14
16
18
20
Displacement (in)


Pier 18 Longitudinal Pushover
Pier 18 Liq
Δ.975yr Liq
Pier 18 NonLiq
Δ.975yr NonLiq
Shear Force (kip)
1000
800
600
400
200
0
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Displacement (in)

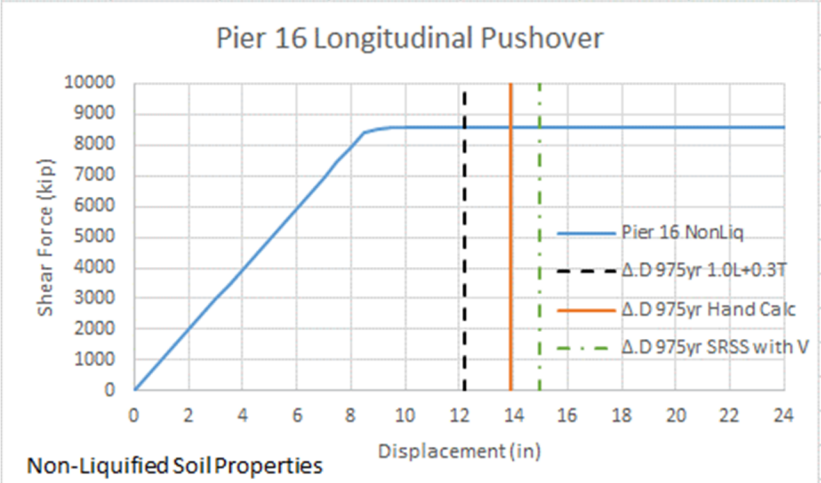


As shown the displacement ductility demands are low and little difference exists in both the pushover results and displacement demands based on the change from liquefied to non-liquefied soil properties following the Geotechnical recommendations.

6.2 HAND-CALCULATION CHECKS ON RSA AND NLSPO RESULTS

The following page provides an overview of hand-calculation checks performed to verify the results from the RSA demand and NLSPO capacity analyses. Overall, the results are in close agreement, as expected. Note that the hand-calculations for Pier 17 with liquified soil properties indicate that 2.1 inches of the total 9.9 inch displacement at yielding are attributable to foundation deformations associated with the soil structure interaction behavior. This is mainly due to the rotation at the CG of the foundation being extrapolated over the height of the tall columns to the superstructure level. Although there is little difference between liquefied and non-liquefied results, inclusion of the behavior is important. The following table includes an evaluation for the AASHTO SGS P-delta limit for Piers 16 and 17. The P-delta requirements were satisfied under the displacement demands.

Hand-Calc Check on Longitudinal RSA Displacements				
Gravitational Constant, g	32.174	ft/s^2		
EIeff at P.dl for M.long	1069585070	$kip\text{-}ft^2$		
Column Height	125	ft		
Keff.col = $12EIeff/L^3$	6572	kip/ft		
Keff.total = 4Keff.col	26286	kip/ft		
P15 Bearing DL Reaction	6407	kip		
P18 Bearing DL Reaction	6764	kip		
P16 DL Reaction with half col	38120	kip		
P17 DL Reaction with half col	38120	kip		
Tributary Longitudinal Weight	89411	kip		
Teff=$2\pi/\sqrt{(Keffg/W)}$	2.04	seconds, without SSI effects		
Sa Spectral Oridnate at Teff	0.340	g		
Δ.demand=SaW/Keff	13.88	in		
Due to spectral shape, demand decreases with SSI period elongation.				
Comparison to RSA, which has Foundation Stiffness				
Note that this is a Tension Model Comparison.				
Compression model long. displacements govern.				
T.long from Tension Model	2.85	seconds	1.0L+0.3T analysis	
Δ.demand Tension Model RSA	12.19	in	1.0L+0.3T analysis	
good comparison to hand calculation				
Δ.demand Comp Model RSA	14.99	in, avg	from SRSS with V	
Sanity Check on Pushover Results				
ϕy.long at P.dl	0.00024968	1/ft		
Column Height	125	ft		
Δy=$\phi y L^2/6$	7.80	in		
θ.foundation at Yield	8.78E-04	rad		
Δ.foundation Lateral at Yield	0.36	in		
Δ.foundation total	1.76	in		
Δ.yield with foundation	9.56	in		
good comparison to pushover results,				
indicates generally low ductility demands				



Combined P16 and P17 Longituidnal Pushover Check on P-delta			
Reduce the bottom moment by the magnitude of P-delta and calculate the remaining lateral shear capacity.			
The displacement corresponding to 25% loss in shear capacity is of interest for establishign the P-delta limit.			
Column Height	125	ft	column is in double bending
P16 Original Shear	4239	kip	using the values of Mp under the covnerged axial forces above
P17 Original Shear	4306	kip	
P16 Long. Disp	84	in	iterate until shear capacity drops 25%
P17 Long. Disp	84	in	
P16 M.bot Remain	131516	kip-ft	reduced by P-delta under the displacement shown above
P17 M.bot Remain	135717	kip-ft	
P16 Shear Capacity	3172	kip	
P17 Shear Capacity	3239	kip	
P16 Capacity Ratio	0.75		mathcing 25% strength loss at limiting P-delta displacement
P17 Capacity Ratio	0.75		mathcing 25% strength loss at limiting P-delta displacement
The dispalcement demands are well below those cooresponding to the SGS P-delta limit.			

6.3 DETAILED RESULTS AND COMPARISON TO DUCTILITY LIMITS

This section provides a detailed summary of the curvature demands in the column plastic hinges used to compute the displacement ductility demands for comparison to the WSDOT BDM limits for an Essential or Critical structure. Displacement ductility demands were computed using AASHTO SGS Equation C4.9-6, shown below. Results are first presented for the 975yr SEE and 210 FEE evaluation with non-liquefied soil properties, followed by an evaluation for the 975yr SEE with liquefied soil properties.

$$\mu_D = 1 + 3\left(\frac{\phi_{col}}{\phi_{yi}} - 1\right)\frac{L_p}{L}\left(1 - 0.5\frac{L_p}{L}\right) \qquad \text{(C4.9-6)}$$

where:

ϕ_{col} = column curvature at maximum displacement demand (computed from push over analysis) (1/in.)

ϕ_{yi} = idealized yield curvature determined in accordance with Article 8.5 (1/in.)

ϕ_{pd} = column plastic curvature demand (1/in.)

L_p = analytical plastic hinge length determined in accordance with Article 4.11.6 (in.)

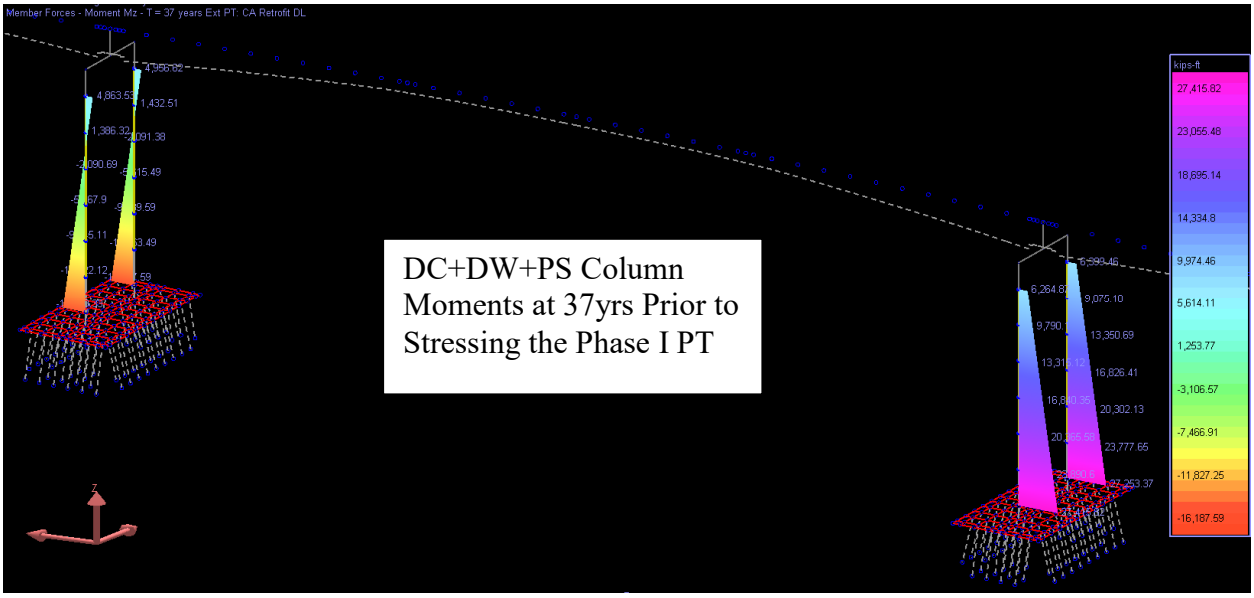
L = length of column from point of maximum moment to the point of moment contraflexure (in.)

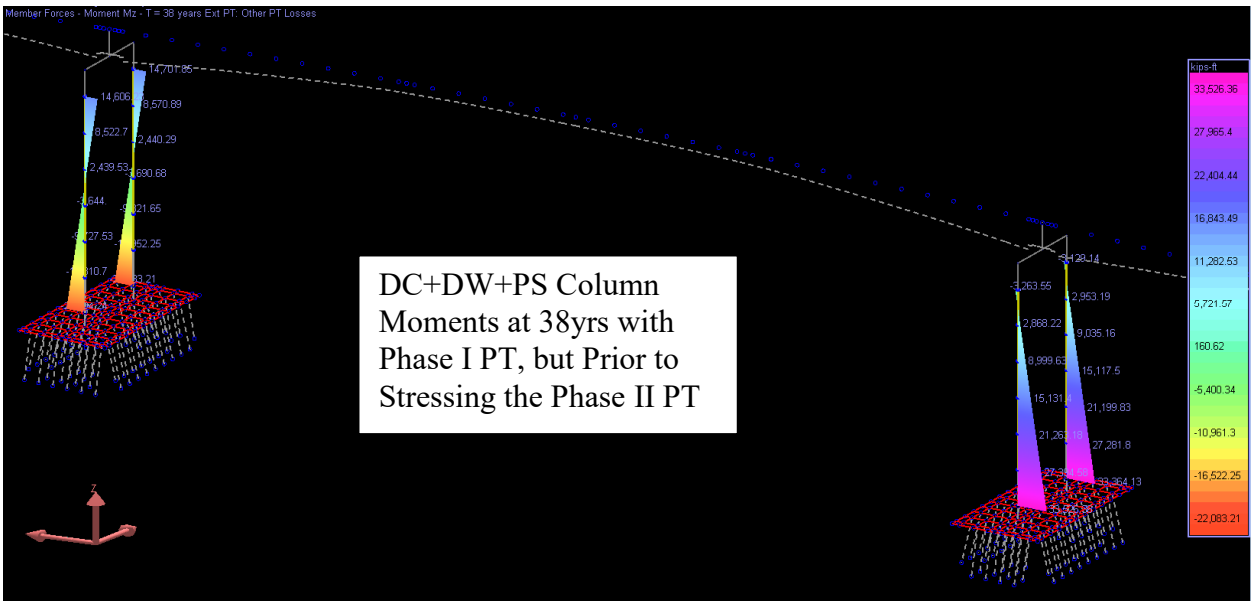
The method described by Eqs. C4.9-1 through C4.9-6, and Figures C4.9-1 and C4.9-2 is based upon the moment-area method for determining a member's rotation and deflection.

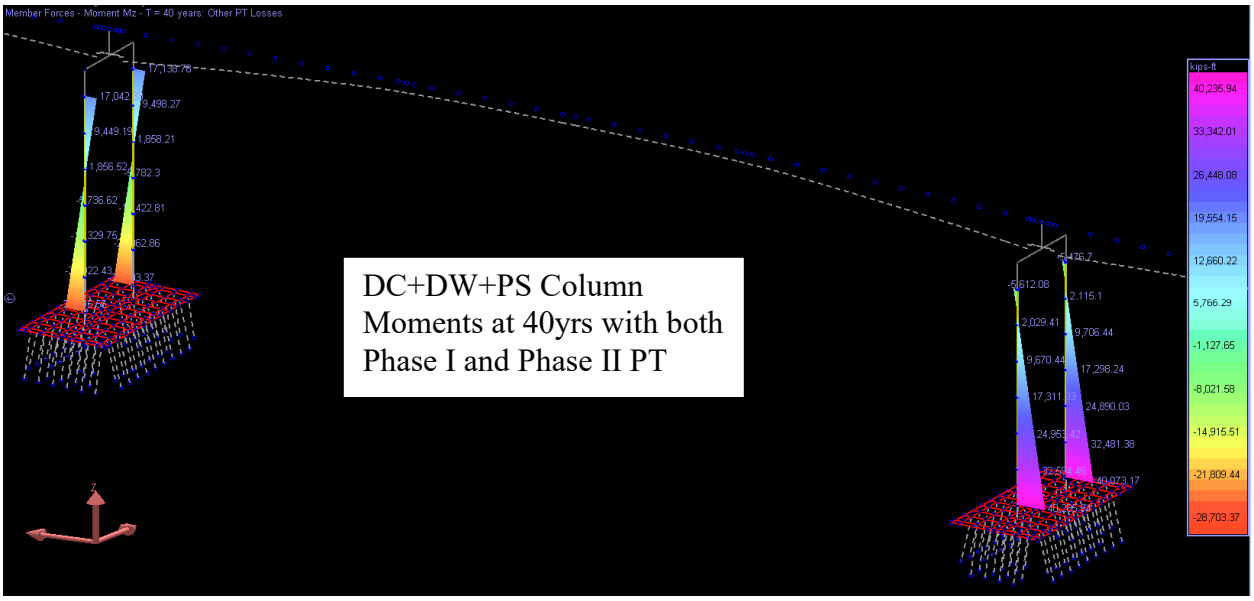
6.3.1 Impact of DC+DW+PS Column Moments on First Hinge Formation

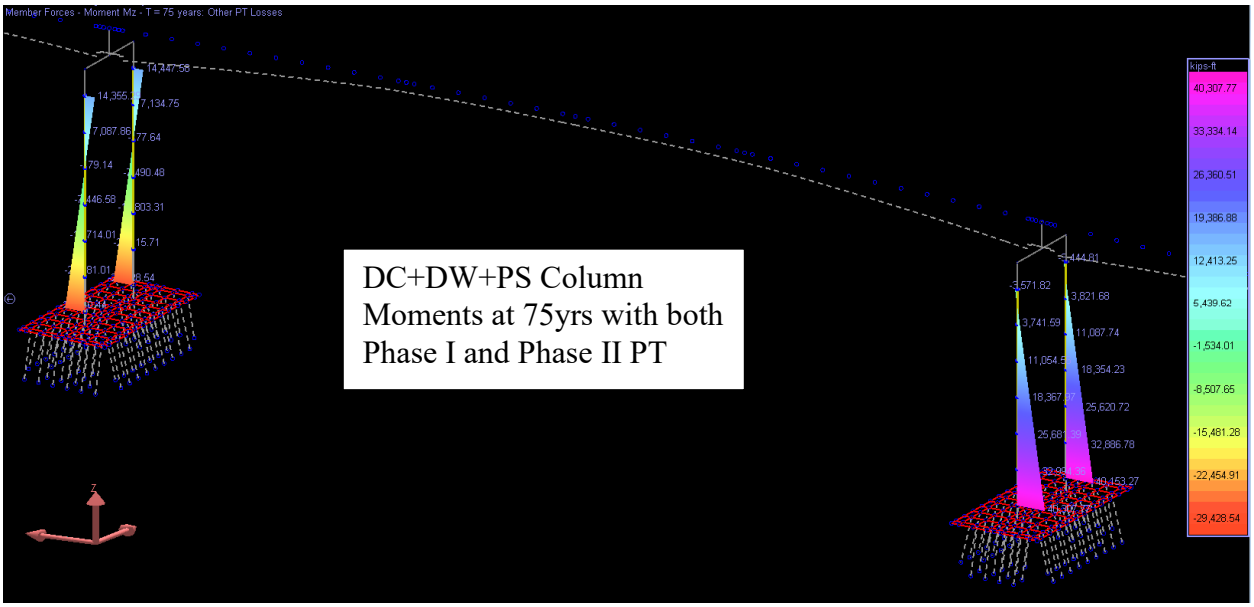
First, it is important to determine the impact of existing column moments on the displacement at initial hinge formation. These initial moments are no longer relevant once a plastic mechanism has formed, since forces are capped. The initial moments are important for evaluation of the potential to hinge in the 210yr FEE. Remember that this analysis was only performed using non-liquefied soil properties.

The following existing moment profiles were considered at various stages of the retrofit process to bound the potential effect of the existing condition and both the Phase I and II retrofit PT including time-dependent creep and shrinkage effects. The asymmetry in the Pier 16 and 17 column moments is due to locked-in forces associated with the staged construction process.









The following table summarizes the initial end moments, which under one direction of longitudinal response will always be additive to the seismic effects. These initial end moments were included in the evaluation to determine the longitudinal displacement at first hinge formation in Piers 16 and 17. Again, note that these effects have little impact on the response once a plastic mechanism has formed, since the forces in the system are capped.

Pier 16 Unsigned Column End Moments			Pier 17 Unsigned Column End Moments		
Stage	**Mbot (kip-ft)**	**Mtop (kip-ft)**	**Stage**	**Mtop (kip-ft)**	**Mbot (kip-ft)**
37yr Existing	27416	6265	37yr Existing	15999	4864
38yr Phase I	33526	3264	38yr Phase I	21894	14606
40yr Phase II	40236	5612	40yr Phase II	28516	17042
75yr Phase II	40308	3572	75yr Phase II	29248	14355

6.3.2 975yr SEE and 210yr FEE Ductility Demands with Non-Liquefied Soil

Pier 16 Longitudinal Pushover with Non-Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 16 Hinge for the North Column

Axial	19060	kip	average of top/bot at mechanism
Mp	267054	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.00025	1/ft	idealized yield curvature
ϕu	0.004129	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.00388	1/ft	plastic curvature at ultimate
θp	0.024047	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Pier 16 Hinge for the South Column

Axial	19060	kip	average of top/bot at mechanism
Mp	267054	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.00025	1/ft	idealized yield curvature
ϕu	0.004129	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.00388	1/ft	plastic curvature at ultimate
θp	0.024047	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA	14.99	in
Δ.demand 210yr FEE RSA	5.17	in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 16 North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
14.99	0.003925	0.004558	0.000633	0.000735	0.000883	0.000985	1.72	1.83	5.39	5.39

Pier 16 South Column at (Δ.demand 975yr SEE RSA)

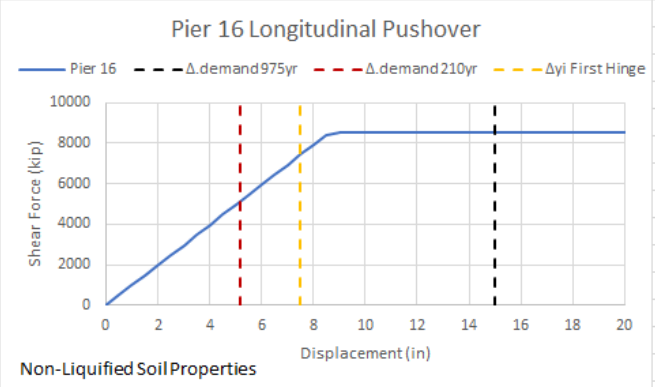
									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
14.99	0.003927	0.004559	0.000633	0.000736	0.000883	0.000985	1.72	1.83	5.39	5.39

Pier 16 Longitudinal Pushover Maximum Displacement Ductility Demand

μD.max	1.83			
			Pass/Fail	μD Ratio
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?			YES	0.52
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?			NO	1.22

Δyi.long at First Hinge (Mp)	7.48	in	when including the existing longitudinal column moments

In the 210yr FEE, Pier 16 Remains Elastic Under Longitudinal Response, Satisfying both Essential and Critical.



Non-Liquified Soil Properties

Calculate the Displacement at First Hinging (Exceedance of Mp) Including the Existing Column Longitudinal Moments											
		Displacement at Exceedance of Mp				267054	kip-ft	Represents First Hinge Formation			
		Pier 16 North Column (South Column Result Symmetric)									
		Bottom Moment (kip-ft)					Top Moment (kip-ft)				
	(in)	Only	at 37yr	at 38yr	at 40yr	at 75yr	Only	at 37yr	at 38yr	at 40yr	at 75yr
Step	Δ.long	Pushover	Existing	Phase I	Phase II	Phase II	Pushover	Existing	Phase I	Phase II	Phase II
1	0.50	15132	42548	48658	55368	55440	15748	22013	19012	21360	19320
2	1.00	30293	57709	63819	70529	70601	31527	37792	34791	37139	35099
3	1.50	45454	72870	78980	85690	85762	47306	53571	50570	52918	50878
4	2.00	60615	88031	94141	100851	100923	63085	69350	66349	68697	66657
5	2.50	75776	103192	109302	116012	116084	78864	85129	82128	84476	82436
6	3.00	90937	118353	124463	131173	131245	94643	100908	97907	100255	98215
7	3.50	106098	133514	139624	146334	146406	110422	116687	113686	116034	113994
8	4.00	121259	148675	154785	161495	161567	126201	132466	129465	131813	129773
9	4.50	136420	163836	169946	176656	176727	141980	148245	145244	147592	145552
10	5.00	151581	178996	185107	191817	191888	157759	164024	161023	163371	161331
11	5.50	166742	194157	200268	206977	207049	173538	179803	176802	179150	177110
12	6.00	181902	209318	215429	222138	222210	189317	195582	192581	194929	192889
13	6.50	197063	224479	230590	237299	237371	205096	211361	208359	210708	208668
14	7.00	212224	239640	245751	252460	252532	220875	227140	224138	226487	224447
15	7.50	227385	254801	260911	267621	267693	236654	242919	239917	242266	240225
16	8.00	242546	269962	276072			252433	258697	255696	258045	256004
17	8.50	257160					267002	273267	270266	272614	270574
18	9.00	265179									
19	9.50	267003									
		Pier 16 Bottom Moment (kip-ft)									
		at 37yr	at 38yr	at 40yr	at 75yr						
	(in)	Existing	Phase I	Phase II	Phase II						
	Δyi.long	7.90	7.70	7.48	7.48						
	Δ.demand 210yr FEE RSA			5.17	in						
	Δyi.long at First Hinge (Mp)			7.48	in	when including the existing longitudinal column moments					
	Pier 16 Remains Elastic Under Longitudinal Response, Satisfying both Essential and Critical in the FEE.										

Note that highlights simply show the pushover steps where Mp is first reached to interpolate the deformation at initial hinge formation for the governing case.

Pier 16 Transverse Pushover with Non-Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 16 Hinge for the Leading North Column

Axial	30045	kip	average of top/bot at mechanism
Mp	354190	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.000256	1/ft	idealized yield curvature
ϕu	0.004093	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.003837	1/ft	plastic curvature at ultimate
θp	0.023785	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Pier 16 Hinge for the Trailing South Column

Axial	8075	kip	average of top/bot at mechanism
Mp	199194	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.000232	1/ft	idealized yield curvature
ϕu	0.003821	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.003589	1/ft	plastic curvature at ultimate
θp	0.022247	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA	13.58	in
Δ.demand 210yr FEE RSA	5.24	in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 16 Leading North Column at (Δ.demand 975yr SEE RSA)

							Ductility Demand		Δu at εsuR or εcu Ductility Capacity	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft				
Δ.trans	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
13.58	0.002014	0.001559	0.000325	0.000251	0.000581	0.000507	1.36	1.28	5.25	5.25

Pier 16 Trailing South Column at (Δ.demand 975yr SEE RSA)

							Ductility Demand		Δu at εsuR or εcu Ductility Capacity	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft				
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
13.58	0.004888	0.0044	0.000789	0.00071	0.001021	0.000942	1.96	1.86	5.37	5.37

Pier 16 Transverse Pushover Maximum Displacement Ductility Demand

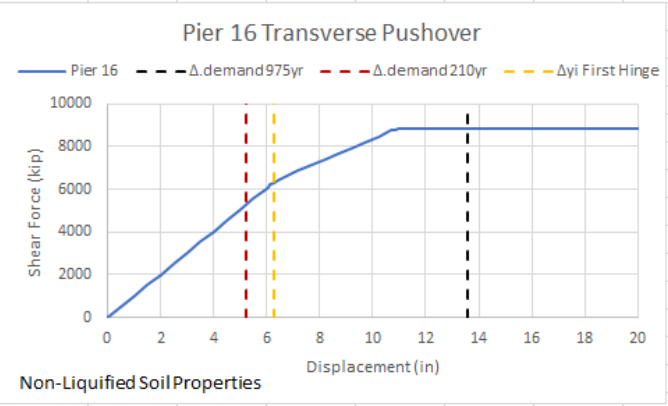
μD.max	1.96

	Pass/Fail	μD Ratio	
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?	YES	0.56	in the 975yr SEE
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?	NO	1.31	in the 975yr SEE

Δyi.trans at First Hinge (Mp)	6.26	in

trailing column hinges first due to reduced axial load

In the 210yr FEE, Pier 16 Remains Elastic Under Transverse Response, Satisfying both Essential and Critical.



Pier 17 Longitudinal Pushover with Non-Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 17 Hinge for the North Column

Axial	19060	kip	average of top/bot at mechanism
Mp	267054	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.00025	1/ft	idealized yield curvature
ϕu	0.004129	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.00388	1/ft	plastic curvature at ultimate
θp	0.024047	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Pier 17 Hinge for the South Column

Axial	19060	kip	average of top/bot at mechanism
Mp	267054	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.00025	1/ft	idealized yield curvature
ϕu	0.004129	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.00388	1/ft	plastic curvature at ultimate
θp	0.024047	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA	15.44	in
Δ.demand 210yr FEE RSA	5.27	in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 17 North Column at (Δ.demand 975yr SEE RSA)									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
15.44	0.003545	0.004735	0.000572	0.000764	0.000822	0.001014	1.65	1.87	5.39	5.39

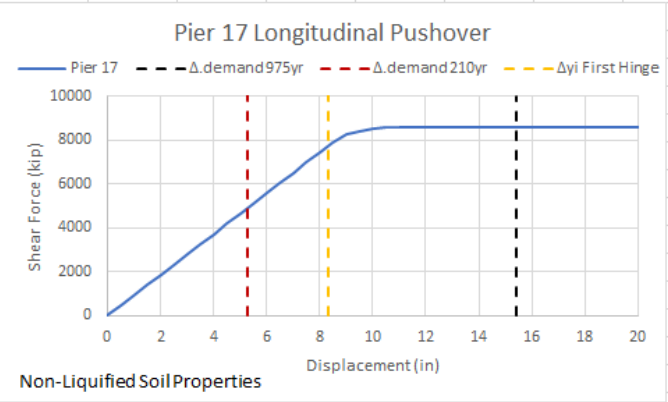
Pier 17 South Column at (Δ.demand 975yr SEE RSA)									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
15.44	0.003549	0.004739	0.000572	0.000765	0.000822	0.001014	1.65	1.87	5.39	5.39

Pier 17 Longitudinal Pushover Maximum Displacement Ductility Demand

μD.max	1.87			
			Pass/Fail	μD Ratio
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?			YES	0.53 in the 975yr SEE
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?			NO	1.24 in the 975yr SEE

Δyi.long at First Hinge (Mp)	8.31	in	when including the existing longitudinal column moments

In the 210yr FEE, Pier 17 Remains Elastic Under Longitudinal Response, Satisfying both Essential and Critical.



Calculate the Displacement at First Hinging (Exceedance of Mp) Including the Existing Column Longitudinal Moments

		Displacement at Exceedance of Mp				267054	kip-ft	Representing First Hinge Formation			
		Pier 17 North Column (South Column Result Symmetric)									
		Bottom Moment (kip-ft)					Top Moment (kip-ft)				
	(in)	Only	at 37yr	at 38yr	at 40yr	at 75yr	Only	at 37yr	at 38yr	at 40yr	at 75yr
Step	Δ.long	Pushover	Existing	Phase I	Phase II	Phase II	Pushover	Existing	Phase I	Phase II	Phase II
1	0.50	13883	29883	35778	42399	43132	14956	19820	29562	31999	29311
2	1.00	27844	43844	49739	56360	57093	29995	34858	44601	47037	44350
3	1.50	41806	57805	63700	70321	71054	45033	49896	59639	62075	59388
4	2.00	55767	71766	77661	84283	85016	60072	64935	74678	77114	74427
5	2.50	69729	85728	91623	98244	98977	75111	79974	89717	92153	89466
6	3.00	83690	99690	105584	112206	112939	90150	95013	104756	107192	104505
7	3.50	97652	113651	119546	126167	126900	105188	110052	119795	122231	119544
8	4.00	111613	127613	133507	140129	140862	120227	125091	134834	137270	134583
9	4.50	125575	141574	147469	154090	154823	135266	140130	149873	152309	149622
10	5.00	139536	155536	161430	168052	168785	150305	155169	164911	167348	164660
11	5.50	153498	169497	175392	182013	182746	165344	170208	179950	182386	179699
12	6.00	167459	183459	189354	195975	196708	180383	185247	194989	197425	194738
13	6.50	181421	197420	203315	209936	210669	195422	200286	210028	212465	209777
14	7.00	195382	211382	217277	223898	224631	210461	215325	225067	227504	224816
15	7.50	209344	225343	231238	237859	238592	225500	230364	240106	242543	239855
16	8.00	223305	239305	245200	251821	252554	240539	245403	255145	257581	254894
17	8.50	237267	253266	259161	265782	266515	255578	260442	270184	272620	269933
18	9.00	249710	265710	271605	278226	278959	267003	271866			
19	9.50	257336	273335								
20	10.00	264961									
21	10.50	267005									

		Pier 17 Top Moment (kip-ft)									
		at 37yr	at 38yr	at 40yr	at 75yr						
	(in)	Existing	Phase I	Phase II	Phase II						
	Δyi.long	8.79	8.39	8.31	8.40						
	Δ.demand 210yr RSA			5.27	in						
	Δyi.long at First Hinge (Mp)			8.31	in	when including the existing longitudinal column moments					
	Pier 17 Remains Elastic Under Longitudinal Response, Satisfying both Essential and Critical in the FEE.										

Pier 17 Transverse Pushover with Non-Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 17 Hinge for the Leading North Column

Axial	30045	kip	average of top/bot at mechanism
Mp	354190	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.000255571	1/ft	idealized yield curvature
ϕu	0.004092752	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.003837181	1/ft	plastic curvature at ultimate
θp	0.023784768	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Pier 17 Hinge for the Trailing South Column

Axial	8075	kip	average of top/bot at mechanism
Mp	199194	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.0002322	1/ft	idealized yield curvature
ϕu	0.003821233	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.003589033	1/ft	plastic curvature at ultimate
θp	0.022246623	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA	13.06	in
Δ.demand 210yr FEE RSA	4.82	in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 17 Leading North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.trans	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
13.06	0.001274996	0.000816	0.000206	0.000132	0.000461	0.000387	1.23	1.15	5.25	5.25

Pier 17 Trailing South Column at (Δ.demand 975yr SEE RSA)

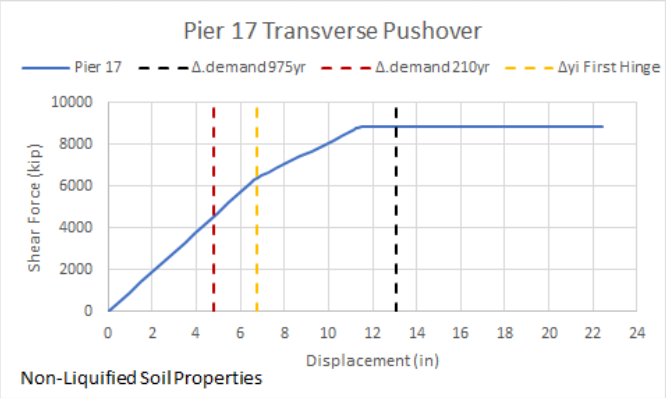
									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
13.06	0.004143467	0.003657	0.000668	0.00059	0.000901	0.000822	1.81	1.72	5.37	5.37

Pier 17 Transverse Pushover Maximum Displacement Ductility Demand

μD.max	1.81			
			Pass/Fail	μD Ratio
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?			YES	0.52 in the 975yr SEE
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?			NO	1.21 in the 975yr SEE

Δyi.trans at First Hinge (Mp)	6.76	in trailing column hinges first due to reduced axial load

In the 210yr FEE, Pier 16 Remains Elastic Under Transverse Response, Satisfying both Essential and Critical.



Pier 15 Longitudinal Pushover with Non-Liquified Soil Properties

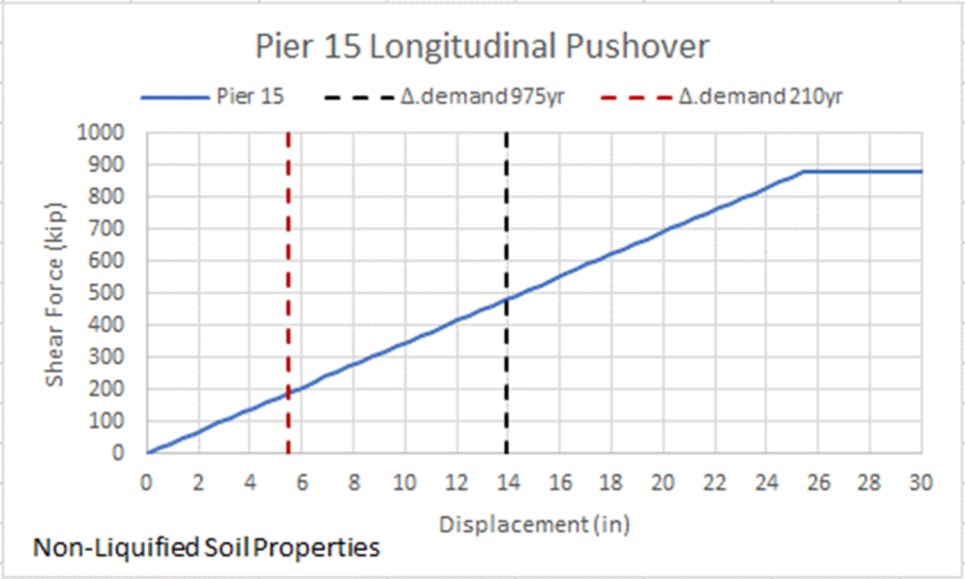
Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 15 Hinge under Longitudinal Response

Axial	4535	kip	average of top/bot at mechanism (note pier remains elastic)
Mp	51041	kip-ft	plastic moment capacity at mechanism
Lp	11.4	ft	plastic hinge length
ϕyi	0.000505	1/ft	idealized yield curvature
ϕu	0.008101	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.007596	1/ft	plastic curvature at ultimate
θp	0.086277	rad	plastic rotation at ultimate
Lc	127	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA	13.93	in
Δ.demand 210yr FEE RSA	5.44	in

Pier 15 Remains Elastic Under Longitudinal Response, Satisfying both Essential and Critical.



Pier 15 Transverse Pushover with Non-Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 15 Hinge for the Leading North Column

Axial	7135	kip	average of top/bot at mechanism
Mp	79458	kip-ft	plastic moment capacity at mechanism
Lp	5.6	ft	plastic hinge length
ϕyi	0.000384	1/ft	idealized yield curvature
ϕu	0.006331	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.005947	1/ft	plastic curvature at ultimate
θp	0.033295	rad	plastic rotation at ultimate
Lc	55	ft	length to point of contraflexure

Pier 15 Hinge for the Trailing South Column

Axial	1935	kip	average of top/bot at mechanism
Mp	55776	kip-ft	plastic moment capacity at mechanism
Lp	5.6	ft	plastic hinge length
ϕyi	0.000364	1/ft	idealized yield curvature
ϕu	0.006016	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.005653	1/ft	plastic curvature at ultimate
θp	0.031646	rad	plastic rotation at ultimate
Lc	55	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA	17.52	in
Δ.demand 210yr FEE RSA	6.85	in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 15 Leading North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.trans	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
17.52	0.004983	0.004146	0.00089	0.000741	0.001274	0.001124	1.67	1.56	5.49	5.49

Pier 15 Trailing South Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
17.52	0.007387	0.006526	0.001319	0.001166	0.001683	0.001529	2.05	1.93	5.50	5.50

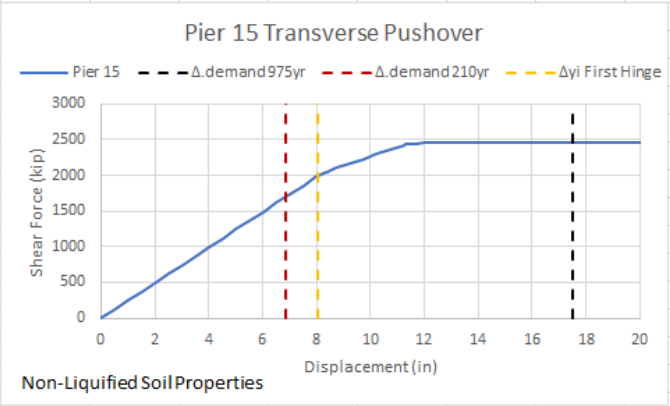
Pier 15 Transverse Pushover Maximum Displacement Ductility Demand

μD.max	2.05

	Pass/Fail	μD Ratio	
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?	YES	0.59	in the 975yr SEE
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?	NO	1.37	in the 975yr SEE

Δyi.trans at First Hinge (Mp)	8.06	in	trailing column hinges first due to reduced axial load

In the 210yr FEE, Pier 15 Remains Elastic Under Transverse Response, Satisfying both Essential and Critical.



Pier 18 Longitudinal Pushover with Non-Liquified Soil Properties

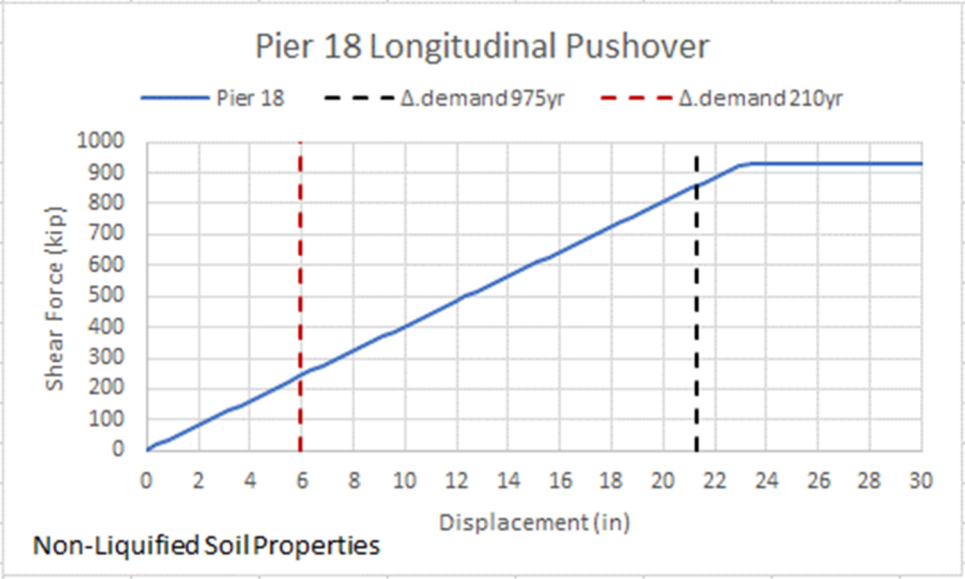
Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 18 Hinge under Longitudinal Response

Axial	4535	kip	average of top/bot at mechanism (note pier remains elastic)
Mp	51041	kip-ft	plastic moment capacity at mechanism
Lp	10.9	ft	plastic hinge length
ϕyi	0.000505	1/ft	idealized yield curvature
ϕu	0.008101	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.007596	1/ft	plastic curvature at ultimate
θp	0.082631	rad	plastic rotation at ultimate
Lc	121	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA	21.33	in
Δ.demand 210yr FEE RSA	5.94	in

Pier 18 Remains Elastic Under Longitudinal Response, Satisfying both Essential and Critical.



Non-Liquified Soil Properties

Pier 18 Transverse Pushover with Non-Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 18 Hinge for the Leading North Column

Axial	7135	kip	average of top/bot at mechanism
Mp	79458	kip-ft	plastic moment capacity at mechanism
Lp	5.4	ft	plastic hinge length
ϕyi	0.000384	1/ft	idealized yield curvature
ϕu	0.006331	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.005947	1/ft	plastic curvature at ultimate
θp	0.031868	rad	plastic rotation at ultimate
Lc	52	ft	length to point of contraflexure

Pier 18 Hinge for the Trailing South Column

Axial	1935	kip	average of top/bot at mechanism
Mp	55776	kip-ft	plastic moment capacity at mechanism
Lp	5.4	ft	plastic hinge length
ϕyi	0.000364	1/ft	idealized yield curvature
ϕu	0.006016	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.005653	1/ft	plastic curvature at ultimate
θp	0.030289	rad	plastic rotation at ultimate
Lc	52	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA	14.57	in
Δ.demand 210yr FEE RSA	5.13	in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 15 Leading North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.trans	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
14.57	0.00358	0.002868	0.000668	0.000535	0.001052	0.000919	1.51	1.41	5.55	5.55

Pier 15 Trailing South Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
14.57	0.005914	0.005121	0.001104	0.000956	0.001467	0.001319	1.89	1.77	5.56	5.56

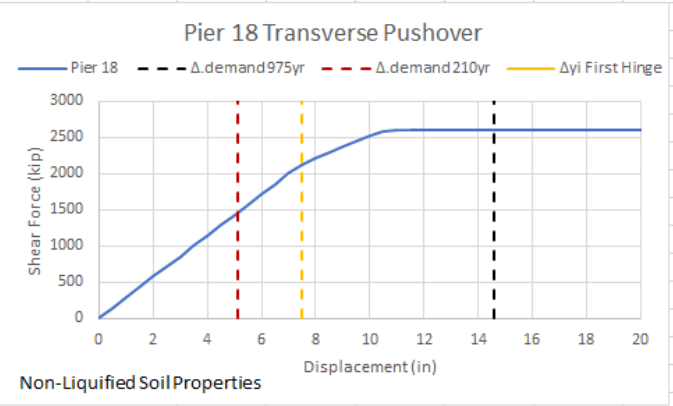
Pier 15 Transverse Pushover Maximum Displacement Ductility Demand

μD.max	1.89

	Pass/Fail	μD Ratio	
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?	YES	0.54	in the 975yr EQ
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?	NO	1.26	in the 975yr EQ

Δyi.trans at First Hinge (Mp)	7.49	in	trailing column hinges first due to reduced axial load

In the 210yr FEE, Pier 18 Remains Elastic Under Transverse Response, Satisfying both Essential and Critical.



6.3.3 975yr SEE Ductility Demands with Liquefied Soil Properties

Pier 16 Longitudinal Pushover with Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 16 Hinge for the North Column

Axial	19060	kip	average of top/bot at mechanism
Mp	267054	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.00025	1/ft	idealized yield curvature
ϕu	0.004129	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.00388	1/ft	plastic curvature at ultimate
θp	0.024047	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Pier 16 Hinge for the South Column

Axial	19060	kip	average of top/bot at mechanism
Mp	267054	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.00025	1/ft	idealized yield curvature
ϕu	0.004129	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.00388	1/ft	plastic curvature at ultimate
θp	0.024047	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA 14.94 in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 16 North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
14.94	0.003891	0.004515	0.000628	0.000728	0.000877	0.000978	1.71	1.82	5.39	5.39

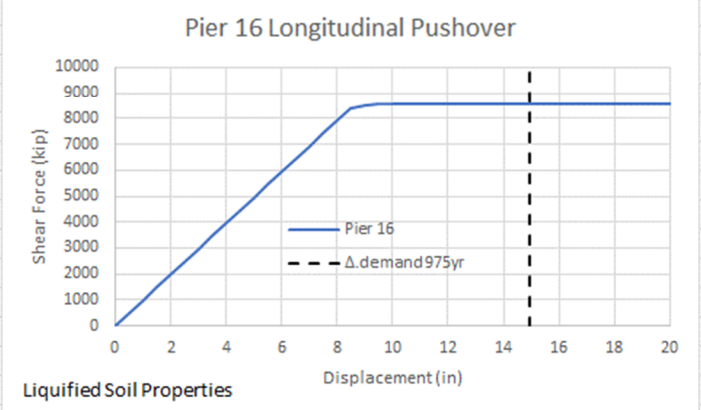
Pier 16 South Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
14.94	0.003892	0.004516	0.000628	0.000729	0.000878	0.000978	1.71	1.83	5.39	5.39

Pier 16 Longitudinal Pushover Maximum Displacement Ductility Demand

μD.max 1.83

	Pass/Fail	μD Ratio	
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?	YES	0.52	in the 975yr EQ
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?	NO	1.22	in the 975yr EQ



Pier 16 Transverse Pushover with Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 16 Hinge for the Leading North Column

Axial	30045	kip	average of top/bot at mechanism
Mp	354190	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.000256	1/ft	idealized yield curvature
ϕu	0.004093	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.003837	1/ft	plastic curvature at ultimate
θp	0.023785	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Pier 16 Hinge for the Trailing South Column

Axial	8075	kip	average of top/bot at mechanism
Mp	199194	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.000232	1/ft	idealized yield curvature
ϕu	0.003821	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.003589	1/ft	plastic curvature at ultimate
θp	0.022247	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA 14.28 in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 16 Leading North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.trans	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
14.28	0.002474	0.002018	0.000399	0.000326	0.000655	0.000581	1.44	1.36	5.25	5.25

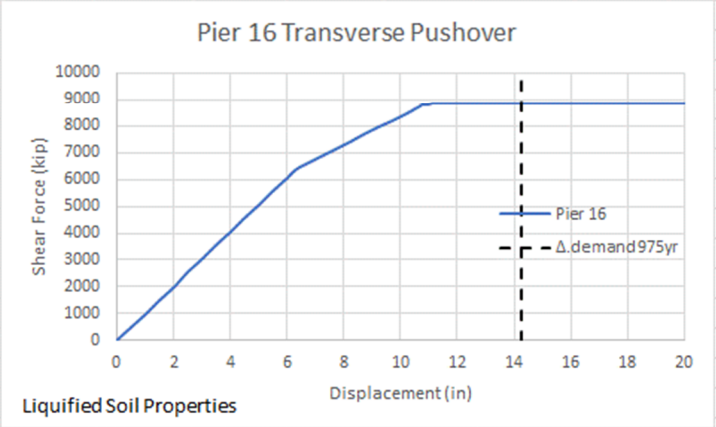
Pier 16 Trailing South Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
14.28	0.005348	0.00486	0.000863	0.000784	0.001095	0.001016	2.05	1.95	5.37	5.37

Pier 16 Transverse Pushover Maximum Displacement Ductility Demand

μD.max 2.05

	Pass/Fail	μD Ratio	
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?	YES	0.59	in the 975yr EQ
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?	NO	1.37	in the 975yr EQ



Pier 17 Longitudinal Pushover with Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 17 Hinge for the North Column

Axial	19060	kip	average of top/bot at mechanism
Mp	267054	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.00025	1/ft	idealized yield curvature
ϕu	0.004129	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.00388	1/ft	plastic curvature at ultimate
θp	0.024047	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Pier 17 Hinge for the South Column

Axial	19060	kip	average of top/bot at mechanism
Mp	267054	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.00025	1/ft	idealized yield curvature
ϕu	0.004129	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.00388	1/ft	plastic curvature at ultimate
θp	0.024047	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA 15.21 in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 17 North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
15.21	0.003392	0.004582	0.000547	0.000739	0.000797	0.000989	1.62	1.84	5.39	5.39

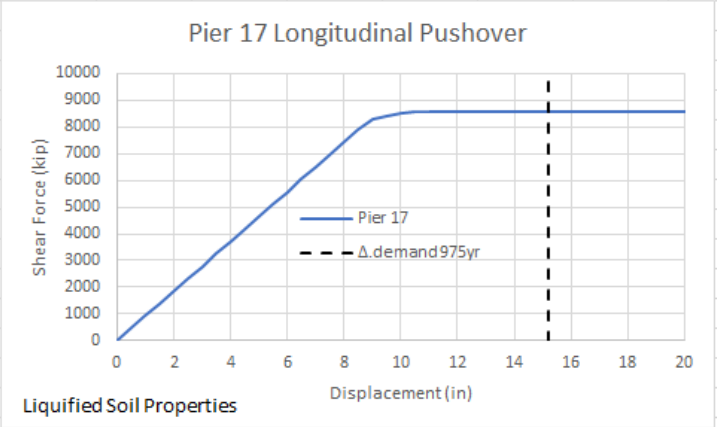
Pier 17 South Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
15.21	0.003395	0.004585	0.000548	0.00074	0.000797	0.000989	1.62	1.84	5.39	5.39

Pier 17 Longitudinal Pushover Maximum Displacement Ductility Demand

μD.max 1.84

	Pass/Fail	μD Ratio	
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?	YES	0.53	in the 975yr EQ
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?	NO	1.23	in the 975yr EQ



Pier 17 Transverse Pushover with Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 17 Hinge for the Leading North Column

Axial	30045	kip	average of top/bot at mechanism
Mp	354190	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.000256	1/ft	idealized yield curvature
ϕu	0.004093	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.003837	1/ft	plastic curvature at ultimate
θp	0.023785	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Pier 17 Hinge for the Trailing South Column

Axial	8075	kip	average of top/bot at mechanism
Mp	199194	kip-ft	plastic moment capacity at mechanism
Lp	6.2	ft	plastic hinge length
ϕyi	0.000232	1/ft	idealized yield curvature
ϕu	0.003821	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.003589	1/ft	plastic curvature at ultimate
θp	0.022247	rad	plastic rotation at ultimate
Lc	62.5	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA 13.25 in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 17 Leading North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.trans	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
13.25	0.001402	0.000943	0.000226	0.000152	0.000482	0.000408	1.25	1.17	5.25	5.25

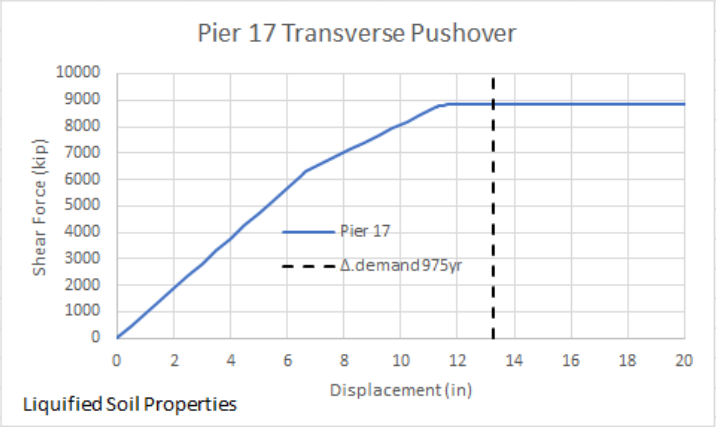
Pier 17 Trailing South Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
13.25	0.00427	0.003784	0.000689	0.00061	0.000921	0.000843	1.84	1.74	5.37	5.37

Pier 17 Transverse Pushover Maximum Displacement Ductility Demand

μD.max 1.84

	Pass/Fail	μD Ratio	
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?	YES	0.53	in the 975yr EQ
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?	NO	1.23	in the 975yr EQ



Pier 15 Longitudinal Pushover with Liquified Soil Properties

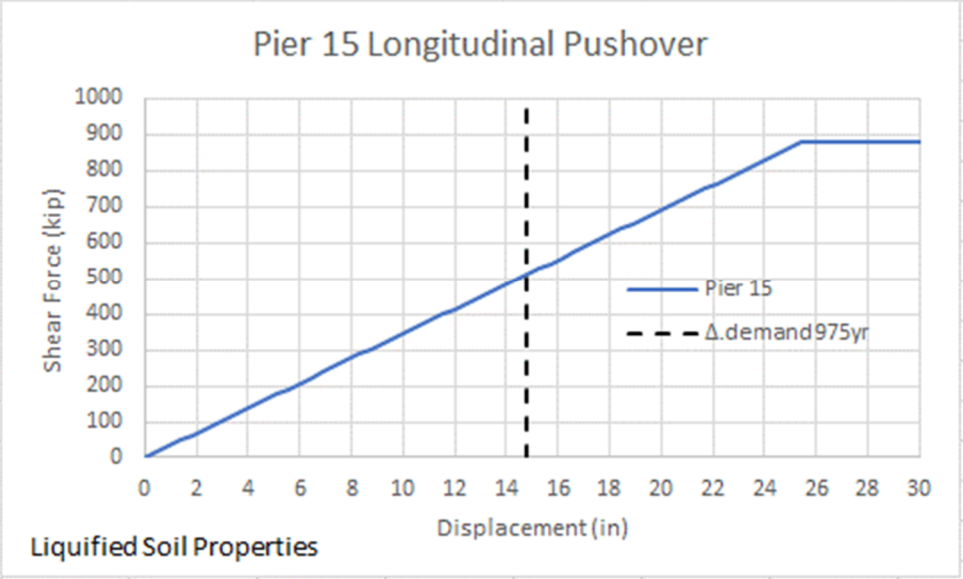
Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 15 Hinge under Longitudinal Response

Axial	4535	kip	average of top/bot at mechanism (note pier remains elastic)
Mp	51041	kip-ft	plastic moment capacity at mechanism
Lp	11.4	ft	plastic hinge length
ϕyi	0.000505	1/ft	idealized yield curvature
ϕu	0.008101	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.007596	1/ft	plastic curvature at ultimate
θp	0.086277	rad	plastic rotation at ultimate
Lc	127	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA	14.8	in

Pier 15 Remains Elastic Under Longitudinal Response, Satisfying both Essential and Critical.



Liquified Soil Properties

Pier 15 Transverse Pushover with Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 15 Hinge for the Leading North Column

Axial	7135	kip	average of top/bot at mechanism
Mp	79458	kip-ft	plastic moment capacity at mechanism
Lp	5.6	ft	plastic hinge length
ϕyi	0.000384	1/ft	idealized yield curvature
ϕu	0.006331	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.005947	1/ft	plastic curvature at ultimate
θp	0.033295	rad	plastic rotation at ultimate
Lc	55	ft	length to point of contraflexure

Pier 15 Hinge for the Trailing South Column

Axial	1935	kip	average of top/bot at mechanism
Mp	55776	kip-ft	plastic moment capacity at mechanism
Lp	5.6	ft	plastic hinge length
ϕyi	0.000364	1/ft	idealized yield curvature
ϕu	0.006016	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.005653	1/ft	plastic curvature at ultimate
θp	0.031646	rad	plastic rotation at ultimate
Lc	55	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA 19.13 in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 15 Leading North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.trans	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
19.13	0.006151	0.005315	0.001099	0.000949	0.001482	0.001333	1.83	1.72	5.49	5.49

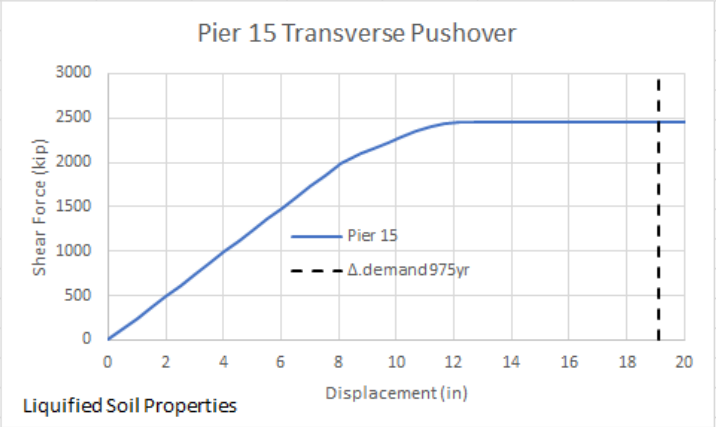
Pier 15 Trailing South Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
19.13	0.008556	0.007694	0.001528	0.001374	0.001892	0.001738	2.22	2.10	5.50	5.50

Pier 15 Transverse Pushover Maximum Displacement Ductility Demand

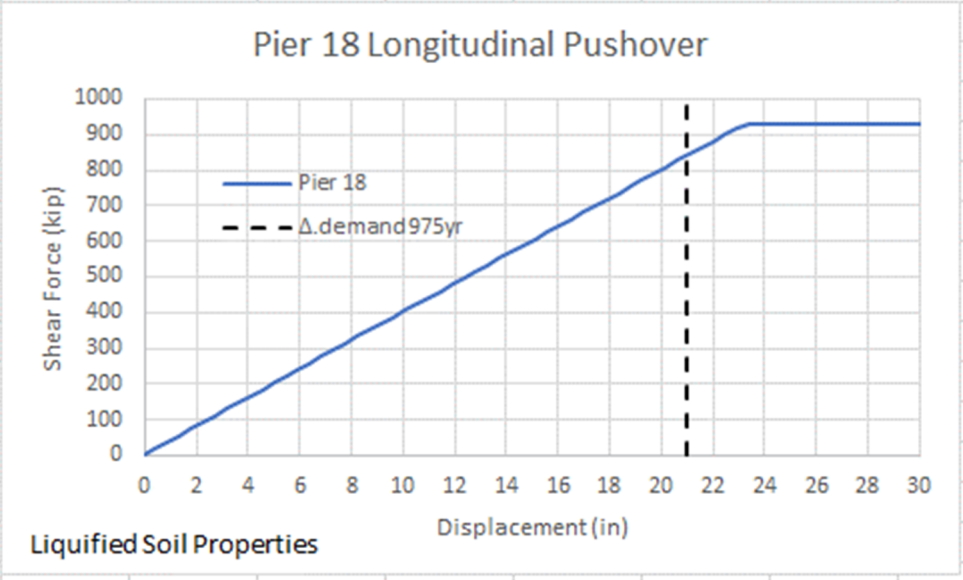
μD.max 2.22

	Pass/Fail	μD Ratio	
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?	YES	0.63	in the 975yr EQ
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?	NO	1.48	in the 975yr EQ



Pier 18 Hinge under Longitudinal Response			
Axial	4535	kip	average of top/bot at mechanism (note pier remains elastic)
Mp	51041	kip-ft	plastic moment capacity at mechanism
Lp	10.9	ft	plastic hinge length
ϕyi	0.000505	1/ft	idealized yield curvature
ϕu	0.008101	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.007596	1/ft	plastic curvature at ultimate
θp	0.082631	rad	plastic rotation at ultimate
Lc	121	ft	length to point of contraflexure
Δ.demand 975yr SEE RSA		20.99	in

Pier 18 Remains Elastic Under Longitudinal Response, Satisfying both Essential and Critical.



Liquified Soil Properties

Pier 18 Transverse Pushover with Liquified Soil Properties

Plastic Hinge Lengths and Limiting Plastic Curvature at Governing of εsuR and εcu

Pier 18 Hinge for the Leading North Column

Axial	7135	kip	average of top/bot at mechanism
Mp	79458	kip-ft	plastic moment capacity at mechanism
Lp	5.4	ft	plastic hinge length
ϕyi	0.000384	1/ft	idealized yield curvature
ϕu	0.006331	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.005947	1/ft	plastic curvature at ultimate
θp	0.031868	rad	plastic rotation at ultimate
Lc	52	ft	length to point of contraflexure

Pier 18 Hinge for the Trailing South Column

Axial	1935	kip	average of top/bot at mechanism
Mp	55776	kip-ft	plastic moment capacity at mechanism
Lp	5.4	ft	plastic hinge length
ϕyi	0.000364	1/ft	idealized yield curvature
ϕu	0.006016	1/ft	ultimate curvature at εsuR and εcu
ϕp	0.005653	1/ft	plastic curvature at ultimate
θp	0.030289	rad	plastic rotation at ultimate
Lc	52	ft	length to point of contraflexure

Δ.demand 975yr SEE RSA 17.25 in

Calculate the Dispalcement Ductility Demand in the Column Plastic Hinges using SGS Equation C4.9-6

Pier 15 Leading North Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.trans	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
17.25	0.005202	0.004491	0.000971	0.000838	0.001355	0.001222	1.74	1.64	5.55	5.55

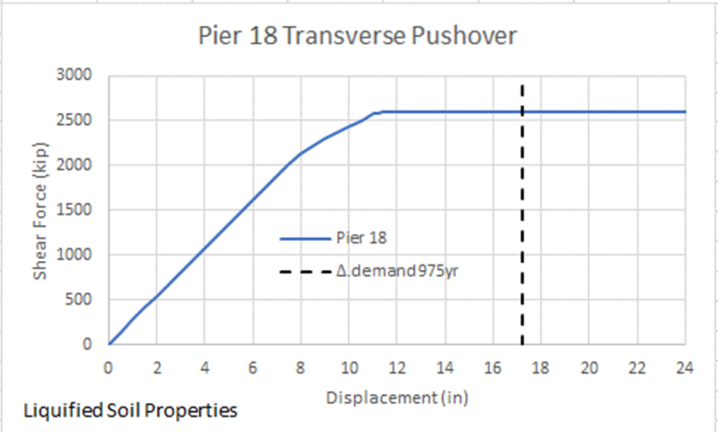
Pier 15 Trailing South Column at (Δ.demand 975yr SEE RSA)

									Δu at εsuR or εcu	
(in)	(rad)	(rad)	1/ft	1/ft	1/ft	1/ft	Ductility Demand		Ductility Capacity	
Δ.long	θp.bot	θp.top	ϕp.bot	ϕp.top	ϕcol.bot	ϕcol.top	μD.bot	μD.top	μΔ.bot	μΔ.top
17.25	0.007537	0.006744	0.001407	0.001259	0.00177	0.001622	2.13	2.01	5.56	5.56

Pier 15 Transverse Pushover Maximum Displacement Ductility Demand

μD.max 2.13

	Pass/Fail	μD Ratio	
Does the pier pass the WSDOT SEE μD = 3.5 limit for an Essential Bridge?	YES	0.61	in the 975yr EQ
Does the pier pass the WSDOT SEE μD = 1.5 limit for a Critical Bridge?	NO	1.42	in the 975yr EQ



6.4 DUCTILITY DEMAND SUMMARY

This section summarizes the displacement ductility demands from the site-specific ground motion evaluation and provides a comparison to WSDOT BDM limits for an Essential or Critical structure. As shown below, the structure meets the column displacement ductility requirements for classification as an Essential structure in the 975yr SEE. As shown in the following table, the structure remains elastic in the 210yr FEE, satisfying the WSDOT BDM displacement ductility requirements for both and Essential and Critical structure.

Therefore, based on the computed displacement ductility demands, the structure can be classified as Essential, so long as there does not exist a deficiency in capacity design that prevents formation of the intended column plastic hinging mechanism. This is the topic of the following investigation. Note that the superstructure does require flexural strengthening at the distressed Joints 14 and 38 to equilibrate the flexural overstrength demands, so the Phase II PT is needed to satisfy seismic requirements as well. The analysis uncovered vulnerabilities for the Pier 18 pile foundation, as will be discussed.

Site-Specific	975yr SEE, Liquefied		975yr SEE, Non-Liquefied	
Pier	μD.long	μD.trans	μD.long	μD.trans
15	Elastic	2.22	Elastic	2.05
16	1.83	2.05	1.83	1.96
17	1.84	1.84	1.87	1.81
18	Elastic	2.13	Elastic	1.89
Maximum Displacement Ductility Demand, μD				2.22
WSDOT BDM SEE μD Limit for an Essential Structure				3.5, PASS
WSDOT BDM SEE μD Limit for a Critical Structure				1.5, FAIL

Site-Specific	210yr FEE, Non-Liquefied					
	Longitudinal Response			Transverse Response		
Pier	ΔD.long (in)	Δyi First Hinge (in)	μD.long	ΔD.trans (in)	Δyi First Hinge (in)	μD.trans
15	5.44	25.85	Elastic	6.85	8.06	Elastic
16	5.17	7.48	Elastic	5.24	6.26	Elastic
17	5.27	8.31	Elastic	4.82	6.76	Elastic
18	5.94	23.38	Elastic	5.13	7.49	Elastic
Maximum Displacement Ductility Demand, μD				Elastic		
WSDOT BDM FEE μD Limit for an Essential Structure				1.5, PASS		
WSDOT BDM FEE μD Limit for a Critical Structure				1.0, PASS		

6.4.1 Comparison to Demands from the Preliminary Site Class D Spectra

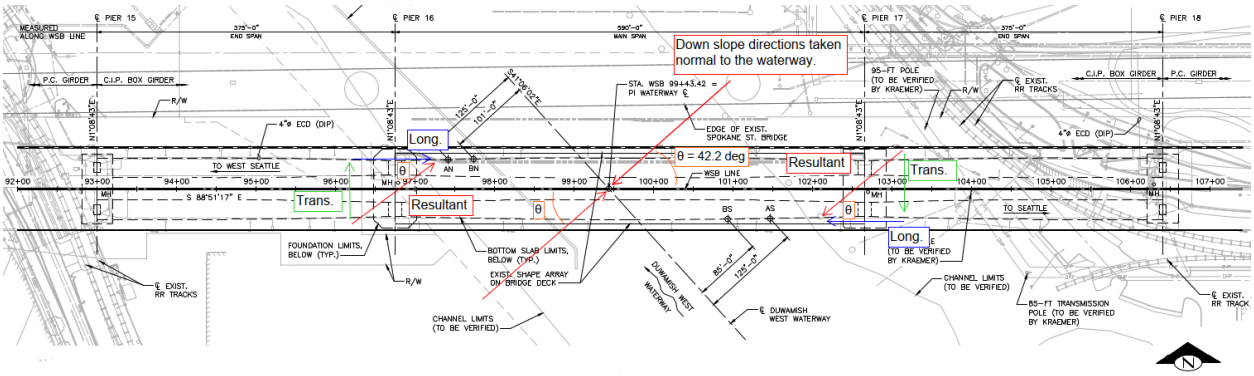
The following represents a comparison to the demands from the preliminary Site Class D Spectra that was superseded by the recommended site-specific ground motions. The same conclusions are obtained, as the columns satisfy the displacement ductility demand limits of an Essential structure.

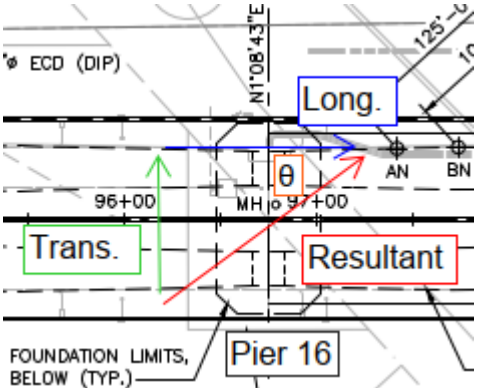
Site Class D	975yr SEE, Liquefied		975yr SEE, Non-Liquefied	
Pier	µD.long	µD.trans	µD.long	µD.trans
15	Elastic	1.91	Elastic	1.88
16	1.69	1.84	1.72	1.78
17	1.72	1.62	1.74	1.58
18	Elastic	1.65	Elastic	1.69
Maximum Displacement Ductility Demand, µD				1.91
WSDOT BDM SEE µD Limit for an Essential Structure				3.5, PASS
WSDOT BDM SEE µD Limit for a Critical Structure				1.5, FAIL

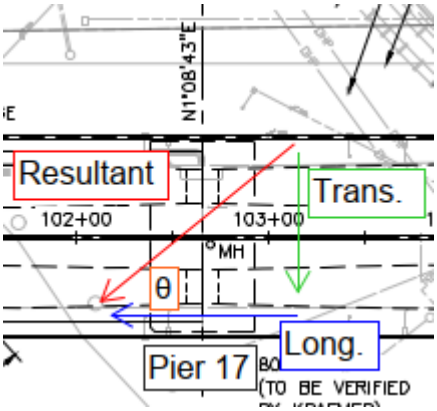
Site Class D	210yr FEE, Non-Liquefied					
	Longitudinal Response			Transverse Response		
Pier	ΔD.long (in)	Δyi First Hinge (in)	µD.long	ΔD.trans (in)	Δyi First Hinge (in)	µD.trans
15	5.91	25.85	Elastic	7.15	8.06	Elastic
16	6.37	7.48	Elastic	5.69	6.26	Elastic
17	6.47	8.31	Elastic	5.28	6.76	Elastic
18	8.19	23.38	Elastic	6.04	7.49	Elastic
Maximum Displacement Ductility Demand, µD				Elastic		
WSDOT BDM FEE µD Limit for an Essential Structure				1.5, PASS		
WSDOT BDM FEE µD Limit for a Critical Structure				1.0, PASS		

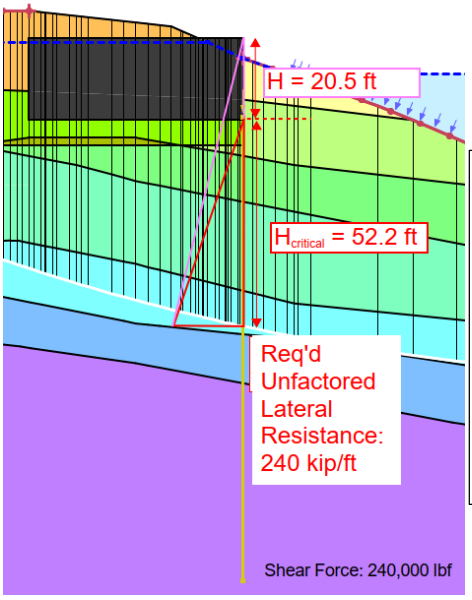
7.0 OVERSTRENGTH EVALUATION OF THE PILE FOUNDATIONS

This section provides an assessment of the combined flexural overstrength demands and recommended lateral soil forces for Piers 16 and 17. Lateral soil forces recommended by Shannon and Wilson are previous presented. These demands are restated here to aid in interpretation of the procedure and results. The foundations for Piers 15 and 18 were evaluated for flexural overstrength demands, since no lateral soil forces corresponding to slope instability effects were recommended in the draft Geotechnical report dated 10/5/2020.

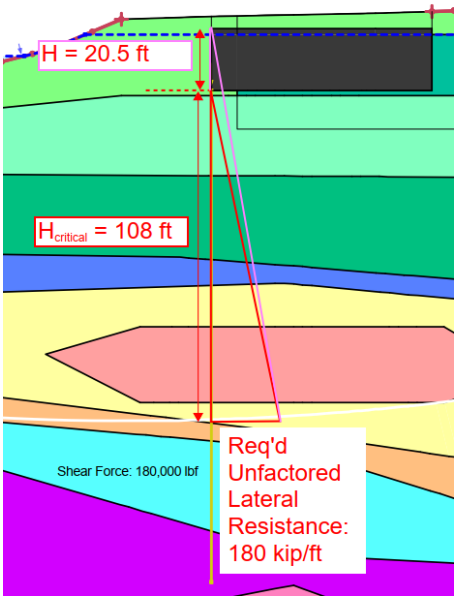








Geotech lateral soil forces to maintain the required factor of safety for slope stability.



The lateral soil forces provided by Shannon and Wilson represent pile loading required to maintain the code-based factor of safety for slope stability. The lateral soil forces were interpreted as resultant forces per foot width needed to restrain the soil from movement and were applied concurrently with the liquified soil properties following geotechnical recommendations. The resultant forces were converted to distributed pile loads as described in the following table. The lateral pile forces were then converted into longitudinal and transverse load components to orient the resultant force in the down slope direction.

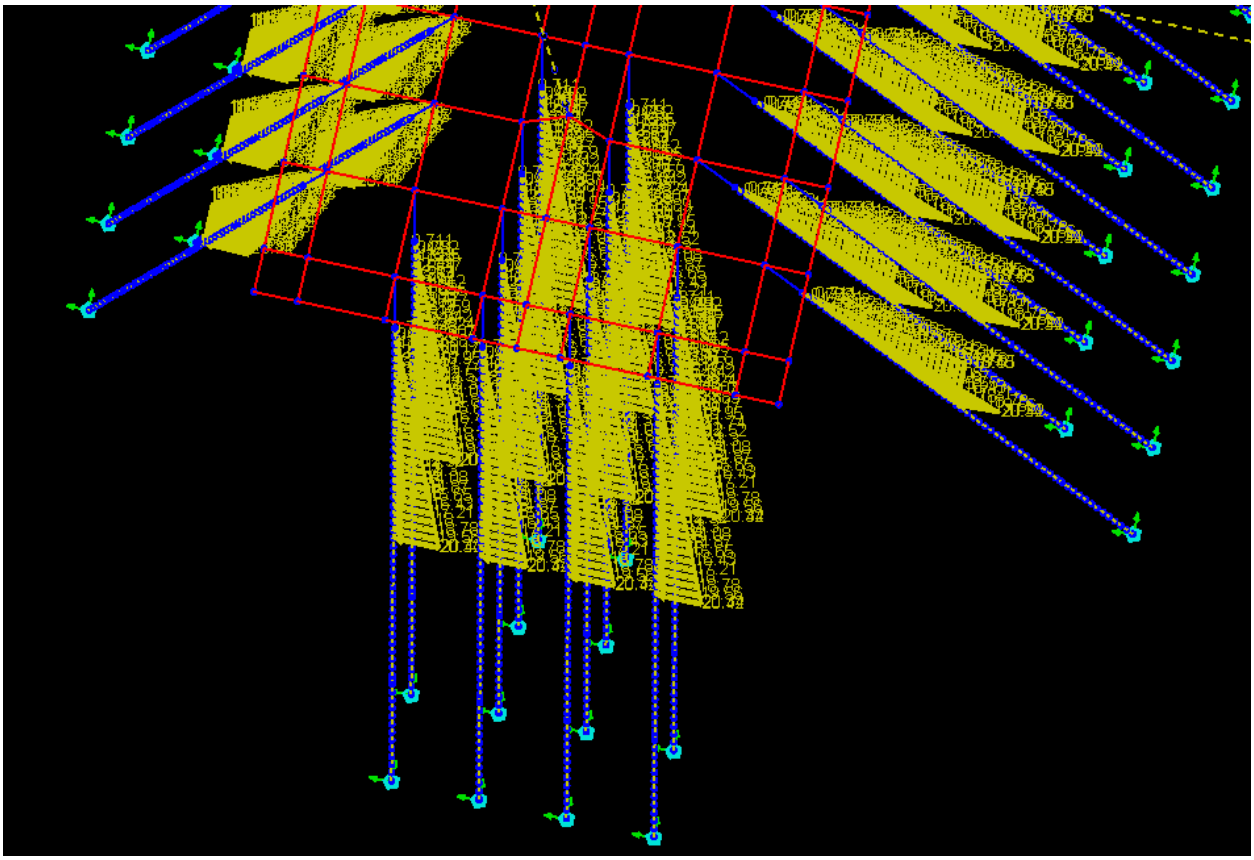
Lateral Loads	Pier 16	Pier 17		
H.critical	52.2	108	ft	below seal soffit
Resultant	240	180	kip/ft	per foot width
Pile Dia	3	3	ft	considered as the width
Resultant	720	540	kips	each pile
Top Load	0	0	klf	at the seal soffit
Base Load	27.59	10	klf	at depth H.critical
Soffit Seal	-23.5	-23.5	ft	elevation pile head
H.critical	-75.7	-131.5	ft	elevation base of loading
D.head	-52.2	-108	ft	depth below pile head

Lateral Load Oriengation		
θ.LS.long	42.2	deg
LS.trans	0.67	sin(θ.LS)
LS.long	0.74	cos(θ.LS)

Later, it was determined that the Geotech had a more simplified recommendation for applying the lateral soil forces. It was recommended to apply the resultant forces on a per foot width basis, where width was taken from the pile cap dimension. The loading applied to the individual piles based on their width exceeded this more simplified recommendations, so this more conservative interpretation of the loading was maintained. The following calculations provide a comparison of the applied loading, using a diagonal dimension of the 99 ft x 54 ft footing as 113 ft.

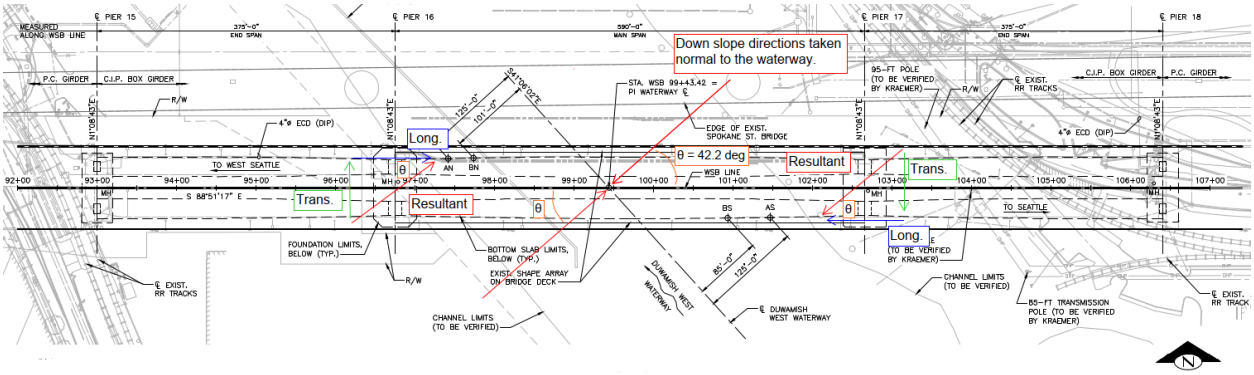
- Pier 16 with 56 Piles
  - Combined Pile Force = 56 (3 ft) (240 kip/ft) = 40320 kip
  - Geotech Recommendation = 113 ft (240 kip/ft) = 27120 kip
  - Conservatism Factor = 40320 / 27120 = 1.5
    - Conservative evaluation maintained; future work can revise the loading.
- Pier 17 with 60 Piles
  - Combined Pile Force = 60 (3 ft) (180 kip/ft) = 32400 kip
  - Geotech Recommendation = 113 ft (180 kip/ft) = 20340 kip
  - Conservatism Factor = 32400 / 20340 = 1.6
    - Conservative evaluation maintained; future work can revise the loading.

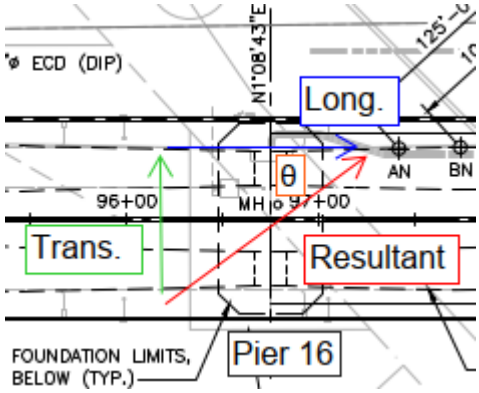
The following diagram depicts the longitudinal and transverse pile loading applied at Pier 16 oriented in the down slope direction. For the batter piles, the loads were assigned such that they were orthogonal to the axial direction of the pile in both the longitudinal and transverse directions. So far, only the more conservative red triangular distribution of loading over the height H.critical has been evaluated. The pink triangular distribution over a height (H + H.critical) represents a second load case including the height of the footing and concrete seal that has the same resultant forces (240 klf for Pier 16 and 180 klf for Pier 17) as the loading evaluated. This case is not expected to significantly alter the results.

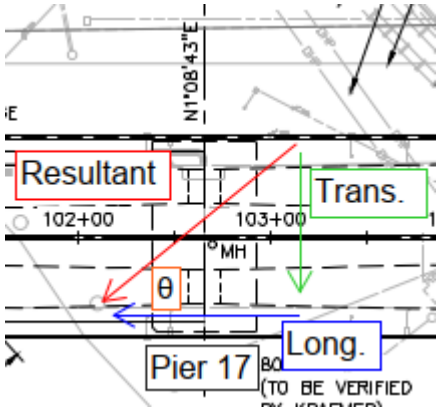


7.1 OVERSTRENGTH PUSHOVER CASES WITH LATERAL SOIL FORCES

The diagram of the resultant load directions for the recommended lateral soil forces is repeated below to aid in interpretation of the directionality for the pushover analyses.



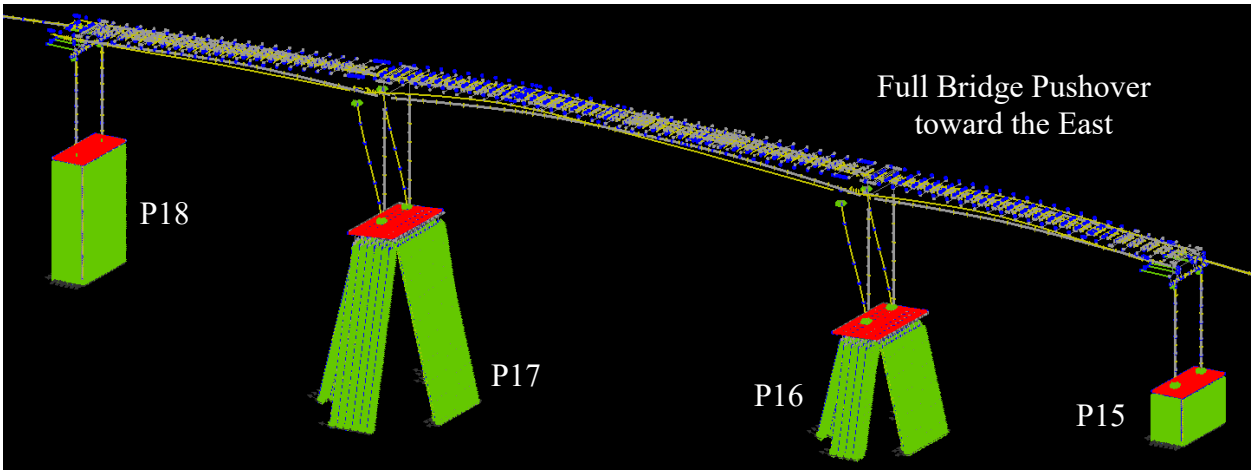


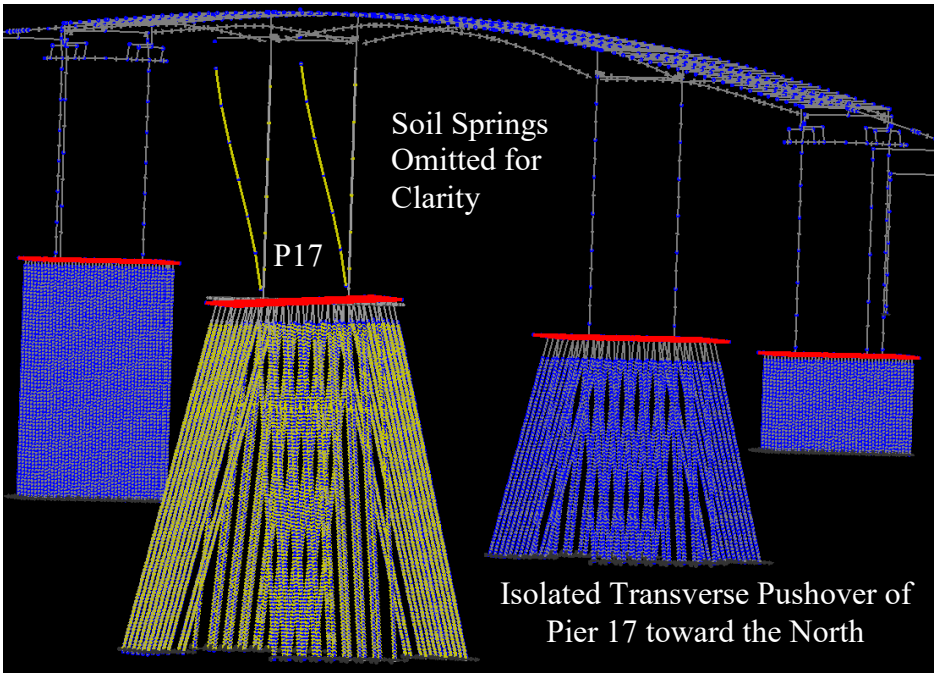


The following pushover cases were considered. The foundations for Piers 15 and 18 were evaluated for flexural overstrength demands, since no lateral soil forces corresponding to slope instability effects were recommended in the draft Geotechnical report dated 10/5/2020.

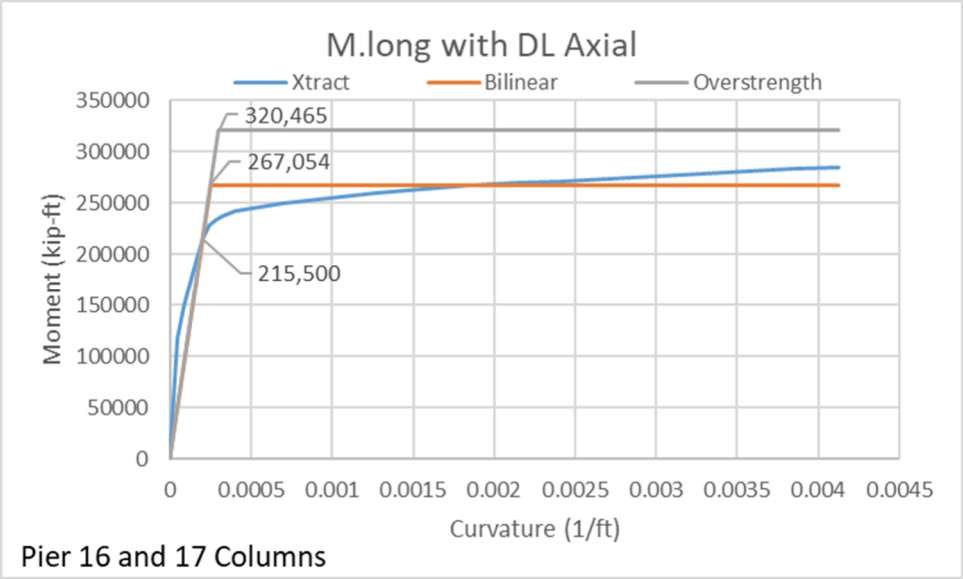
- Full Bridge Overstrength (M_{po}) Longitudinal Pushover with Lateral Soil Forces
  - Pushover to the East
  - Pushover to the West
- Pier 16 Overstrength (M_{po}) Transverse Pushover with Lateral Soil Forces
  - Pushover to the North
  - Pushover to the South
- Pier 17 Overstrength (M_{po}) Transverse Pushover with Lateral Soil Forces
  - Pushover to the North
  - Pushover to the South
- Pier 15 Overstrength (M_{po}) Longitudinal Pushover (same EW since no lateral soil forces)
- Pier 15 Overstrength (M_{po}) Transverse Pushover (same NS since no lateral soil forces)
- Pier 18 Overstrength (M_{po}) Longitudinal Pushover (no lateral soil forces provided)
  - Separate East/West Pushover Cases with Asymmetric 36” Repair Piles
- Pier 18 Overstrength (M_{po}) Transverse Pushover (no lateral soil forces provided)
  - Separate North/South Pushover Cases with Asymmetric 36” Repair Piles

In the full bridge longitudinal pushover, the entire structure was pushed in either the East or West directions as shown below. In the isolated Pier pushover cases, only the specific pier being evaluated was included with the correct column axial forces and restraint conditions. A sample case is shown below, where only Pier 17 is active in the model and the remaining portion of the structure (region greyed out) is excluded in the analysis using staged construction features in the software.



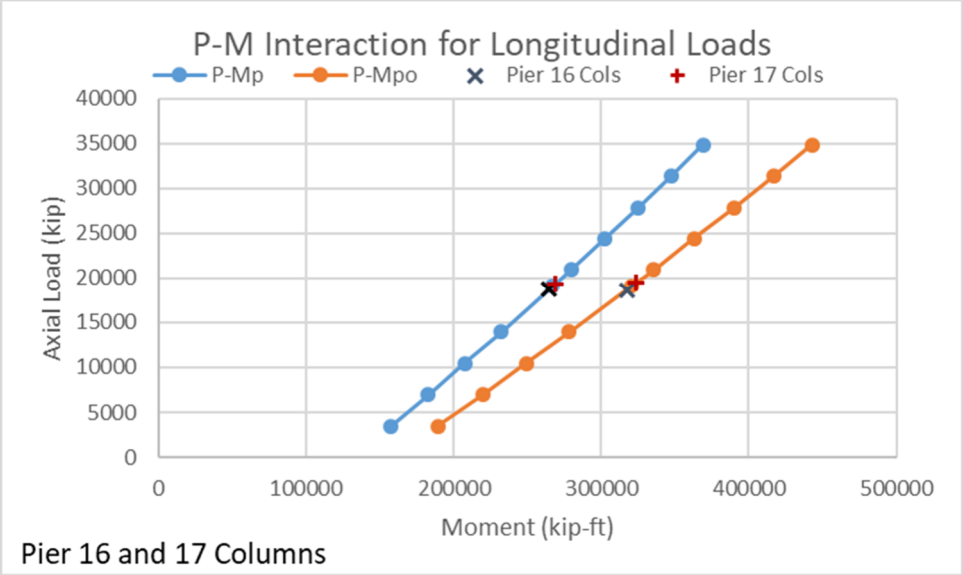


In the pushover analyses, the hinge properties were modified to include the overstrength moments Mpo consistent with the converged overstrength mechanism axial forces. A sample moment-curvature response is shown to illustrate the differences in the bilinear approximation to the moment-curvature response using both expected properties and amplified results using the SGS sectional overstrength factor of $\lambda mo = 1.2$ for the A706 column longitudinal reinforcement.

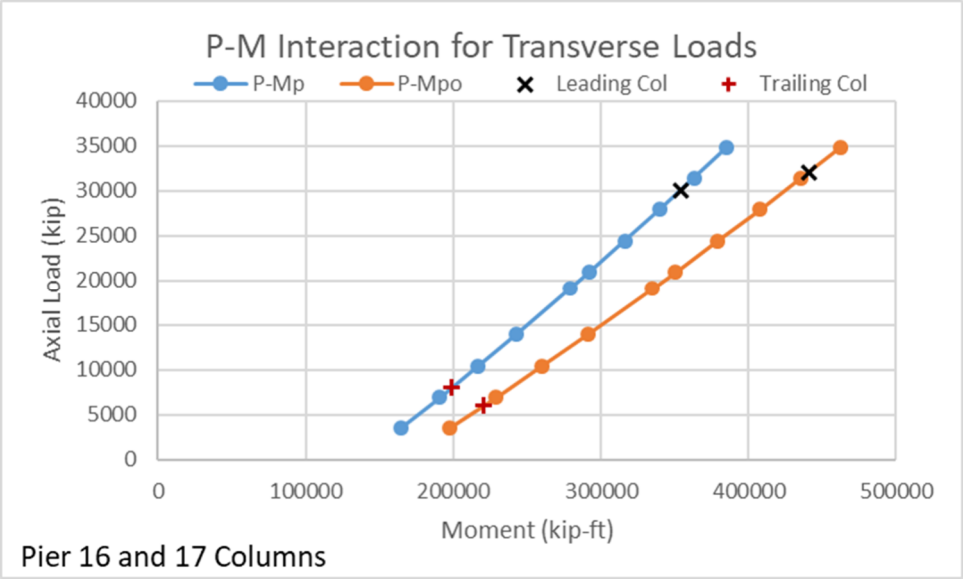


Pier 16 and 17 Columns

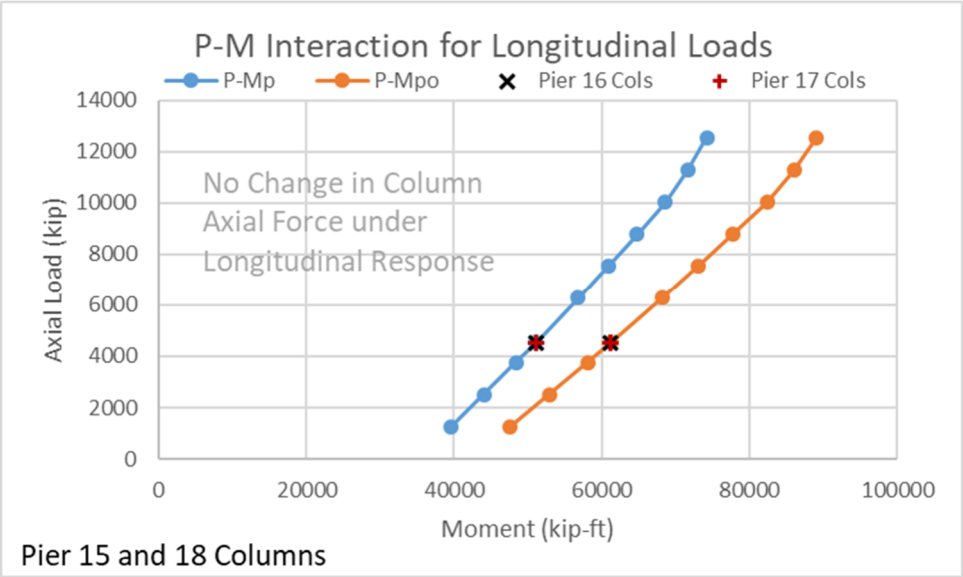
The following provides the axial load versus moment interaction diagram at Mpo considered in the overstrength pushover analyses. Converged axial forces from the pushover results are shown.



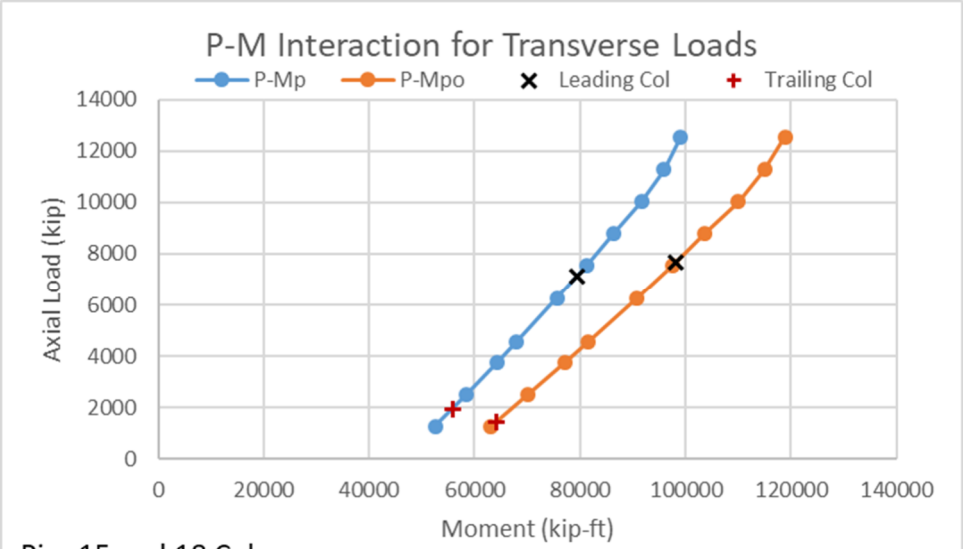
Pier 16 and 17 Columns



Pier 16 and 17 Columns



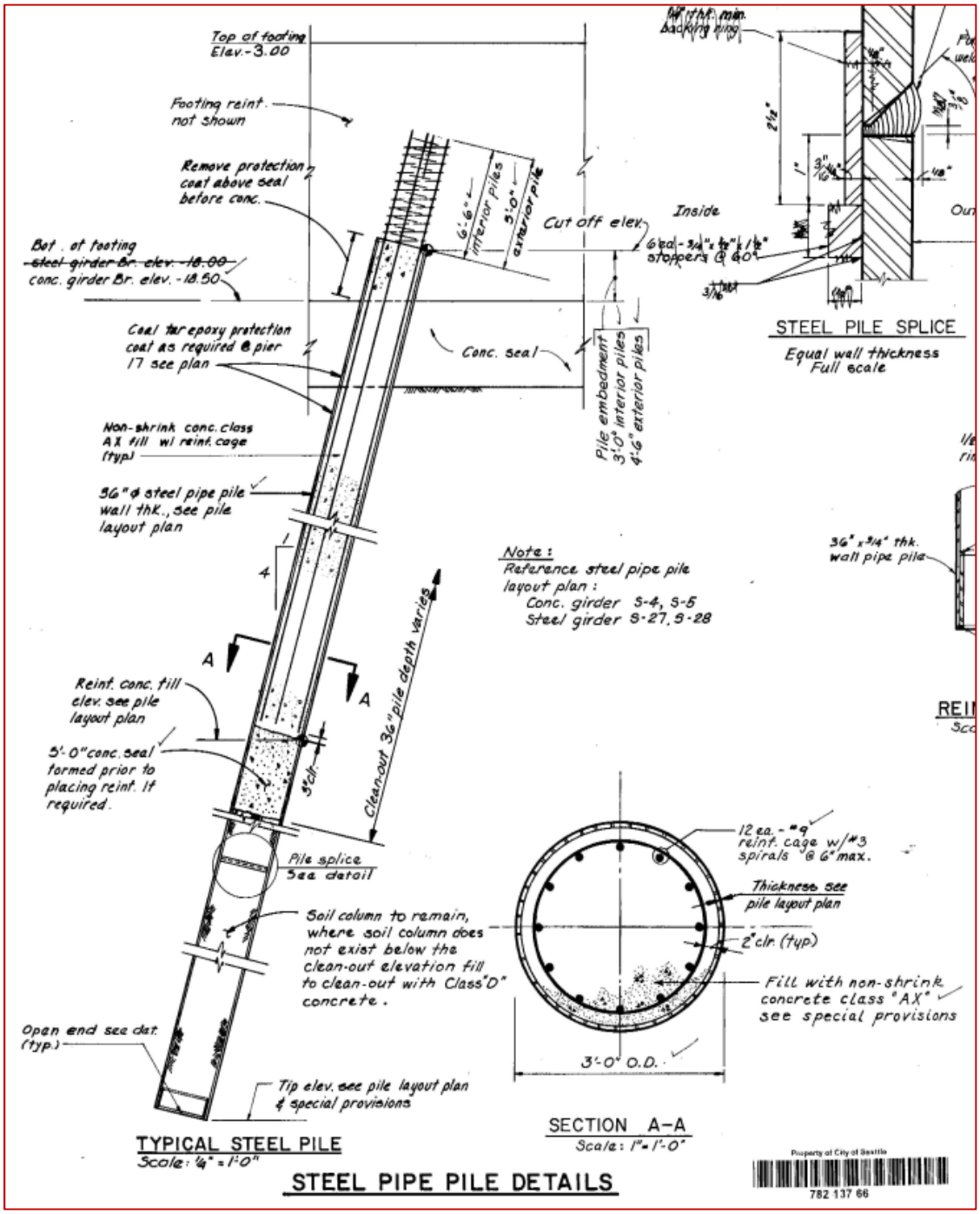
Pier 15 and 18 Columns



Pier 15 and 18 Columns

7.2 P16/17 PILE GEOMETRY AND DETAILS FOR THE RC FILLED PORTION

Details for the 36” diameter steel pipe piles from the as-built plans are shown below. The upper region of the pile acts as a reinforced concrete filled steel tube (RCFST), with consideration for lack of axial development of the pile wall material near the foundation soffit. Elevations for the RCFST to open pile transition and transition between 1” and 0.75” wall thickness transition were obtained from the as-built plans. The maximum pile moments occur within the RCFST region. Special attention was given to the effective material properties and moment-curvature response.



7.3 P16/17 RCFST MOMENT-CURVATURE AND PILE SHELL DEVELOPMENT

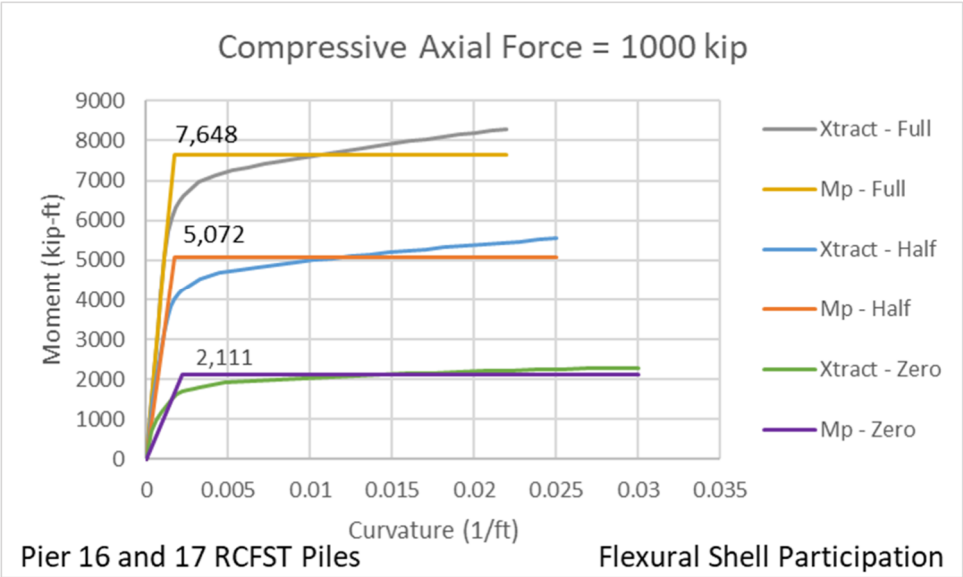
Due to uncertainty regarding the axial development of the pile at the head locations beneath the concrete seal, moment-curvature analysis was performed with full, half, and no flexural participation of the steel shell. In all three cases, the steel shell provides confinement to the core concrete. The pile wall material was assumed as A252 Gr3 material with a yield stress fy = 45 ksi, since the steel designation for the steel piles cannot be located within the as-built plans. Results, presented later, indicate that the comparison of strength is insensitive to the assumed steel designation, and an adequate flexural/shear capacity is achieved with only minor participation of the pile wall material to the RCFST section.

Expected material properties for the steel pile were taken as 1.1fy and 1.1fu according to WSDOT BDM Chapter 7.10, which agrees with factors applied in the SDOT BSRPPC. Expected concrete and mild reinforcing steel properties were considered in moment-curvature analysis to determine the plastic moment capacity of the section, Mp, following the SGS. The ultimate curvature was taken as first exceedance of the SGS reduced ultimate steel tensile strain for the A615 dowels bars or the Mander et al. (1988) ultimate concrete compressive strain (conservative max of 0.02). For cases that considered full or half participation of the steel shell, the ultimate concrete compressive strain controlled. The piles satisfied D/t limits for both elastic and plastic forces in WSDOT BDM Chapter 7.10.

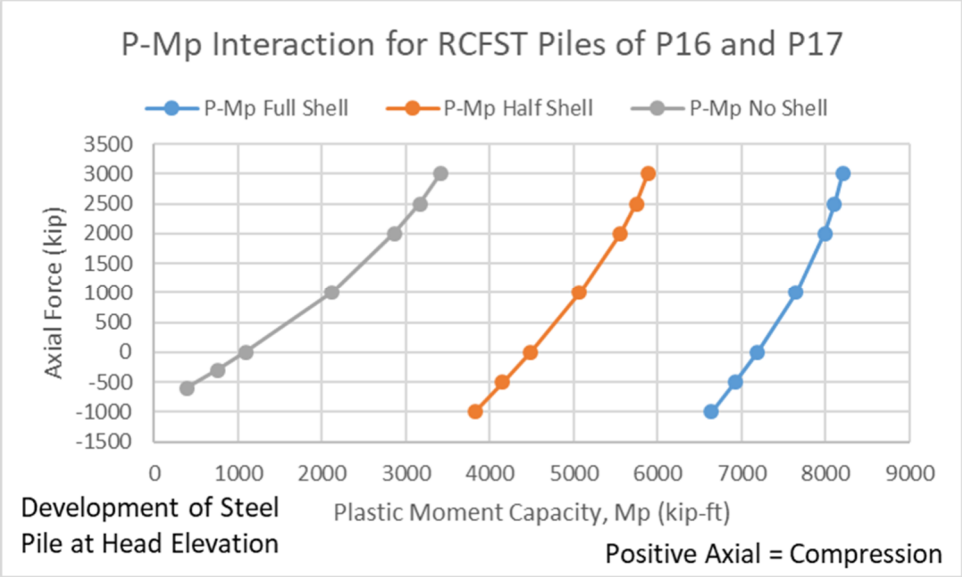
> For capacity protected members at the extreme event limit state, expected material properties may be used to determine the expected nominal moment capacity. The expected yield strength, F_{ye}, for steel tubes shall be taken as $1.1F_y$.

WSDOT BDM 7.10

Sample moment-curvature results with full, half, and no flexural participation of the steel shell at a compressive axial force of 1000 kips are shown below for the RCFST section with 1” wall thickness.



A complete evaluation of the axial load versus moment interaction behavior on the plastic moment capacity of the RCFST piles considering full, half and no flexural participation of the steel shell is shown in the following figure and tables.



Full Flexural Participation of the Steel Shell			
Axial (kip)	Mp (kip-ft)	ϕy (1/ft)	ϕu (1/ft)
3000	8208	0.00168	0.01700
2500	8112	0.00170	0.01794
2000	7992	0.00171	0.01918
1000	7648	0.00170	0.02198
0	7195	0.00166	0.02554
-500	6925	0.00164	0.02723
-1000	6641	0.00162	0.02926

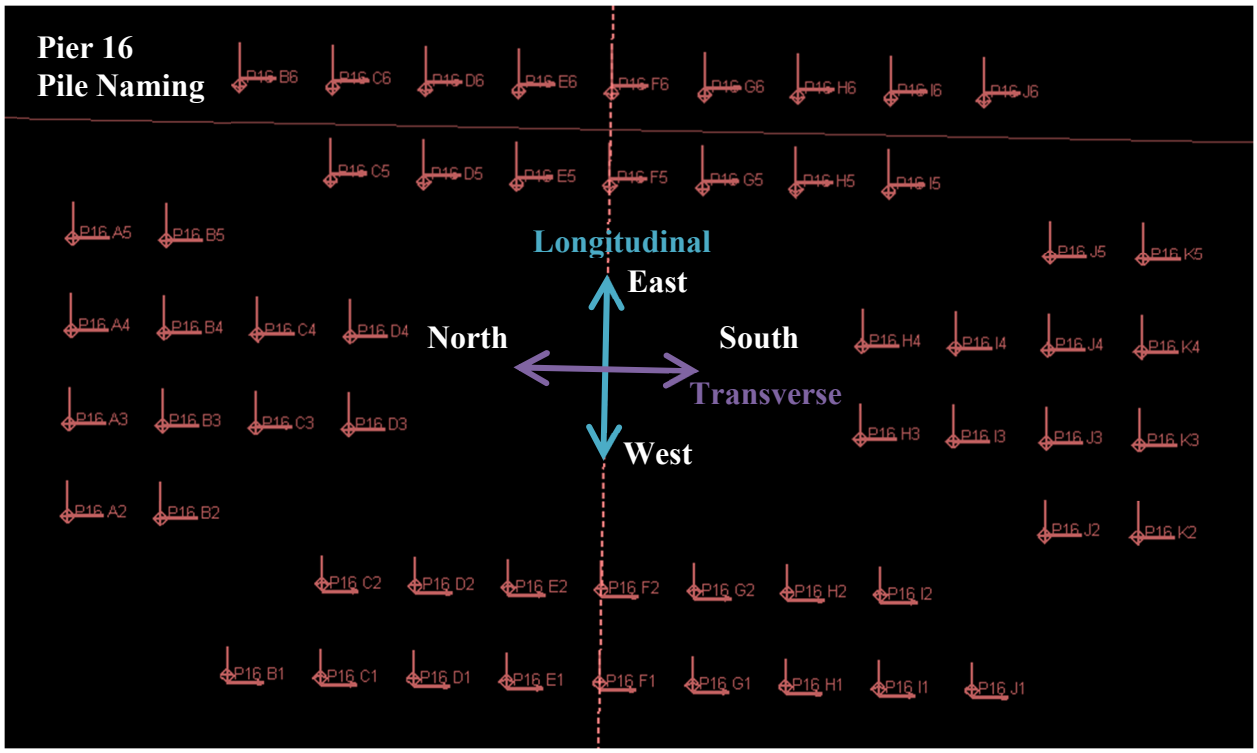
Half Flexural Participation of the Steel Shell			
Axial (kip)	Mp (kip-ft)	ϕy (1/ft)	ϕu (1/ft)
3000	5888	0.00167	0.01786
2500	5747	0.00171	0.01951
2000	5559	0.00174	0.02110
1000	5072	0.00173	0.02502
0	4488	0.00169	0.03100
-500	4155	0.00168	0.03508
-1000	3828	0.00171	0.04116

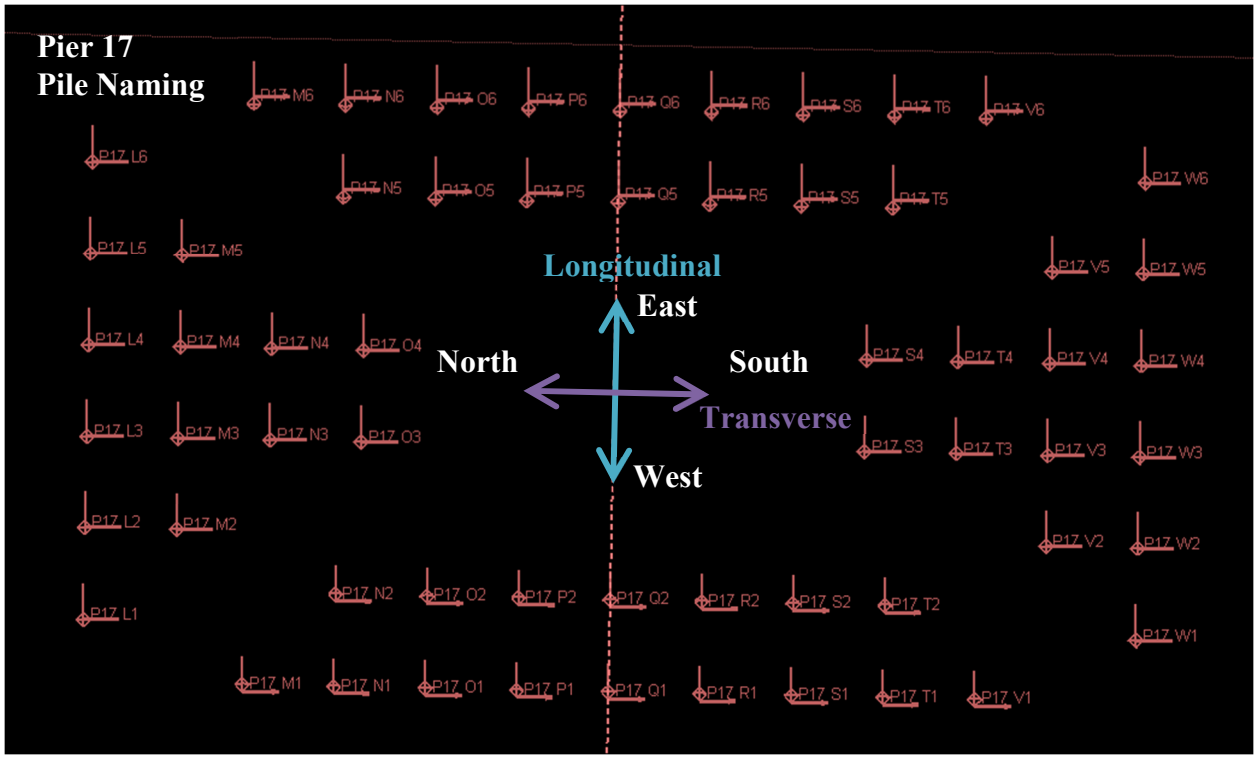
No Flexural Participation of the Steel Shell			
Axial (kip)	Mp (kip-ft)	ϕy (1/ft)	ϕu (1/ft)
3000	3410	0.00229	0.02005
2500	3166	0.00249	0.02254
2000	2860	0.00250	0.02474
1000	2111	0.00221	0.02999
0	1094	0.00201	0.02720
-300	755	0.00222	0.02602
-600	391	0.00167	0.02500

The sensitivity of the results to the participation of the steel shell at the pile head elevation will be demonstrated later, once the demands are shown.

7.4 PIER 16 AND PIER 17 FOUNDATION RESULTS

The following pile naming convention from the as-built plans is used for presentation of results.





Pier 16 Pile Head Load Effects		Full Bridge Mpo Long. Pushover (East) with Soil Forces						Full Bridge Mpo Long. Pushover (West) with Soil Forces					
		(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell
Pile	Member	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr
A2	10100-0	-423	56	428	17.3	11.1	3.6	-1647	59	620	12.7	8.7	4.2
A3	10200-0	-800	55	430	17.6	11.5	4.5	-1226	58	611	12.7	8.5	3.8
A4	10300-0	-1177	55	434	17.8	11.9	5.2	-803	58	605	12.5	8.2	3.2
A5	10400-0	-1552	56	439	17.9	12.2	5.8	-380	57	601	12.3	7.9	2.5
B2	10500-0	-447	55	429	17.3	11.1	3.7	-1671	60	624	12.6	8.7	4.2
B3	10600-0	-825	56	428	17.7	11.6	4.5	-1252	59	617	12.6	8.4	3.8
B4	10700-0	-1202	56	433	17.9	12.0	5.3	-828	58	607	12.5	8.2	3.2
B5	10800-0	-1576	56	440	17.9	12.2	5.8	-403	57	600	12.3	7.9	2.6
C3	10900-0	-849	51	455	16.7	11.0	4.3	-1279	55	595	13.0	8.8	3.9
C4	11000-0	-1230	52	454	17.1	11.4	5.1	-852	54	582	13.0	8.6	3.4
D3	11100-0	-871	55	442	17.2	11.3	4.5	-1297	59	622	12.5	8.4	3.8
D4	11200-0	-1248	55	442	17.5	11.8	5.2	-874	58	613	12.4	8.2	3.2
B6	11300-0	-2146	24	834	9.6	6.7	3.6	391	41	507	13.8	8.4	1.3
C5	11400-0	-1797	10	755	10.5	7.2	3.6	-56	58	590	12.2	7.7	2.0
C6	11500-0	-2167	25	838	9.6	6.7	3.6	368	42	511	13.7	8.3	1.3
D5	11600-0	-1817	10	757	10.5	7.2	3.6	-79	60	602	12.0	7.6	2.0
D6	11700-0	-2187	26	842	9.5	6.7	3.5	344	43	518	13.5	8.3	1.4
E5	11800-0	-1836	10	760	10.4	7.2	3.6	-102	61	611	11.9	7.5	2.0
E6	11900-0	-2206	27	846	9.5	6.6	3.5	322	44	526	13.4	8.2	1.4
F5	12000-0	-1855	11	763	10.4	7.2	3.6	-123	62	618	11.7	7.4	2.0
F6	12100-0	-2226	28	850	9.5	6.6	3.5	300	45	533	13.2	8.1	1.4
G5	12200-0	-1878	12	767	10.4	7.2	3.6	-143	63	622	11.7	7.4	2.0
G6	12300-0	-2248	29	855	9.4	6.6	3.5	281	46	539	13.1	8.0	1.4
H5	12400-0	-1901	13	775	10.3	7.1	3.6	-161	63	623	11.7	7.4	2.0
H6	12500-0	-2271	31	862	9.3	6.6	3.5	263	47	542	13.0	8.0	1.5
I5	12600-0	-1923	15	786	10.1	7.0	3.6	-180	64	624	11.7	7.4	2.1
I6	12700-0	-2293	32	872	9.2	6.5	3.5	245	47	544	13.0	8.0	1.5
J6	12800-0	-2314	34	882	9.2	6.4	3.5	226	48	549	12.9	7.9	1.5
B1	12900-0	336	44	492	14.3	8.7	1.4	-2255	46	353	22.8	16.0	8.6
C1	13000-0	313	45	490	14.4	8.8	1.5	-2276	46	356	22.6	15.9	8.5
C2	13100-0	-65	58	432	16.7	10.5	2.7	-1861	29	346	22.9	15.8	8.0
D1	13200-0	289	46	485	14.5	8.9	1.6	-2296	46	359	22.5	15.8	8.5
D2	13300-0	-89	60	427	16.9	10.7	2.8	-1880	29	350	22.7	15.7	8.0
E1	13400-0	265	47	481	14.7	9.0	1.7	-2314	47	359	22.5	15.8	8.5
E2	13500-0	-112	61	423	17.1	10.8	2.9	-1898	29	351	22.7	15.7	8.0
F1	13600-0	243	48	480	14.7	9.1	1.7	-2334	48	357	22.6	15.9	8.6
F2	13700-0	-134	62	421	17.2	10.9	3.0	-1918	30	348	22.9	15.8	8.1
G1	13800-0	223	48	480	14.8	9.1	1.8	-2356	48	356	22.7	16.0	8.7
G2	13900-0	-154	62	421	17.3	10.9	3.0	-1940	31	345	23.1	16.0	8.2
H1	14000-0	204	49	480	14.8	9.1	1.8	-2379	49	357	22.7	16.0	8.7
H2	14100-0	-174	62	422	17.3	10.9	3.1	-1964	32	346	23.1	16.0	8.2
I1	14200-0	186	49	478	14.9	9.2	1.9	-2401	50	361	22.4	15.8	8.6
I2	14300-0	-192	62	424	17.2	10.9	3.1	-1986	34	349	22.9	15.9	8.2
J1	14400-0	166	50	474	15.0	9.3	1.9	-2422	51	366	22.1	15.6	8.5
H3	14500-0	-742	57	478	15.8	10.3	3.9	-1169	57	637	12.1	8.1	3.5
H4	14600-0	-1119	58	470	16.4	10.9	4.7	-746	56	634	11.9	7.8	2.9
I3	14700-0	-762	53	500	15.1	9.9	3.8	-1192	54	619	12.5	8.4	3.7
I4	14800-0	-1143	55	488	15.8	10.5	4.6	-766	52	616	12.3	8.0	3.1
J2	14900-0	-401	56	490	15.1	9.7	3.1	-1626	57	638	12.3	8.4	4.1
J3	15000-0	-781	57	485	15.6	10.2	3.9	-1207	58	644	12.0	8.1	3.5
J4	15100-0	-1158	58	475	16.2	10.8	4.7	-784	57	646	11.7	7.7	2.9
J5	15200-0	-1531	59	468	16.8	11.4	5.4	-359	57	644	11.4	7.3	2.3
K2	15300-0	-420	57	488	15.2	9.7	3.2	-1644	57	636	12.4	8.5	4.1
K3	15400-0	-798	57	484	15.6	10.3	4.0	-1223	57	641	12.1	8.1	3.6
K4	15500-0	-1175	58	478	16.1	10.8	4.7	-801	57	643	11.8	7.7	3.0
K5	15600-0	-1550	58	470	16.7	11.4	5.4	-377	57	646	11.4	7.3	2.3

Pier 16 Pile Loading	Full Bridge Mpo Long. Pushover (East) with Soil Forces						Full Bridge Mpo Long. Pushover (West) with Soil Forces					
	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell
Location	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr
Min for Pile Head	-2314	10	421	9.2	6.4	1.45	-2422	29	345	11.4	7.3	1.27
Max for Pile Head	336	62	882				391	64	646			
Min for Entire Pile	-2424	0	0	8.6	5.3	0.97	-2527	0	0	8.6	5.3	0.91
Max for Entire Pile	336	207	882				391	208	848			

Pier 16 Pile Head Load Effects		Pier 16 Mpo Transverse Pushover (North) with Soil Forces						Pier 16 Mpo Transverse Pushover (South) with Soil Forces					
		(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell
Pile	Member	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr
A2	10100-0	-2607	57	413	19.7	14.0	7.8	504	61	660	10.5	6.3	0.8
A3	10200-0	-2593	57	407	20.0	14.2	7.9	522	61	656	10.5	6.3	0.7
A4	10300-0	-2573	57	397	20.5	14.5	8.1	540	60	655	10.5	6.3	0.7
A5	10400-0	-2550	57	392	20.7	14.7	8.1	557	60	655	10.5	6.3	0.7
B2	10500-0	-2362	58	415	19.5	13.7	7.4	220	62	663	10.7	6.6	1.3
B3	10600-0	-2350	57	407	19.9	14.0	7.6	238	61	658	10.8	6.6	1.3
B4	10700-0	-2331	57	399	20.2	14.2	7.7	256	61	655	10.8	6.6	1.2
B5	10800-0	-2304	57	389	20.7	14.6	7.9	273	61	657	10.7	6.6	1.2
C3	10900-0	-2106	57	414	19.4	13.5	7.1	-48	61	660	10.9	6.9	1.7
C4	11000-0	-2087	57	397	20.2	14.1	7.4	-30	61	656	11.0	6.9	1.7
D3	11100-0	-1853	57	415	19.1	13.2	6.7	-332	62	663	11.1	7.1	2.2
D4	11200-0	-1833	56	397	20.0	13.8	6.9	-314	61	661	11.1	7.1	2.2
B6	11300-0	-1951	100	857	9.3	6.4	3.3	155	66	645	11.0	6.8	1.4
C5	11400-0	-1730	93	805	9.8	6.7	3.3	-148	65	689	10.5	6.7	1.8
C6	11500-0	-1704	92	797	9.9	6.8	3.3	-128	65	685	10.6	6.7	1.8
D5	11600-0	-1482	86	749	10.5	7.1	3.3	-431	68	739	10.0	6.4	2.1
D6	11700-0	-1458	85	741	10.6	7.2	3.3	-410	67	733	10.1	6.5	2.1
E5	11800-0	-1238	81	693	11.2	7.5	3.3	-714	74	791	9.5	6.2	2.3
E6	11900-0	-1215	80	685	11.3	7.6	3.3	-692	73	784	9.6	6.3	2.3
F5	12000-0	-997	75	632	12.1	8.0	3.3	-997	80	843	9.1	6.0	2.5
F6	12100-0	-975	74	624	12.2	8.1	3.3	-975	79	836	9.1	6.1	2.5
G5	12200-0	-760	69	568	13.3	8.7	3.3	-1283	87	892	8.7	5.9	2.6
G6	12300-0	-738	68	560	13.5	8.8	3.3	-1261	86	885	8.8	5.9	2.6
H5	12400-0	-524	64	505	14.7	9.5	3.3	-1575	94	941	8.3	5.7	2.7
H6	12500-0	-503	64	498	14.9	9.6	3.3	-1550	93	934	8.4	5.7	2.7
I5	12600-0	-287	62	443	16.6	10.6	3.2	-1869	102	995	8.0	5.5	2.8
I6	12700-0	-267	61	437	16.8	10.7	3.2	-1843	101	988	8.0	5.5	2.8
J6	12800-0	-30	61	379	19.0	11.9	3.0	-2137	110	1048	7.7	5.3	2.8
B1	12900-0	-2009	102	156	51.2	35.6	18.4	89	65	699	10.2	6.4	1.4
C1	13000-0	-1763	95	122	65.0	44.7	22.2	-193	66	658	11.1	7.0	2.0
C2	13100-0	-1752	94	124	63.7	43.8	21.7	-177	65	657	11.1	7.0	2.0
D1	13200-0	-1518	88	112	69.9	47.6	22.5	-475	69	623	11.9	7.7	2.6
D2	13300-0	-1505	87	116	67.2	45.7	21.6	-459	68	621	11.9	7.7	2.6
E1	13400-0	-1276	82	129	60.2	40.5	18.1	-756	73	592	12.8	8.4	3.2
E2	13500-0	-1262	82	133	58.3	39.2	17.5	-741	72	590	12.8	8.4	3.2
F1	13600-0	-1038	77	166	46.2	30.7	12.9	-1038	78	565	13.6	9.0	3.8
F2	13700-0	-1023	77	169	45.4	30.2	12.6	-1023	78	562	13.6	9.0	3.8
G1	13800-0	-802	72	215	35.2	23.1	8.9	-1323	84	543	14.3	9.6	4.4
G2	13900-0	-787	72	218	34.6	22.7	8.7	-1308	84	539	14.4	9.7	4.4
H1	14000-0	-567	68	270	27.7	17.9	6.3	-1611	91	530	14.8	10.1	4.9
H2	14100-0	-552	67	274	27.3	17.7	6.1	-1599	90	526	15.0	10.2	4.9
I1	14200-0	-332	64	326	22.6	14.4	4.5	-1902	98	528	15.1	10.4	5.3
I2	14300-0	-316	64	331	22.2	14.2	4.4	-1891	98	525	15.2	10.5	5.3
J1	14400-0	-97	62	383	18.9	11.9	3.2	-2195	107	538	14.9	10.4	5.6
H3	14500-0	-196	54	405	18.0	11.4	3.2	-1717	64	681	11.6	8.0	3.9
H4	14600-0	-179	54	406	18.0	11.4	3.2	-1698	64	667	11.8	8.1	4.0
I3	14700-0	41	53	412	17.4	10.8	2.6	-2017	64	678	11.8	8.2	4.2
I4	14800-0	59	53	409	17.5	10.9	2.5	-1998	64	664	12.0	8.3	4.3
J2	14900-0	263	54	412	17.1	10.5	1.9	-2319	65	684	11.8	8.3	4.5
J3	15000-0	280	53	415	17.0	10.4	1.9	-2307	65	674	12.0	8.4	4.5
J4	15100-0	298	53	414	17.0	10.4	1.8	-2289	64	667	12.1	8.5	4.6
J5	15200-0	316	53	413	17.0	10.4	1.8	-2262	65	662	12.2	8.5	4.6
K2	15300-0	500	54	415	16.7	10.0	1.2	-2611	65	677	12.0	8.5	4.8
K3	15400-0	518	53	417	16.6	10.0	1.2	-2597	65	672	12.1	8.6	4.8
K4	15500-0	535	53	419	16.5	9.9	1.1	-2578	64	663	12.3	8.7	4.8
K5	15600-0	553	53	419	16.5	9.9	1.1	-2555	64	658	12.4	8.8	4.9

Pier 16 Pile Loading	Pier 16 Mpo Transverse Pushover (North) with Soil Forces						Pier 16 Mpo Transverse Pushover (South) with Soil Forces					
	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell
Location	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr
Min for Pile Head	-2607	53	112	9.3	6.4	1.07	-2611	60	525	7.7	5.3	0.68
Max for Pile Head	553	102	857				557	110	1048			
Min for Entire Pile	-2610	0	0	8.4	5.1	0.80	-2826	0	0	7.7	5.0	0.48
Max for Entire Pile	553	207	857				604	209	1048			

Pier 17 Pile Head Load Effects		Full Bridge Mpo Long. Pushover (East) with Soil Forces						Full Bridge Mpo Long. Pushover (West) with Soil Forces					
		(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell
Pile	Member	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr
L1	20100-0	11	164	146	49.4	30.9	7.5	-1758	172	192	41.2	28.3	14.1
L2	20200-0	-352	163	140	52.8	33.8	10.6	-1404	170	182	42.7	28.9	13.4
L3	20300-0	-715	162	137	55.0	35.9	13.4	-1049	167	167	46.0	30.6	12.9
L4	20400-0	-1077	162	138	55.8	37.1	15.7	-693	164	150	50.1	32.7	12.1
L5	20500-0	-1438	162	137	56.9	38.6	18.0	-336	161	137	53.8	34.4	10.7
L6	20600-0	-1797	161	132	60.2	41.5	20.8	21	159	126	56.9	35.5	8.5
M2	20700-0	-345	161	133	55.2	35.3	11.0	-1397	171	189	41.3	27.9	12.9
M3	20800-0	-709	162	138	54.6	35.7	13.3	-1043	169	176	43.6	29.0	12.2
M4	20900-0	-1071	163	142	54.2	36.1	15.3	-686	165	155	48.4	31.6	11.7
M5	21000-0	-1431	162	139	56.3	38.2	17.7	-328	161	135	54.5	34.8	10.8
N3	21100-0	-701	151	72	104.0	67.9	25.2	-1037	160	127	60.4	40.1	16.8
N4	21200-0	-1065	153	87	88.7	59.0	24.9	-679	155	96	78.5	51.2	18.9
O3	21300-0	-692	159	121	62.2	40.6	15.0	-1027	169	177	43.3	28.7	12.0
O4	21400-0	-1055	161	131	58.5	38.9	16.4	-670	164	152	49.5	32.2	11.8
M6	21500-0	-2075	329	2130	3.8	2.6	1.4	174	143	31	226.4	139.7	28.7
N5	21600-0	-1708	243	1743	4.5	3.1	1.5	-175	216	387	18.8	11.9	3.3
N6	21700-0	-2067	327	2120	3.8	2.6	1.4	182	141	22	318.2	196.2	40.0
O5	21800-0	-1699	241	1729	4.6	3.1	1.5	-167	216	390	18.7	11.8	3.3
O6	21900-0	-2058	325	2110	3.8	2.6	1.4	190	141	19	373.0	229.9	46.4
P5	22000-0	-1689	238	1718	4.6	3.1	1.5	-159	216	391	18.6	11.8	3.3
P6	22100-0	-2048	323	2102	3.8	2.6	1.4	198	140	17	413.8	254.9	51.0
Q5	22200-0	-1679	237	1711	4.6	3.2	1.5	-150	215	387	18.8	11.9	3.3
Q6	22300-0	-2039	321	2095	3.8	2.7	1.4	207	139	14	509.0	313.3	62.0
R5	22400-0	-1671	235	1705	4.6	3.2	1.5	-140	213	377	19.3	12.1	3.3
R6	22500-0	-2030	320	2089	3.8	2.7	1.4	217	137	12	573.6	352.8	69.0
S5	22600-0	-1662	235	1702	4.6	3.2	1.5	-130	210	362	20.1	12.7	3.4
S6	22700-0	-2021	319	2083	3.8	2.7	1.4	227	135	18	402.5	247.4	47.8
T5	22800-0	-1653	234	1702	4.6	3.2	1.5	-119	207	345	21.1	13.3	3.6
T6	22900-0	-2011	318	2080	3.8	2.7	1.4	238	132	26	275.5	169.2	32.2
V6	23000-0	-2001	317	2076	3.8	2.7	1.4	249	131	30	235.5	144.5	27.2
M1	23100-0	172	151	71	100.8	62.2	12.8	-2029	327	2119	3.8	2.6	1.4
N1	23200-0	180	149	57	125.0	77.1	15.8	-2021	324	2104	3.8	2.6	1.4
N2	23300-0	-184	224	426	17.1	10.8	3.1	-1668	242	1733	4.6	3.1	1.5
O1	23400-0	187	147	49	145.6	89.8	18.2	-2012	320	2088	3.8	2.7	1.4
O2	23500-0	-177	223	424	17.2	10.9	3.0	-1659	238	1714	4.6	3.1	1.5
P1	23600-0	195	145	41	171.7	105.8	21.2	-2002	317	2075	3.8	2.7	1.4
P2	23700-0	-169	222	421	17.3	10.9	3.1	-1649	234	1697	4.6	3.2	1.5
Q1	23800-0	204	143	31	231.6	142.6	28.3	-1993	315	2062	3.9	2.7	1.4
Q2	23900-0	-160	221	411	17.7	11.2	3.1	-1640	231	1685	4.7	3.2	1.6
R1	24000-0	213	140	16	433.2	266.5	52.4	-1984	312	2050	3.9	2.7	1.4
R2	24100-0	-151	218	398	18.3	11.6	3.2	-1631	229	1673	4.7	3.2	1.6
S1	24200-0	223	137	5	1355.0	833.1	161.7	-1975	310	2039	3.9	2.7	1.4
S2	24300-0	-141	214	377	19.3	12.2	3.3	-1623	227	1664	4.7	3.2	1.6
T1	24400-0	234	134	18	384.4	236.1	45.2	-1966	307	2030	3.9	2.7	1.4
T2	24500-0	-130	210	354	20.5	12.9	3.5	-1614	225	1659	4.7	3.2	1.6
V1	24600-0	244	131	29	247.6	152.0	28.7	-1956	305	2020	3.9	2.7	1.4
S3	24700-0	-777	167	164	46.0	30.1	11.5	-1113	158	119	64.4	43.0	18.4
S4	24800-0	-1140	172	190	40.6	27.1	11.7	-756	156	110	68.5	44.8	17.0
T3	24900-0	-702	155	93	80.6	52.6	19.6	-1039	152	95	81.0	53.9	22.6
T4	25000-0	-1132	163	138	55.7	37.2	16.0	-747	148	64	117.5	76.9	29.1
V2	25100-0	-392	164	146	50.5	32.4	10.4	-1446	159	127	61.7	41.9	19.5
V3	25200-0	-757	168	167	45.3	29.7	11.3	-1092	160	130	58.9	39.3	16.7
V4	25300-0	-1120	171	187	41.1	27.5	11.8	-736	159	128	59.0	38.6	14.5
V5	25400-0	-1480	174	200	39.1	26.6	12.5	-378	158	124	59.5	38.1	12.1
W1	25500-0	-18	162	137	52.8	33.0	8.2	-1788	157	119	66.5	45.8	22.9
W2	25600-0	-381	164	147	50.2	32.2	10.3	-1434	159	125	62.3	42.2	19.6
W3	25700-0	-745	167	161	47.0	30.7	11.6	-1080	159	127	60.6	40.4	17.1
W4	25800-0	-1107	170	177	43.4	29.0	12.4	-724	159	127	59.4	38.9	14.6
W5	25900-0	-1469	173	193	40.6	27.5	12.9	-368	160	130	56.9	36.4	11.5
W6	26000-0	-1829	174	202	39.2	27.0	13.6	-11	161	136	53.0	33.1	8.2

Pier 17 Pile Loading		Full Bridge Mpo Long. Pushover (East) with Soil Forces						Full Bridge Mpo Long. Pushover (West) with Soil Forces					
		(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell
Location		Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr
Min for Pile Head		-2075	131	5	3.8	2.6	1.37	-2029	131	12	3.8	2.6	1.36
Max for Pile Head		244	329	2130				249	327	2119			
Min for Entire Pile		-2211	0	0	3.8	2.6	1.37	-2168	0	0	3.8	2.6	1.36
Max for Entire Pile		244	330	2130				249	328	2119			

Pier 17 Pile Head Load Effects		Pier 17 Mpo Transverse Pushover (North) with Soil Forces						Pier 17 Mpo Transverse Pushover (South) with Soil Forces					
		(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell
Pile	Member	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr
L1	20100-0	-2180	59	183	43.8	30.6	16.3	369	55	129	54.1	32.9	5.2
L2	20200-0	-2184	58	182	44.2	30.9	16.4	369	55	130	53.9	32.8	5.2
L3	20300-0	-2187	57	180	44.8	31.3	16.6	370	54	132	53.2	32.3	5.1
L4	20400-0	-2186	56	177	45.3	31.7	16.8	370	55	129	54.3	33.0	5.2
L5	20500-0	-2183	55	178	45.0	31.5	16.7	371	57	122	57.6	35.0	5.5
L6	20600-0	-2178	54	182	44.1	30.8	16.4	372	58	116	60.6	36.8	5.8
M2	20700-0	-1945	59	177	45.1	31.2	16.0	153	55	125	56.8	35.1	7.4
M3	20800-0	-1949	57	177	45.0	31.2	16.0	153	54	131	54.3	33.6	7.1
M4	20900-0	-1948	56	176	45.3	31.4	16.1	154	55	128	55.5	34.3	7.2
M5	21000-0	-1944	56	173	46.1	32.0	16.4	155	57	117	60.9	37.6	7.9
N3	21100-0	-1709	55	186	42.4	29.1	14.3	-64	55	129	56.0	35.1	9.1
N4	21200-0	-1709	54	183	43.1	29.6	14.6	-64	55	126	57.5	36.1	9.3
O3	21300-0	-1466	55	188	41.6	28.2	13.2	-281	56	122	60.0	38.2	11.5
O4	21400-0	-1466	53	186	42.1	28.6	13.4	-280	57	119	61.6	39.2	11.8
M6	21500-0	-1869	414	1970	4.0	2.8	1.4	-51	70	166	43.5	27.3	7.0
N5	21600-0	-1634	372	1752	4.5	3.1	1.5	-268	104	383	19.1	12.2	3.6
N6	21700-0	-1629	371	1743	4.5	3.1	1.5	-267	104	379	19.3	12.3	3.7
O5	21800-0	-1394	330	1526	5.1	3.5	1.6	-484	149	612	12.1	7.8	2.6
O6	21900-0	-1390	328	1517	5.1	3.5	1.6	-482	148	606	12.3	7.9	2.7
P5	22000-0	-1154	286	1298	5.9	4.0	1.7	-700	197	844	8.9	5.8	2.2
P6	22100-0	-1152	284	1289	6.0	4.0	1.7	-698	196	837	9.0	5.9	2.2
Q5	22200-0	-917	239	1057	7.2	4.8	1.9	-916	244	1074	7.1	4.7	1.9
Q6	22300-0	-914	237	1049	7.3	4.8	1.9	-913	242	1065	7.1	4.7	1.9
R5	22400-0	-680	188	809	9.3	6.1	2.2	-1133	288	1299	5.9	4.0	1.7
R6	22500-0	-678	187	803	9.4	6.1	2.2	-1130	286	1289	6.0	4.0	1.7
S5	22600-0	-444	138	561	13.2	8.5	2.8	-1351	328	1510	5.2	3.5	1.6
S6	22700-0	-442	137	556	13.3	8.6	2.8	-1348	327	1500	5.2	3.5	1.6
T5	22800-0	-207	93	318	22.9	14.5	4.2	-1571	368	1721	4.6	3.1	1.5
T6	22900-0	-206	93	316	23.1	14.6	4.2	-1566	367	1711	4.6	3.1	1.5
V6	23000-0	30	67	114	62.8	39.2	9.3	-1785	407	1925	4.1	2.8	1.4
M1	23100-0	-1867	423	2014	3.9	2.7	1.4	-51	69	178	40.5	25.4	6.5
N1	23200-0	-1627	380	1784	4.4	3.0	1.5	-267	101	388	18.9	12.0	3.6
N2	23300-0	-1632	381	1793	4.4	3.0	1.5	-268	102	390	18.8	12.0	3.6
O1	23400-0	-1388	336	1553	5.0	3.4	1.6	-483	144	609	12.2	7.9	2.6
O2	23500-0	-1392	338	1562	5.0	3.4	1.6	-484	145	614	12.1	7.8	2.6
P1	23600-0	-1150	292	1322	5.8	3.9	1.7	-698	190	834	9.0	5.9	2.2
P2	23700-0	-1152	294	1331	5.8	3.9	1.7	-700	191	840	9.0	5.8	2.2
Q1	23800-0	-913	244	1078	7.1	4.7	1.9	-914	237	1060	7.2	4.7	1.9
Q2	23900-0	-915	245	1086	7.0	4.6	1.9	-916	238	1068	7.1	4.7	1.9
R1	24000-0	-677	193	827	9.1	5.9	2.2	-1131	280	1278	6.0	4.0	1.7
R2	24100-0	-679	194	834	9.0	5.9	2.2	-1133	281	1287	6.0	4.0	1.7
S1	24200-0	-441	142	576	12.9	8.3	2.7	-1349	321	1489	5.2	3.5	1.6
S2	24300-0	-443	142	581	12.8	8.2	2.7	-1352	322	1498	5.2	3.5	1.6
T1	24400-0	-206	96	330	22.1	14.0	4.0	-1568	360	1695	4.6	3.2	1.5
T2	24500-0	-207	96	334	21.9	13.9	4.0	-1572	362	1704	4.6	3.1	1.5
V1	24600-0	30	68	119	60.1	37.5	8.9	-1787	400	1907	4.2	2.9	1.4
S3	24700-0	-361	61	127	57.9	37.1	11.7	-1546	49	179	43.8	29.8	14.2
S4	24800-0	-361	60	130	56.7	36.3	11.4	-1546	50	183	42.8	29.2	13.9
T3	24900-0	-58	57	148	48.9	30.7	7.9	-1703	52	161	49.0	33.6	16.5
T4	25000-0	-124	59	137	52.9	33.3	9.0	-1769	51	181	43.8	30.1	15.0
V2	25100-0	114	61	126	56.8	35.2	7.7	-1984	52	165	48.3	33.5	17.3
V3	25200-0	114	59	136	52.3	32.4	7.1	-1988	52	169	47.2	32.7	16.9
V4	25300-0	114	59	139	51.3	31.8	7.0	-1988	53	171	46.7	32.4	16.7
V5	25400-0	114	60	133	53.6	33.2	7.3	-1984	55	171	46.6	32.3	16.7
W1	25500-0	351	61	125	56.1	34.2	5.6	-2198	50	175	46.1	32.2	17.2
W2	25600-0	351	60	131	53.7	32.7	5.3	-2202	51	171	47.1	33.0	17.6
W3	25700-0	351	59	137	51.0	31.1	5.0	-2205	52	170	47.3	33.1	17.7
W4	25800-0	351	59	140	50.2	30.6	5.0	-2205	53	173	46.5	32.5	17.3
W5	25900-0	351	59	138	51.0	31.0	5.0	-2203	54	176	45.8	32.0	17.1
W6	26000-0	351	59	137	51.2	31.2	5.1	-2199	55	177	45.5	31.8	16.9

Pier 17 Pile Loading	Pier 17 Mpo Transverse Pushover (North) with Soil Forces						Pier 17 Mpo Transverse Pushover (South) with Soil Forces					
	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell	(kip)	(kip)	(kip-ft)	Full Shell	Half Shell	No Shell
Location	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr	Axial	Shear	Moment	Mp/Mr	Mp/Mr	Mp/Mr
Min for Pile Head	-2187	53	114	3.9	2.7	1.38	-2205	49	116	4.1	2.8	1.41
Max for Pile Head	351	423	2014				372	407	1925			
Min for Entire Pile	-2407	0	0	3.9	2.7	1.38	-2249	0	0	4.1	2.8	1.41
Max for Entire Pile	351	424	2014				372	408	1925			

7.4.1 P16/17 Investigation of Pile Axial Forces from the Governing Results

The following represents the envelope of pile axial forces from the Overstrength Pushover and Lateral Soil Force Cases for Piers 16 and 17.

Envelope of Pile Axial Forces (kip)		
Description	**Pier 16**	**Pier 17**
Tension	604	372
Comp	-2826	-2407

An adequate dowel capacity was provided to develop the maximum pile tensile axial force, as shown. Note that cases with tensile axial force require some participation of the steel shell for resistance to the applied moments, since the RC section plastic moment capacity is reduced when approaching axial force corresponding to tensile yielding of all reinforcement.

Tensile Strength of the Doweled Connection, Zero Bond for the Steel Pile into the Footing

$Ast := 12 \cdot \left(1in^2\right) = 12 \cdot in^2$ area of mild longitudinal reinforcement

$fy_{rebar} := 60ksi$ nominal yield stress of the dowel reinforcement

$fye_{rebar} := 68ksi$ expected yield stress of the dowel reinforcement

$Tn_{nominal} := Ast \cdot fy_{rebar} = 720 \cdot kip$ tensile capacity of the doweled connection

$Tn_{expected} := Ast \cdot fye_{rebar} = 816 \cdot kip$

Therefore, the maximum tensile axial load on the piles is less than the nominal yield strength of the doweled connection without considering any development for the steel pile embedded into the footing and concrete seal. Consideration for the higher strength provided by expected properties for the steel reinforcement was not needed.

A comparison of the envelope pile axial forces and the unfactored axial resistance of the piles from recommendations provided by Shannon and Wilson appears in the following figures. Annotations have been made over their recommendations to determine the unfactored axial resistance of the piles at the average pile tip elevation. Communications with the Shannon and Wilson indicated that downdrag effects on piles due to settlement are only applicable to the post-earthquake evaluation; therefore, they were not considered to act with the overstrength demands.

An adequate unfactored tension and compressive axial resistance exists in the piles relative to the maximum individual pile axial force demands from the assessment of overstrength and concurrent recommended lateral soil forces. This is demonstrated through annotations over the Geotechnical recommendations on the following pages.

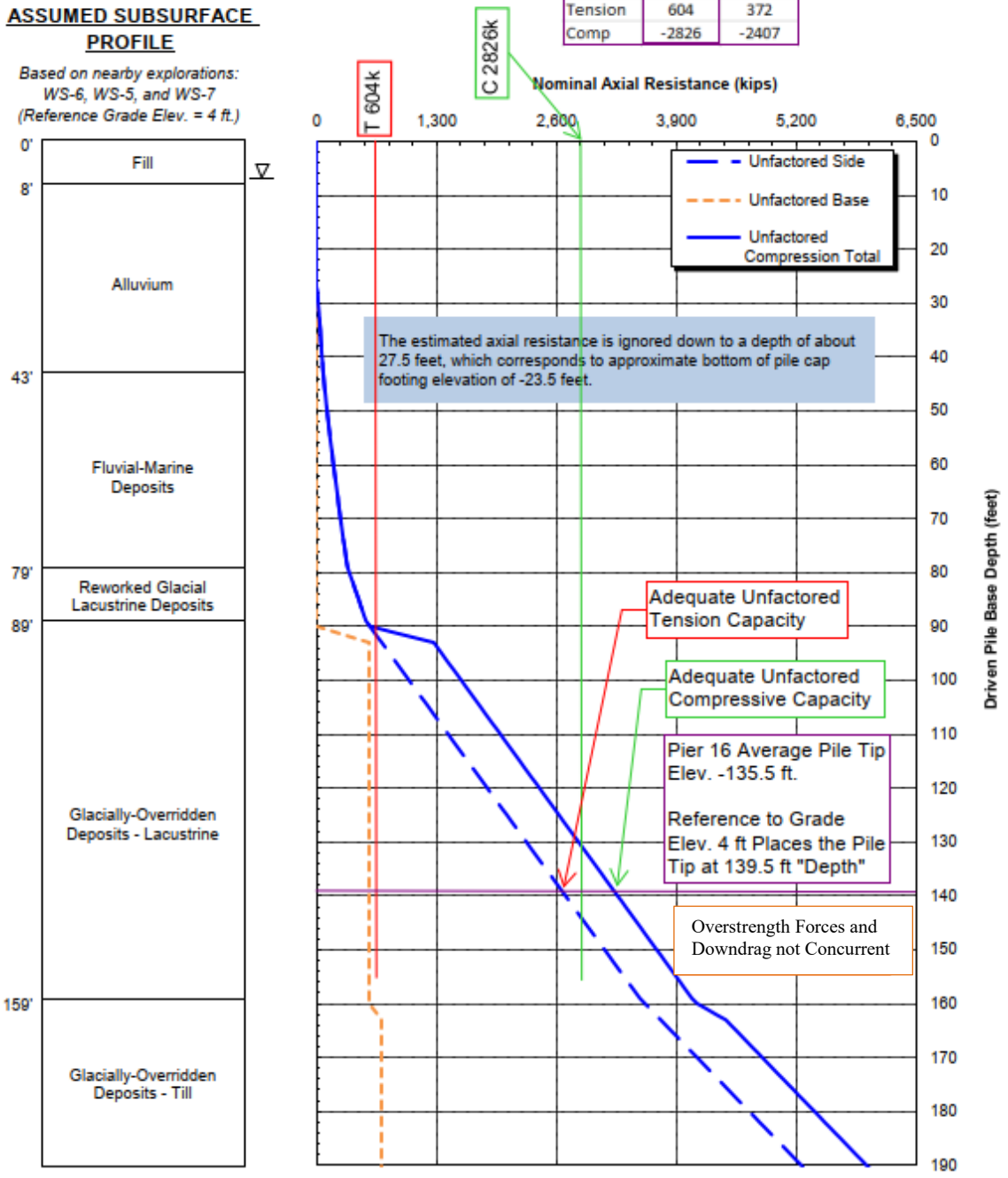
Fig_Pier 16_36in_Steel Pipe Pile_WS-6_WS-5_WS-7_09/16/2020 KDK

Envelope of Pile Axial Forces

	Pier 16	Pier 17
Tension	604	372
Comp	-2826	-2407

ASSUMED SUBSURFACE PROFILE

Based on nearby explorations: WS-6, WS-5, and WS-7 (Reference Grade Elev. = 4 ft.)



NOTES

1. The analyses were performed for a single pile based on guidelines included in AASHTO, and our experience.
2. The factored total shaft resistance is determined by adding the unfactored side and base resistances, multiplied by appropriate resistance factors per AASHTO.

West Seattle High Level Bridge
Seattle, Washington

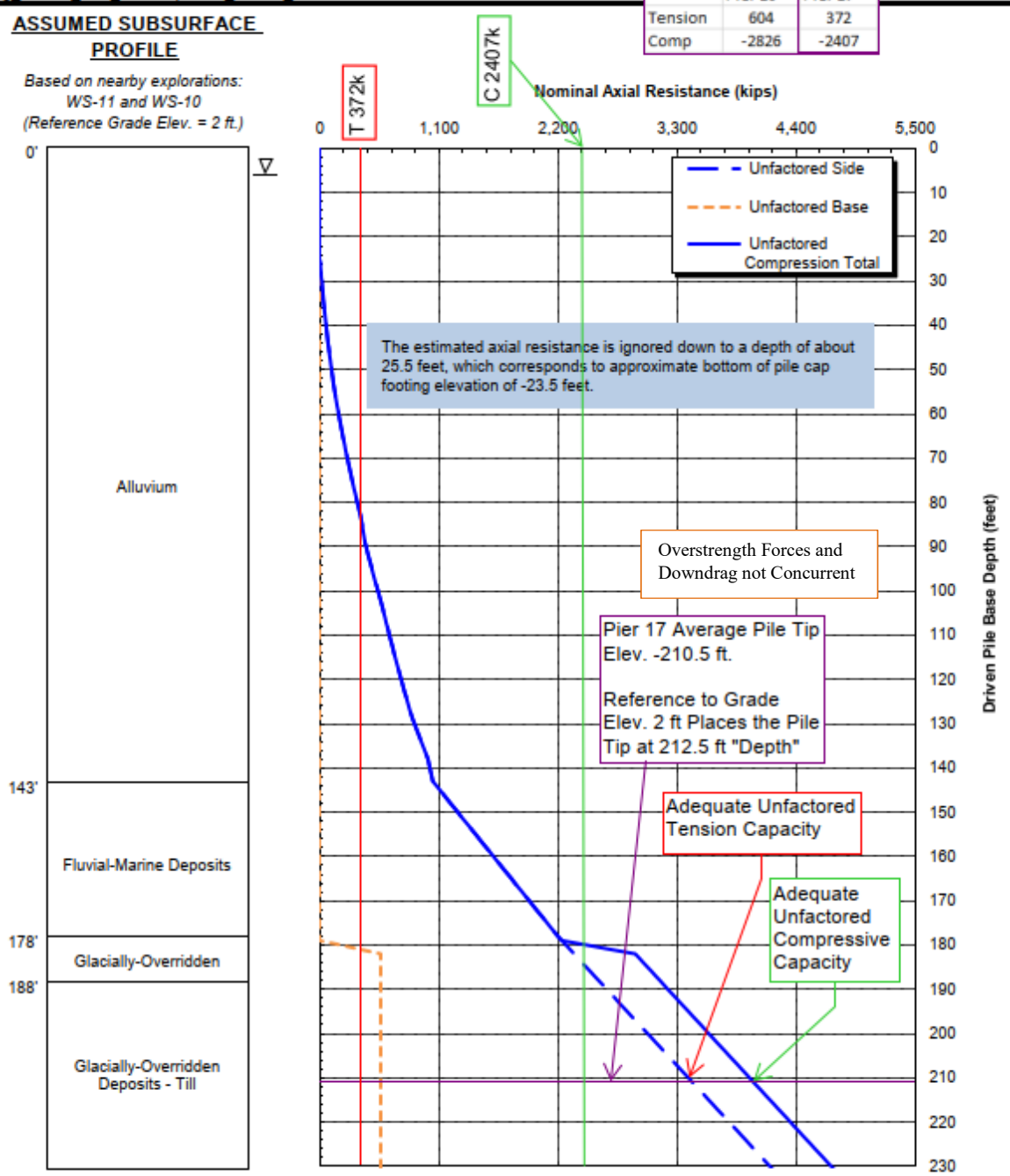
ESTIMATED AXIAL RESISTANCE OF 36-IN DRIVEN STEEL PIPE PILE PIER 16

September 2020 105492-001

SHANNON & WILSON FIG.

Fig_Pier 17_36in_Steel Pipe Pile_WS-11_WS-10 edits 09/16/2020 KDK

Envelope of Pile Axial Forces	Pier 16	Pier 17
Tension	604	372
Comp	-2826	-2407



NOTES

1. The analyses were performed for a single pile based on guidelines included in AASHTO, and our experience.
2. The factored total shaft resistance is determined by adding the unfactored side and base resistances, multiplied by appropriate resistance factors per AASHTO.

West Seattle High Level Bridge
Seattle, Washington

ESTIMATED AXIAL RESISTANCE OF 36-IN DRIVEN STEEL PIPE PILE PIER 17

September 2020 105492-001

SHANNON & WILSON FIG.

7.4.2 P16/17 Investigation of Pile Shear Forces from the Governing Results

The following evaluation confirms that the piles have an adequate shear resistance.

Envelope of Pile Shear Forcess (kip)		
Description	**Pier 16**	**Pier 17**
Pile Head Max	110	423
Entire Pile Max	209	424

Shear Resistance of the RCFST using WSDOT BDM Chapter 7.10 using Eq 7.10.2-15

$fyh = 45 \cdot ksi$ nominal steel yield stress

$As = 109.956 \cdot in^2$ area of the steel pile section

$Ac = 907.92 \cdot in^2$ area of the concrete core section

$f'c := 4ksi$ nominal concrete strength

$g_1 := 2.0$ coefficient for the shear capacity of the steel tube

$g_2 := 0.5$ coefficient for the effective shear area of the steel tube

$g_3 := 3.0$ coefficient for the effect on concrete strength in shear due to confinement from the steel tube

$g_{4.upper} := 1.0$ coefficient for bond development between the concrete and steel tube

$g_{4.lower} := 0.25$ lower bound value, since the result is sensitive to this parameter

$\phi := 1.0$ for extreme event evaluation

$$Vn_{upper} := g_{4.upper} \cdot \left[g_1 \cdot (0.6 \cdot fyh \cdot g_2 \cdot As) + 0.0316 \cdot g_3 \cdot Ac \cdot \sqrt{\frac{f'c}{ksi}} \cdot ksi \right] = 3140.947 \cdot kip$$

$$Vn_{lower} := g_{4.lower} \cdot \left[g_1 \cdot (0.6 \cdot fyh \cdot g_2 \cdot As) + 0.0316 \cdot g_3 \cdot Ac \cdot \sqrt{\frac{f'c}{ksi}} \cdot ksi \right] = 785.237 \cdot kip$$

The conservative lower bound value assuming a severe penalty for bond development between the concrete and steel tube exceeds the computed shear demands.

$0.6 \cdot fyh \cdot g_2 \cdot As = 1484.403 \cdot kip$

$\frac{Vn_{upper}}{2} = 1570.473 \cdot kip$

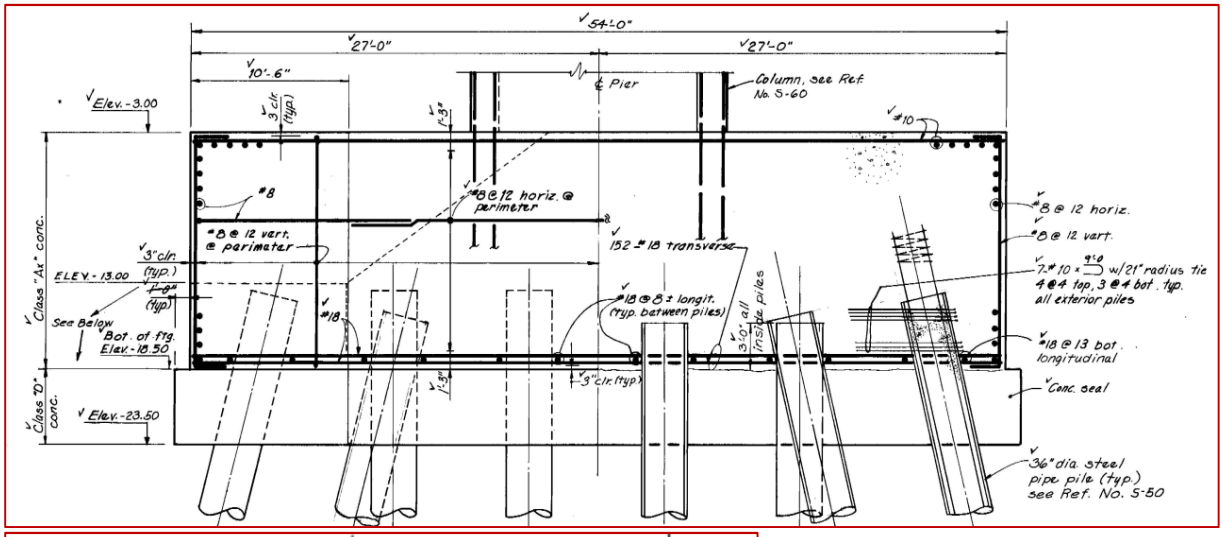
The pile itself provides a significant shear resistance, showing the conservatism in the g.4 parameter. Just the steel pile resistance would correspond roughly to the Equation 7.10.2-16 result with g.4 set to 0.5. No further refinement is necessary, as an adequate shear resistance is provided.

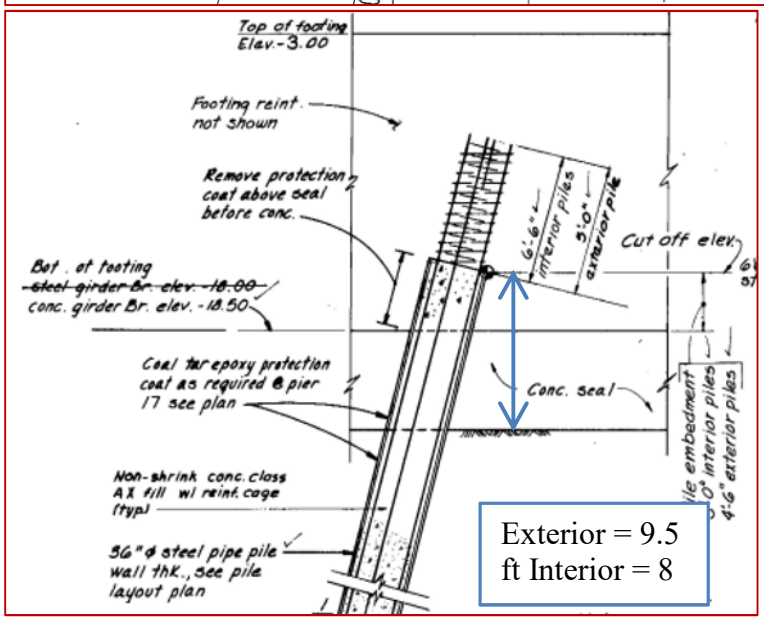
7.4.3 P16/17 Investigation of Pile Moments and P-M Interaction Behavior

The following represents the envelope of pile moments from the Overstrength Pushover and Lateral Soil Force Cases for Piers 16 and 17. Note that cases with tensile axial force require some participation of the steel shell for resistance to the applied moments, since the RC section plastic moment capacity is reduced when approaching axial force corresponding to tensile yielding of all reinforcement.

Envelope of Pile Moment Capacity/Demand Ratios (Mp/Mr)						
Description	Pier 16			Pier 17		
	Full Shell	Half Shell	No Shell	Full Shell	Half Shell	No Shell
Pile Head Min C/D	7.7	5.3	0.68	3.8	2.6	1.36
Entire Pile Min C/D	7.7	5.0	0.48	3.8	2.6	1.36

The following represents the embedment of the steel pile into the concrete seal and foundation.



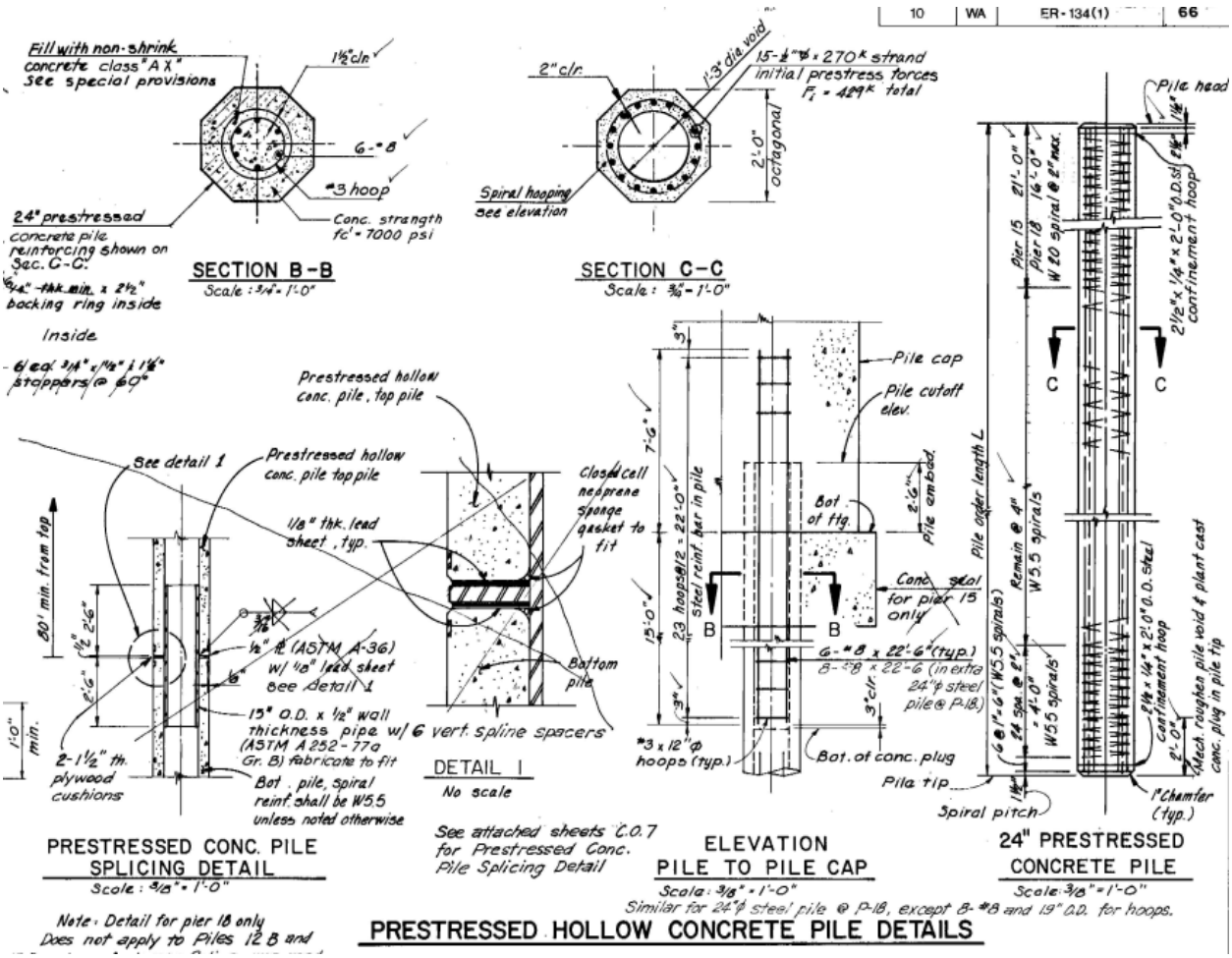


With minimal participation of the steel pile as flexural resistance, an adequate demand to capacity ratio is obtained. Note that the pile embedment will provide flexural resistance even if the pile is not developed for tensile axial forces.

for both Piers 16 and 17

7.5 PIER 15 AND 18 HOLLOW PRESTRESSED PILES AND DOWELLED CONN.

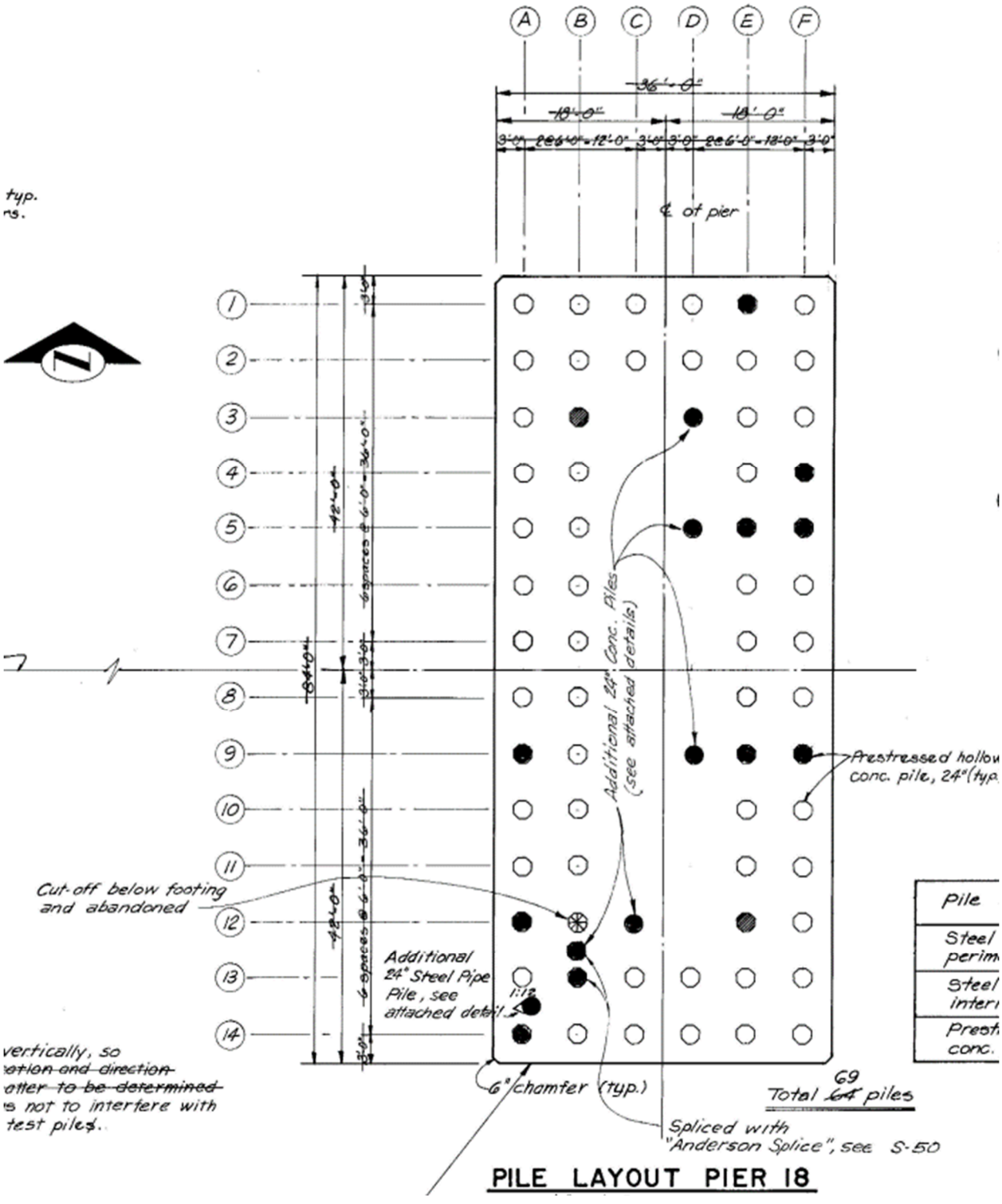
The following represents a typical hollow prestressed pile and dowelled head connection.



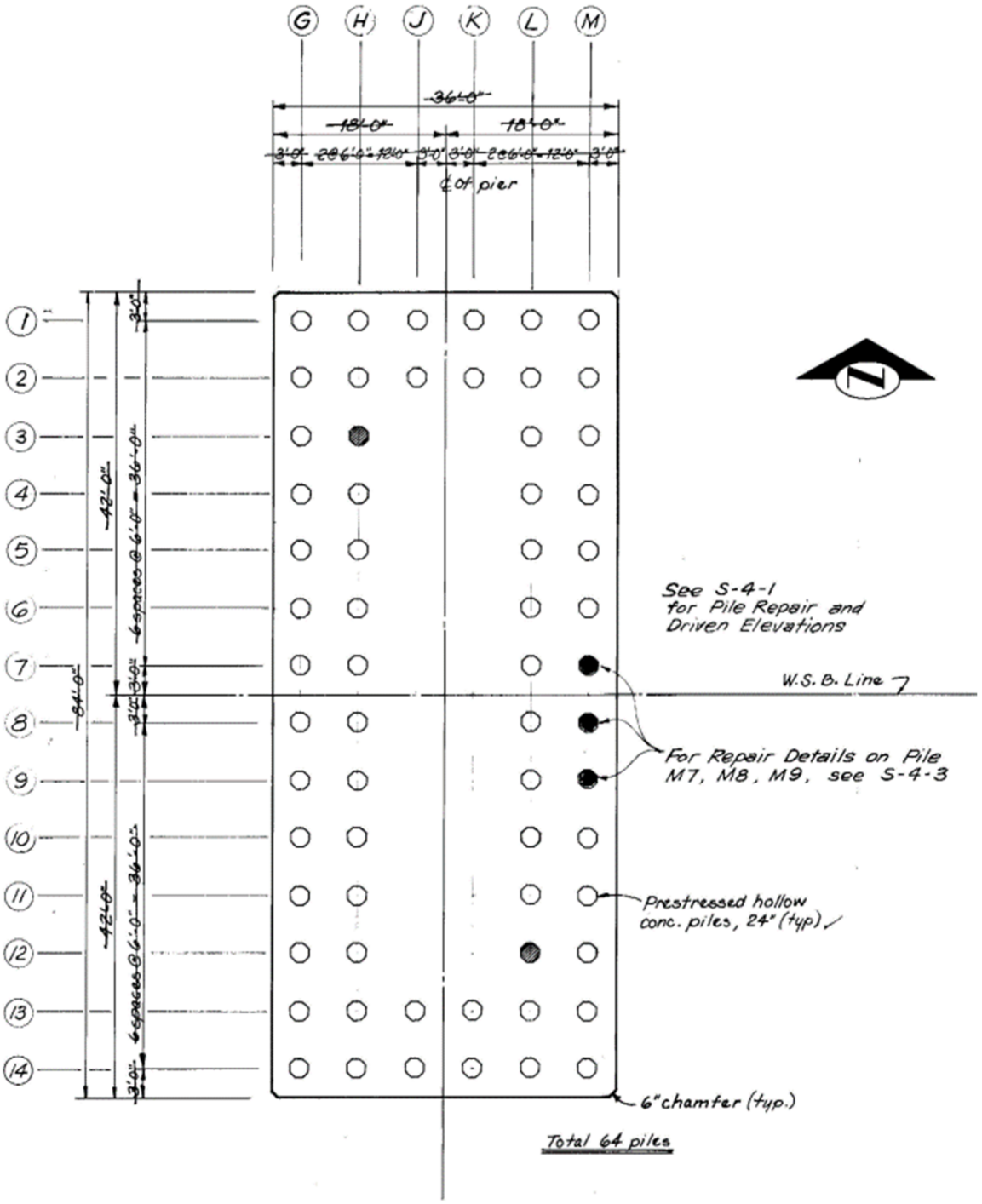
Pier 18 had multiple piles repaired (see diagram on the next page) using a driven 36"x0.75" pile that was noted to be concrete filled. Presumably, this pile was driven over the existing prestressed pile and integrated into the footing; however, the as-built plans do not provide a detail for the repaired piles.

Pile Repair						Pile Tip Elevation	
Concrete Fill Class AX	Reinf. Cage	Steel Pipe Collar / Extension				Min. Tip Elev.	Final Tip Elev.
		Diam.	Shell Thickness	Elevation Bot.	Elevation Top		
							143.72
							140.59
							141.41
Fill w/ Cl AX conc.		36"	3/4"	8.10	-2.50		140.58
							141.87
							141.16
							141.43
Fill w/ Cl AX conc.		36"	3/4"	-120.5	-2.50		142.86

Sample for Pier 18 pile repair noted in the as-built drawings. The driven 36"x0.75" repair pile is driven to considerable depth, concrete filled, and integrated into the footing based on the elevations shown. No changes to the pile head doweled connection are noted.



Tables provided in the as-built plans reference the pile repair, but do not show a sample detail.



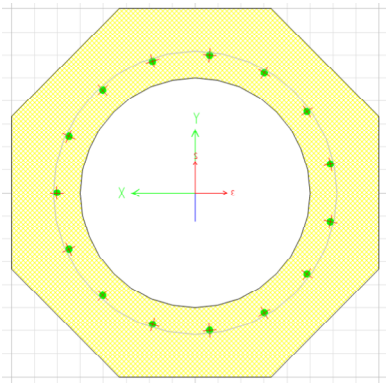
PILE LAYOUT PIER 15

Repairs for the Pier 15 piles were limited to concrete filling. The as-built plans did not note the inclusion of the 36”x0.75” piles driven over the damaged piles at Pier 15 like Pier 18.

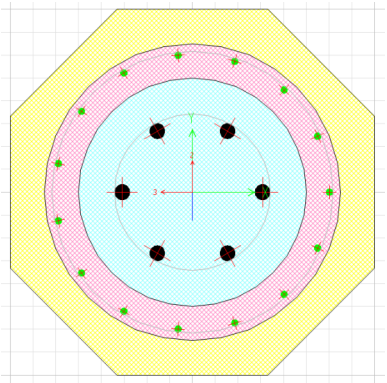
The following pile sections were evaluated using moment-curvature for the range of axial forces.

Pile	Sections
Typical	Typical Hollow Prestressed Pile
	Typical Prestressed Pile with Dowels
	Typical Pile Connection with Dowels
Repair	Repaired Prestress Pile with Full Steel Shell
	Repair Pile Connection with 25% Shell Participation
	Repair Pile Connection with No Shell Participation

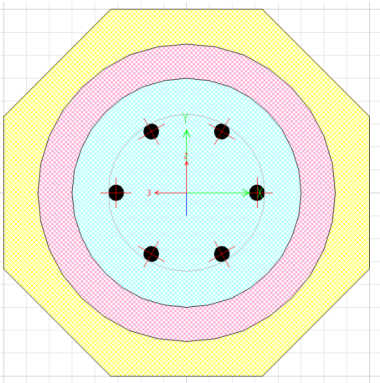
The pile concrete and void fill concrete have different strength; therefore, the following figures show the unique regions considered for confinement properties. Note that no confinement was considered for the typical hollow pile. Prestress losses were calculated; however, the pile head moments govern where only the dowels are effective for flexural and axial resistance.



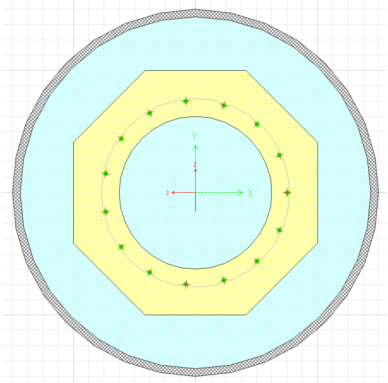
Typ. Hollow PS Pile



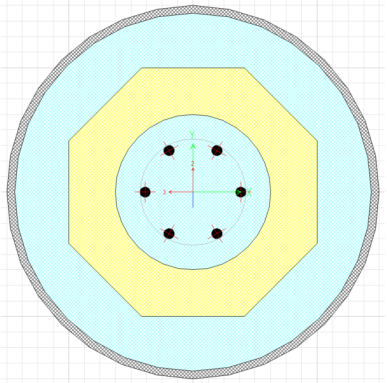
PS Pile with Dowels



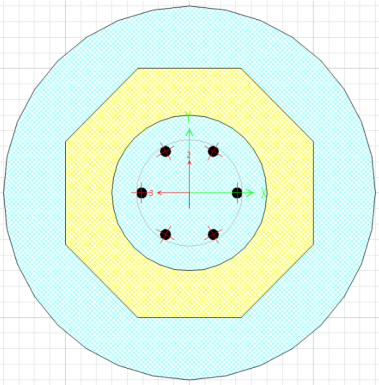
Typ. Conn. with Dowels



Typ. Repair Pile Full Shell

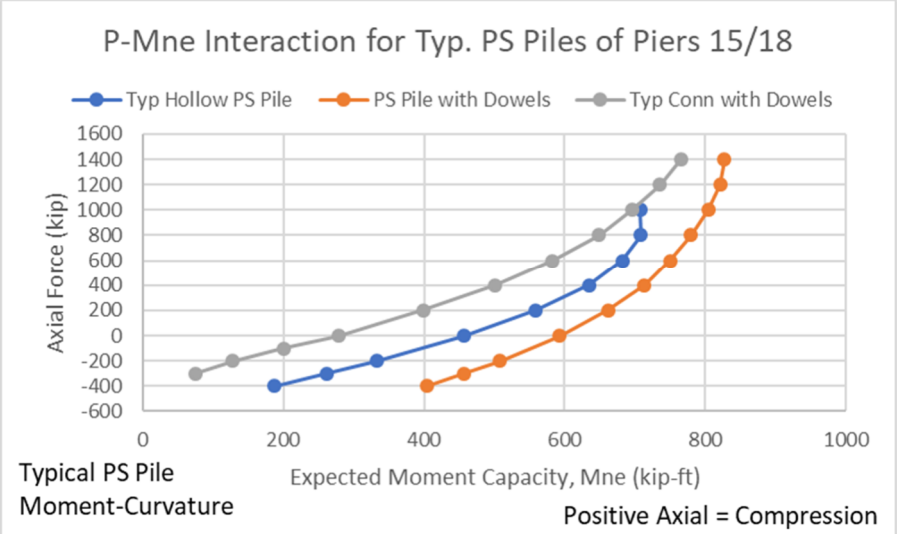


Repair Pile Conn. 25% Shell

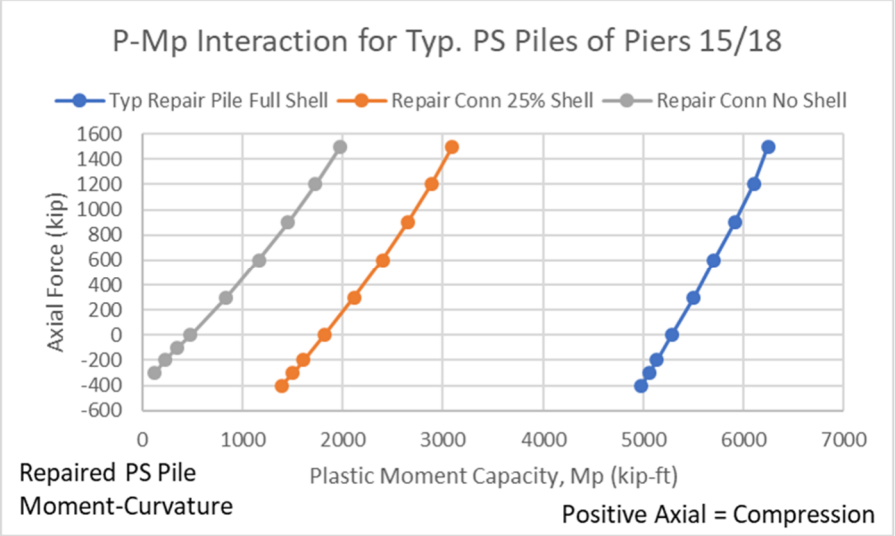


Repair Pile Conn. No Shell

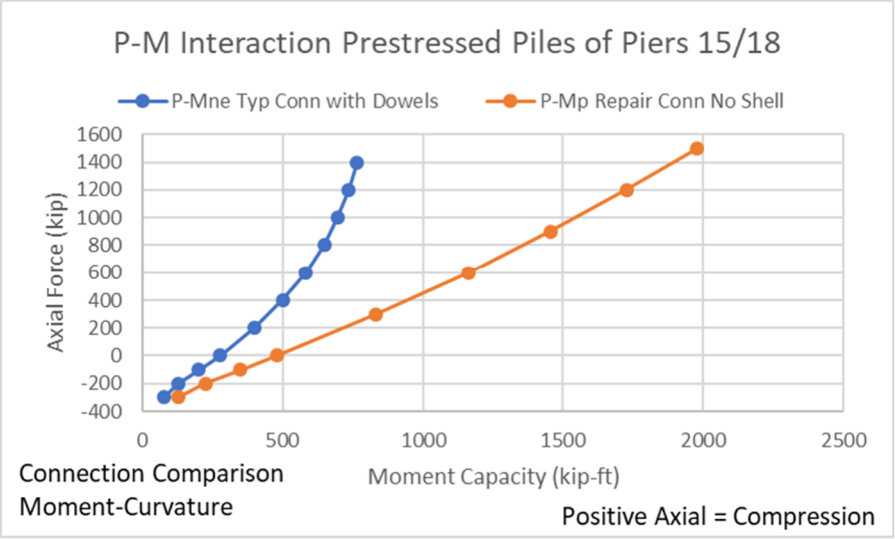
The following axial load versus moment interaction relationships were created for assessment.



Typical Prestressed Pile



Repaired PS Pile



Pile Head Connections

Concrete Filled Steel Tube Pile Properties using WSDOT BDM Section 7.10.2

$Es := 29000ksi$

$f'ce := 1.3 \cdot (4ksi) = 5.2 \cdot ksi$

$Ece := 120000 \cdot (1.0) \cdot (0.155)^{2.0} \cdot (5.2)^{0.33} \cdot ksi = 4967.361 \cdot ksi$

$do := 36in \qquad t := 0.75in \qquad di := do - 2 \cdot t = 34.5 \cdot in$

$As := \frac{\pi \cdot do^2}{4} - \frac{\pi \cdot di^2}{4} = 83.056 \cdot in^2 \qquad Is := \frac{\pi \cdot do^4}{64} - \frac{\pi \cdot di^4}{64} = 12906.101 \cdot in^4$

$Ac := \frac{\pi \cdot di^2}{4} = 934.82 \cdot in^2 \qquad Ic := \frac{\pi \cdot di^4}{64} = 69541.856 \cdot in^4$

The piles have a large range of axial forces, set P/Po using the dead load axial.

$P := 600kip$ approximate

For now, assume the pipe piles are A252 Gr3 with fy = 45 ksi.

$fye := 1.1 \cdot (45ksi) = 49.5 \cdot ksi$

$Po := 0.95 \cdot f'ce \cdot Ac + fye \cdot As = 8729.276 \cdot kip$

$C' := 0.15 + \frac{P}{Po} + \frac{As}{As + Ac} = 0.3$ which is less than the upper bound of 0.9

$EI_{eff} := Es \cdot Is + C' \cdot Ece \cdot Ic = 3.32 \times 10^6 \cdot kip \cdot ft^2$

The repaired piles were modeled with a 36" diameter concrete section at Ece.

$EI_{conc} := Ece \cdot \frac{\pi \cdot do^4}{64} = 2.844 \times 10^6 \cdot kip \cdot ft^2$ value initially included in the model

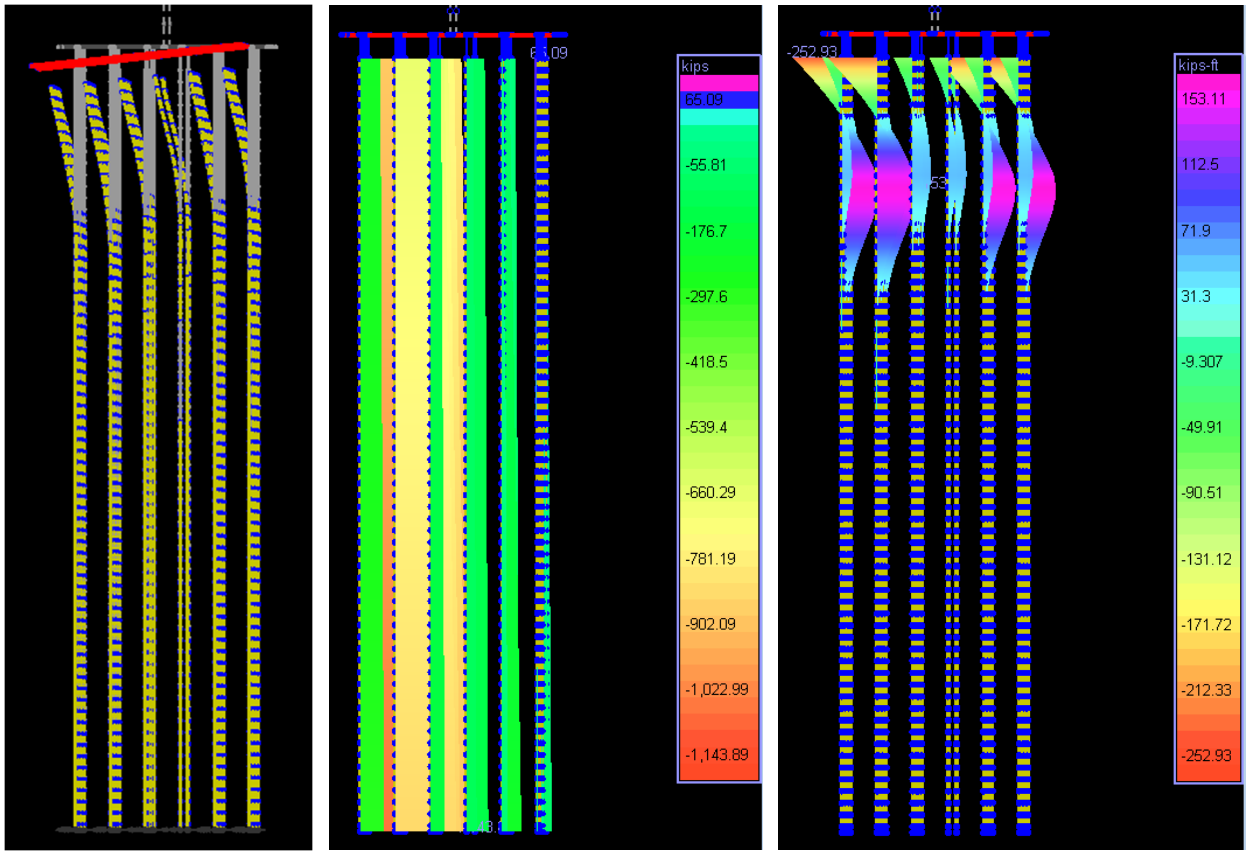
$I_{ratio} := \frac{EI_{eff}}{EI_{conc}} = 1.167$

The computed I.ratio and A.ratio represent property modifiers on the 36" concrete section included in the model to represent the effect of the steel shell.

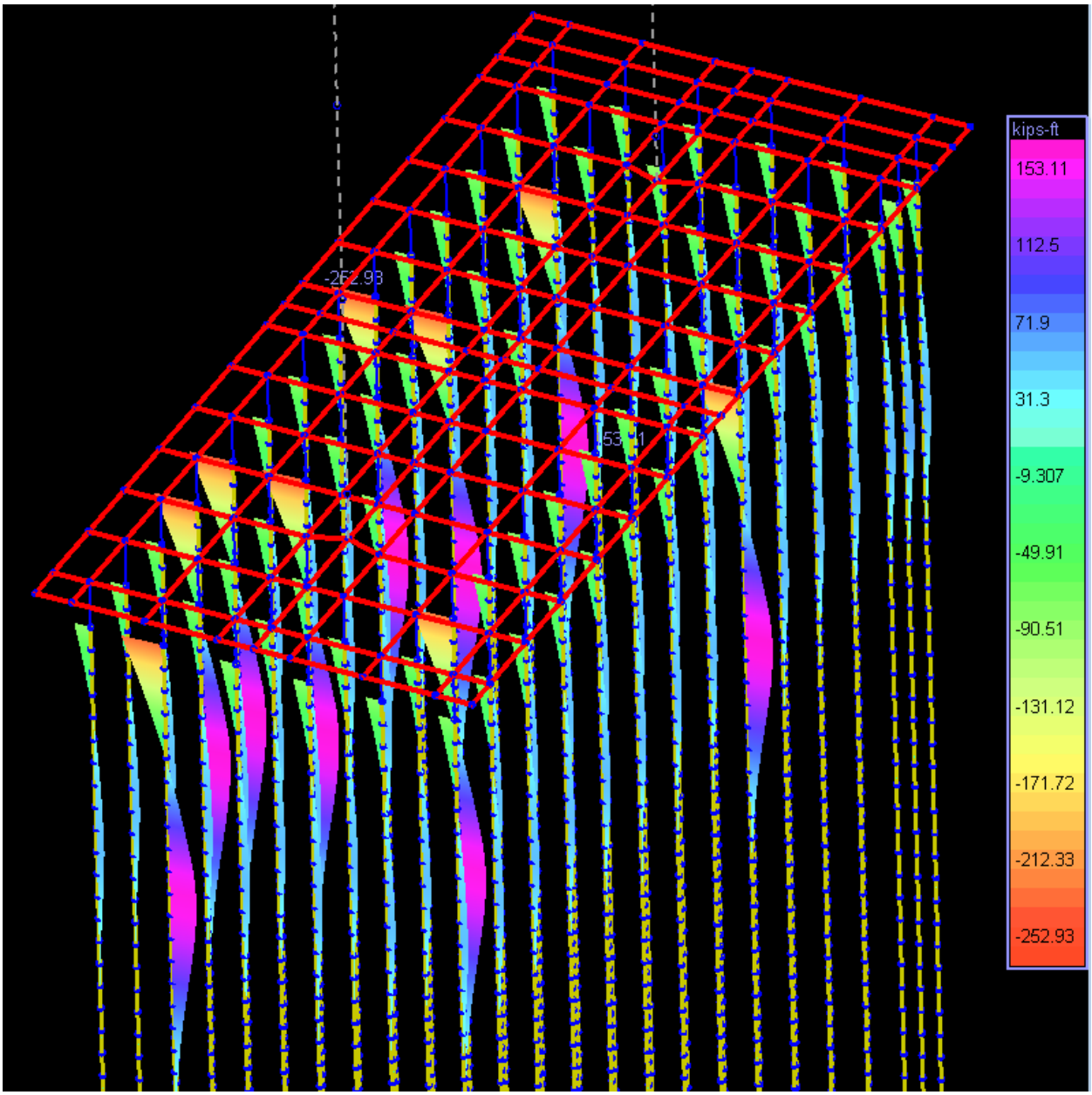
$A_{ratio} := \frac{As \cdot \frac{Es}{Ece} + Ac}{\frac{\pi \cdot do^2}{4}} = 1.395$

Note that this property modifier was not included within the current iteration of the model. Exceedance of the moment/axial capacity of the pile head would only be made worse upon inclusion of this property modifier.

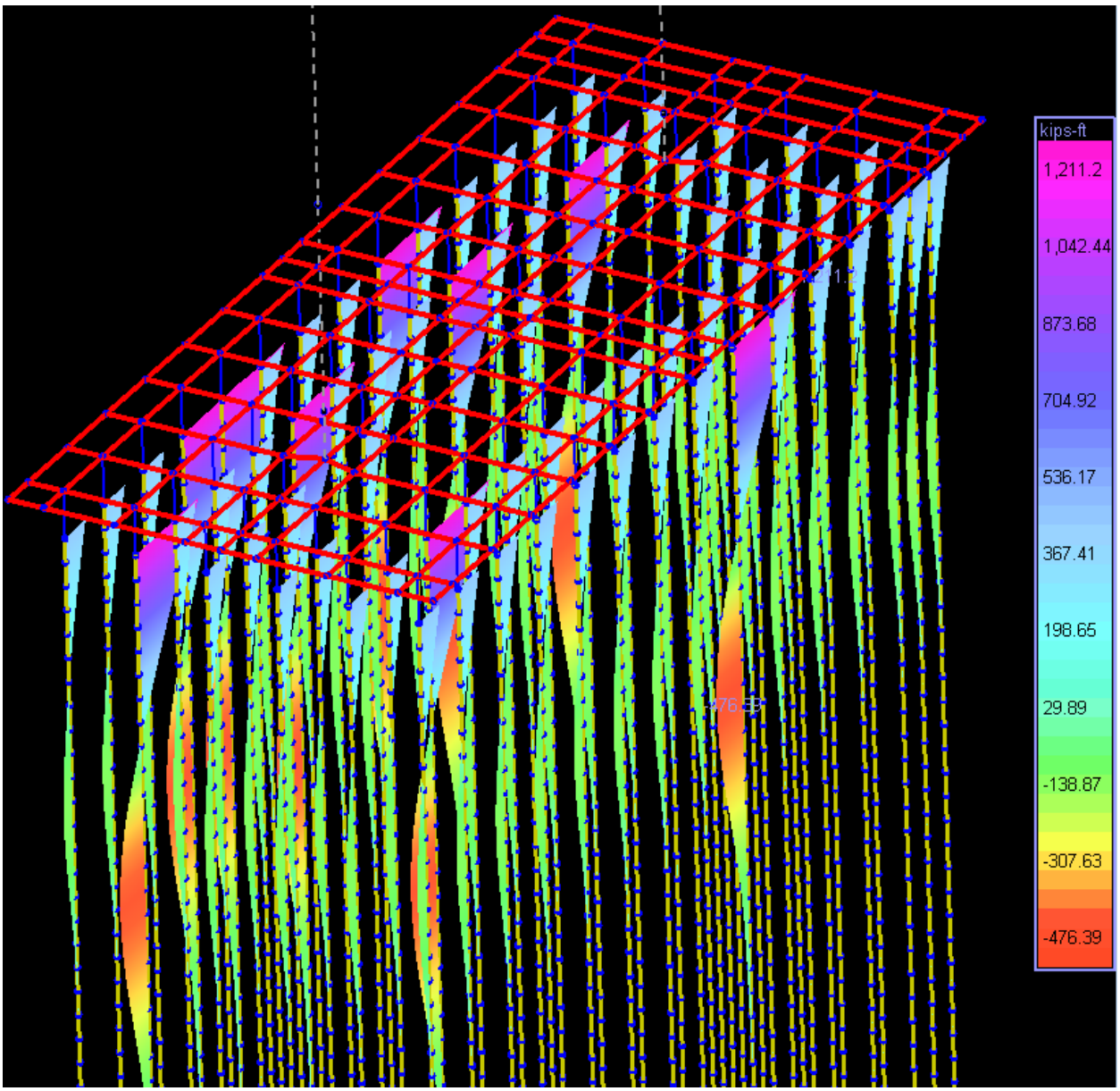
In terms of the observed response, the use of group factor reductions for soil lateral strengths significantly influenced the behavior. The use of liquefied versus non-liquefied soil properties was found to have a larger effect at Pier 18. The following general figures show the longitudinal behavior of Pier 18.



The unequal distribution of load is due to the significant increased stiffness of both the repair piles themselves and the mobilized soil reaction (36” versus 24” diameter). The following figures show the significant difference in pile moments under the longitudinal and transverse overstrength pushover effects.



Pier 18 longitudinal overstrength pushover, with repair piles taking more load. As previously discussed, there is a smaller group factor reduction for soil lateral stiffness in the longitudinal direction.



Pier 18 transverse overstrength pushover, with repair piles taking more load. As previously discussed, there is a larger group factor reduction for soil lateral stiffness in the transverse direction.

7.5.1 P15/18 Investigation for Pile Head Moments

The following table summarizes the results, with exceedances noted in yellow highlights.

Pile Foundation Assessment for Longitudinal / Transverse Flexural Overstrength Demands							
Typ. Dowel	Ast (fye)	322	kip	Governing Pile Head Connection			
Conn. 6 - #8	Ast (fue)	450	kip	Axial Force (kip)		P-Mp Relation	Moment
Pier	Pushover	Direction	Liquefied?	Max	Min	for Connection	Mp / Mr
15	Long.	East & West	Liquefied	-16	-698	Typ.	5.58
			Non-Liq	-16	-697	Typ.	6.06
15	Trans.	North & South	Liquefied	97	-578	Typ.	0.94
			Non-Liq	95	-576	Typ.	1.06
18	Long.	East	Liquefied	41	-324	Typ.	2.68
				65	-967	Repair 36"	1.64
			Non-Liq	32	-316	Typ.	7.14
				40	-945	Repair 36"	34.02
18	Long.	West	Liquefied	39	-372	Typ.	2.69
				118	-1180	Repair 36"	1.50
			Non-Liq	32	-363	Typ.	8.59
				94	-1153	Repair 36"	12.90
18	Trans.	North	Liquefied	142	-452	Typ.	0.45
				172	-1311	Repair 36"	0.22
			Non-Liq	127	-438	Typ.	1.00
				138	-1266	Repair 36"	0.72
18	Trans.	South	Liquefied	116	-448	Typ.	0.50
				388	-1158	Repair 36"	0.00
			Non-Liq	103	-434	Typ.	1.09
				343	-1124	Repair 36"	0.08

Pier 18 transverse overstrength pushover toward the south places the repair pile doweled connections under tensile axial forces that exceed their expected yield strength, significantly reducing any moment that can be resisted. Capacity to demand ratios (Mp / Mr) for the pile head moments are exceeded in both the typical and repair piles of Pier 18 using both liquefied and non-liquefied properties. Consideration for a higher pile head capacity using partial flexural participation of the steel shell does not address the high tensile axial forces or the fact that the typical (unrepaired) piles still have exceedances. Redistribution of forces will only increase the demands on the typical (unrepaired) piles.

The following detailed results show the number of piles with moment exceedances at the pile heads for the governing Pier 18 transverse overstrength pushover to the south.

Pier 18 Pile Head Load Effects		Pier 18 Mpo Transverse Pushover (South) with Liquefied Soil					
		(kip)	(kip)	(kip-ft)	P-M Connection	(kip-ft)	Moment
Pile	Member	Axial	Shear	Moment	Relationship	Mp	Mp/Mr
A1	70100-0	109	39	402	Typ Dowel Conn	203	0.50
A2	70200-0	68	39	403	Typ Dowel Conn	231	0.57
A3	70300-0	27	39	403	Typ Dowel Conn	257	0.64
A4	70400-0	-13	39	403	Typ Dowel Conn	283	0.70
A5	70500-0	-51	39	404	Typ Dowel Conn	307	0.76
A6	70600-0	-90	39	404	Typ Dowel Conn	330	0.82
A7	70700-0	-128	39	404	Typ Dowel Conn	352	0.87
A8	70800-0	-167	39	403	Typ Dowel Conn	375	0.93
A9	70900-0	-698	86	1203	Repair Conn No Shell	1257	1.04
A10	71000-0	-251	39	400	Typ Dowel Conn	421	1.05
A11	71100-0	-297	39	399	Typ Dowel Conn	445	1.12
A12	71200-0	-345	39	398	Typ Dowel Conn	469	1.18
A13	71300-0	-393	39	397	Typ Dowel Conn	492	1.24
A13-14	71400-0	-661	42	439	Unique Capacity	604	1.38
A14	71500-0	-442	39	397	Typ Dowel Conn	515	1.30
B1	71600-0	111	39	402	Typ Dowel Conn	202	0.50
B2	71700-0	69	39	402	Typ Dowel Conn	229	0.57
B3	71800-0	95	86	1206	Repair Conn No Shell	366	0.30
B4	71900-0	-12	39	403	Typ Dowel Conn	282	0.70
B5	72000-0	-51	39	403	Typ Dowel Conn	306	0.76
B6	72100-0	-89	39	403	Typ Dowel Conn	329	0.82
B7	72200-0	-127	39	403	Typ Dowel Conn	352	0.87
B8	72300-0	-166	39	402	Typ Dowel Conn	374	0.93
B9	72400-0	-208	39	401	Typ Dowel Conn	397	0.99
B10	72500-0	-252	39	399	Typ Dowel Conn	421	1.06
B11	72600-0	-298	39	397	Typ Dowel Conn	446	1.12
B12	72700-0	-347	39	397	Typ Dowel Conn	470	1.18
B13	72800-0	-396	39	396	Typ Dowel Conn	494	1.25
B14	72900-0	-445	39	396	Typ Dowel Conn	516	1.30
C1	73000-0	113	39	401	Typ Dowel Conn	200	0.50
C2	73100-0	72	39	401	Typ Dowel Conn	228	0.57
C12	73200-0	-350	39	395	Typ Dowel Conn	471	1.19
C13	73300-0	-399	39	396	Typ Dowel Conn	495	1.25
C14	73400-0	-448	39	396	Typ Dowel Conn	517	1.31
D1	73500-0	114	39	400	Typ Dowel Conn	199	0.50
D2	73600-0	73	39	400	Typ Dowel Conn	227	0.57
D3	73700-0	31	39	400	Typ Dowel Conn	254	0.64
D5	73800-0	-48	39	402	Typ Dowel Conn	305	0.76
D9	73900-0	-206	39	399	Typ Dowel Conn	396	0.99
D13	74000-0	-399	39	395	Typ Dowel Conn	495	1.25
D14	74100-0	-447	39	395	Typ Dowel Conn	517	1.31
E1	74200-0	388	86	1199	Repair Conn No Shell	0	0.00
E2	74300-0	74	39	400	Typ Dowel Conn	226	0.57
E3	74400-0	33	39	400	Typ Dowel Conn	253	0.63
E4	74500-0	-7	39	401	Typ Dowel Conn	279	0.70
E5	74600-0	-154	86	1205	Repair Conn No Shell	665	0.55
E6	74700-0	-84	39	401	Typ Dowel Conn	326	0.81
E7	74800-0	-122	39	401	Typ Dowel Conn	349	0.87
E8	74900-0	-162	39	400	Typ Dowel Conn	372	0.93
E9	75000-0	-683	85	1194	Repair Conn No Shell	1242	1.04
E10	75100-0	-248	39	397	Typ Dowel Conn	419	1.06
E11	75200-0	-295	39	395	Typ Dowel Conn	444	1.12
E12	75300-0	-1158	85	1178	Repair Conn No Shell	1692	1.44
E13	75400-0	-395	39	394	Typ Dowel Conn	493	1.25
E14	75500-0	-445	39	394	Typ Dowel Conn	516	1.31
F1	75600-0	116	39	399	Typ Dowel Conn	198	0.50
F2	75700-0	76	39	400	Typ Dowel Conn	225	0.56
F3	75800-0	35	39	400	Typ Dowel Conn	252	0.63
F4	75900-0	-15	86	1201	Repair Conn No Shell	500	0.42
F5	76000-0	-145	86	1203	Repair Conn No Shell	655	0.54
F6	76100-0	-81	39	401	Typ Dowel Conn	325	0.81
F7	76200-0	-120	39	400	Typ Dowel Conn	348	0.87
F8	76300-0	-159	39	399	Typ Dowel Conn	370	0.93
F9	76400-0	-675	85	1192	Repair Conn No Shell	1233	1.03
F10	76500-0	-245	39	396	Typ Dowel Conn	418	1.05
F11	76600-0	-292	39	395	Typ Dowel Conn	442	1.12
F12	76700-0	-341	39	394	Typ Dowel Conn	467	1.19
F13	76800-0	-391	39	393	Typ Dowel Conn	491	1.25
F14	76900-0	-441	39	393	Typ Dowel Conn	514	1.31
		P.max	P.min		P-M Relationship	Minimum	Mp/Mr
		116	-448		Typ Dowel Conn		0.50
		388	-1158		Repair Conn No Shell		0.00

Results with Liquefied Soil

Pier 18 Pile Head Load Effects		Pier 18 Mpo Transverse Pushover (South) with Liquefied Soil (kip)	(kip)	(kip-ft)	P-M Connection	(kip-ft)	Moment
Pile	Member	Axial	Shear	Moment	Relationship	Mp	Mp/Mr
A1	70100-0	94	42	192	Typ Dowel Conn	213	1.11
A2	70200-0	56	42	193	Typ Dowel Conn	238	1.24
A3	70300-0	17	42	193	Typ Dowel Conn	263	1.36
A4	70400-0	-20	43	194	Typ Dowel Conn	287	1.48
A5	70500-0	-57	43	195	Typ Dowel Conn	310	1.59
A6	70600-0	-93	43	196	Typ Dowel Conn	332	1.70
A7	70700-0	-129	43	195	Typ Dowel Conn	353	1.81
A8	70800-0	-166	42	193	Typ Dowel Conn	374	1.94
A9	70900-0	-690	69	428	Repair Conn No Shell	1249	2.92
A10	71000-0	-247	42	187	Typ Dowel Conn	419	2.24
A11	71100-0	-291	41	184	Typ Dowel Conn	442	2.40
A12	71200-0	-336	41	182	Typ Dowel Conn	465	2.55
A13	71300-0	-383	41	181	Typ Dowel Conn	487	2.68
A13-14	71400-0	-642	43	196	Unique Capacity	597	3.05
A14	71500-0	-429	41	181	Typ Dowel Conn	509	2.81
B1	71600-0	97	42	191	Typ Dowel Conn	211	1.10
B2	71700-0	58	42	192	Typ Dowel Conn	237	1.24
B3	71800-0	64	69	436	Repair Conn No Shell	405	0.93
B4	71900-0	-19	42	194	Typ Dowel Conn	286	1.48
B5	72000-0	-56	43	195	Typ Dowel Conn	309	1.59
B6	72100-0	-92	43	195	Typ Dowel Conn	331	1.70
B7	72200-0	-128	43	194	Typ Dowel Conn	352	1.82
B8	72300-0	-166	42	192	Typ Dowel Conn	374	1.95
B9	72400-0	-205	42	189	Typ Dowel Conn	396	2.09
B10	72500-0	-247	41	186	Typ Dowel Conn	419	2.26
B11	72600-0	-292	41	183	Typ Dowel Conn	442	2.42
B12	72700-0	-338	41	181	Typ Dowel Conn	466	2.57
B13	72800-0	-385	41	181	Typ Dowel Conn	488	2.70
B14	72900-0	-432	41	180	Typ Dowel Conn	510	2.83
C1	73000-0	99	42	191	Typ Dowel Conn	210	1.10
C2	73100-0	60	42	191	Typ Dowel Conn	235	1.24
C12	73200-0	-340	40	179	Typ Dowel Conn	467	2.60
C13	73300-0	-387	41	180	Typ Dowel Conn	489	2.72
C14	73400-0	-434	41	180	Typ Dowel Conn	511	2.84
D1	73500-0	101	42	191	Typ Dowel Conn	208	1.09
D2	73600-0	62	42	190	Typ Dowel Conn	234	1.23
D3	73700-0	23	42	191	Typ Dowel Conn	260	1.36
D5	73800-0	-53	42	194	Typ Dowel Conn	307	1.59
D9	73900-0	-202	42	188	Typ Dowel Conn	394	2.10
D13	74000-0	-386	40	179	Typ Dowel Conn	489	2.73
D14	74100-0	-433	40	179	Typ Dowel Conn	511	2.85
E1	74200-0	343	69	431	Repair Conn No Shell	33	0.08
E2	74300-0	63	42	191	Typ Dowel Conn	234	1.22
E3	74400-0	25	42	191	Typ Dowel Conn	259	1.35
E4	74500-0	-13	42	192	Typ Dowel Conn	283	1.47
E5	74600-0	-168	70	441	Repair Conn No Shell	681	1.54
E6	74700-0	-86	42	194	Typ Dowel Conn	328	1.69
E7	74800-0	-122	42	193	Typ Dowel Conn	349	1.81
E8	74900-0	-160	42	190	Typ Dowel Conn	371	1.95
E9	75000-0	-670	68	421	Repair Conn No Shell	1229	2.92
E10	75100-0	-242	41	183	Typ Dowel Conn	416	2.27
E11	75200-0	-287	41	180	Typ Dowel Conn	440	2.44
E12	75300-0	-1124	65	393	Repair Conn No Shell	1661	4.23
E13	75400-0	-382	40	178	Typ Dowel Conn	487	2.73
E14	75500-0	-430	40	178	Typ Dowel Conn	509	2.86
F1	75600-0	103	42	190	Typ Dowel Conn	207	1.09
F2	75700-0	65	42	191	Typ Dowel Conn	233	1.22
F3	75800-0	27	42	191	Typ Dowel Conn	257	1.34
F4	75900-0	-35	69	438	Repair Conn No Shell	525	1.20
F5	76000-0	-158	70	440	Repair Conn No Shell	670	1.52
F6	76100-0	-83	42	193	Typ Dowel Conn	326	1.69
F7	76200-0	-120	42	192	Typ Dowel Conn	347	1.81
F8	76300-0	-157	42	190	Typ Dowel Conn	369	1.94
F9	76400-0	-660	68	420	Repair Conn No Shell	1218	2.90
F10	76500-0	-239	41	183	Typ Dowel Conn	414	2.26
F11	76600-0	-284	41	180	Typ Dowel Conn	438	2.43
F12	76700-0	-330	40	179	Typ Dowel Conn	462	2.59
F13	76800-0	-378	40	178	Typ Dowel Conn	485	2.73
F14	76900-0	-426	40	177	Typ Dowel Conn	507	2.86
		P.max	P.min		P-M Relationship	Minimum	Mp/Mr
		103	-434		Typ Dowel Conn		1.09
		343	-1124		Repair Conn No Shell		0.08

Results with Non-Liquefied Soil

Minor exceedances for the Pier 15 piles under liquefied conditions are shown in the following table. The magnitude and limited number of piles with exceedances indicate the foundation has the capability to redistribute the excess demands to adjacent piles as pile head hinges form.

Pier 15 Pile Head Load Effects		Pier 15 Mpo Transverse Pushover (North) with Liquefied Soil					
		(kip)	(kip)	(kip-ft)	P-M Connection	(kip-ft)	Moment
Pile	Member	Axial	Shear	Moment	Relationship	Mp	Mp/Mr
G1	30100-0	-578	46	224	Typ Dowel Conn	572	2.55
H1	30200-0	-575	46	224	Typ Dowel Conn	571	2.55
J1	30300-0	-573	46	224	Typ Dowel Conn	570	2.54
K1	30400-0	-569	46	224	Typ Dowel Conn	569	2.54
L1	30500-0	-565	46	224	Typ Dowel Conn	567	2.53
M1	30600-0	-560	46	224	Typ Dowel Conn	565	2.52
G2	30700-0	-525	46	224	Typ Dowel Conn	551	2.46
H2	30800-0	-523	46	224	Typ Dowel Conn	550	2.45
J2	30900-0	-521	46	224	Typ Dowel Conn	549	2.45
K2	31000-0	-517	46	224	Typ Dowel Conn	548	2.44
L2	31100-0	-512	46	224	Typ Dowel Conn	545	2.43
M2	31200-0	-507	46	224	Typ Dowel Conn	543	2.42
G3	31300-0	-473	46	224	Typ Dowel Conn	528	2.35
H3	31400-0	-470	46	224	Typ Dowel Conn	527	2.35
L3	31500-0	-460	46	224	Typ Dowel Conn	523	2.33
M3	31600-0	-455	46	224	Typ Dowel Conn	520	2.32
G4	31700-0	-420	46	225	Typ Dowel Conn	505	2.25
H4	31800-0	-418	46	224	Typ Dowel Conn	504	2.24
L4	31900-0	-407	46	224	Typ Dowel Conn	499	2.22
M4	32000-0	-402	46	224	Typ Dowel Conn	497	2.21
G5	32100-0	-369	46	225	Typ Dowel Conn	481	2.14
H5	32200-0	-366	46	225	Typ Dowel Conn	479	2.13
L5	32300-0	-355	46	225	Typ Dowel Conn	474	2.11
M5	32400-0	-351	46	225	Typ Dowel Conn	472	2.10
G6	32500-0	-318	46	225	Typ Dowel Conn	456	2.02
H6	32600-0	-315	46	225	Typ Dowel Conn	454	2.02
L6	32700-0	-304	46	225	Typ Dowel Conn	448	1.99
M6	32800-0	-300	46	225	Typ Dowel Conn	446	1.98
G7	32900-0	-268	46	225	Typ Dowel Conn	430	1.91
H7	33000-0	-265	46	225	Typ Dowel Conn	428	1.90
L7	33100-0	-254	46	225	Typ Dowel Conn	422	1.88
M7	33200-0	-395	48	245	Typ Dowel Conn	493	2.02
G8	33300-0	-219	46	225	Typ Dowel Conn	403	1.79
H8	33400-0	-215	46	225	Typ Dowel Conn	402	1.78
L8	33500-0	-204	46	225	Typ Dowel Conn	396	1.76
M8	33600-0	-317	48	245	Typ Dowel Conn	455	1.86
G9	33700-0	-170	46	226	Typ Dowel Conn	376	1.67
H9	33800-0	-166	46	225	Typ Dowel Conn	374	1.66
L9	33900-0	-155	46	225	Typ Dowel Conn	368	1.63
M9	34000-0	-239	48	245	Typ Dowel Conn	415	1.69
G10	34100-0	-120	46	225	Typ Dowel Conn	348	1.54
H10	34200-0	-117	46	225	Typ Dowel Conn	346	1.53
L10	34300-0	-106	46	225	Typ Dowel Conn	340	1.51
M10	34400-0	-102	46	225	Typ Dowel Conn	337	1.50
G11	34500-0	-71	46	225	Typ Dowel Conn	319	1.41
H11	34600-0	-68	46	225	Typ Dowel Conn	317	1.40
L11	34700-0	-57	46	225	Typ Dowel Conn	310	1.38
M11	34800-0	-53	46	225	Typ Dowel Conn	308	1.36
G12	34900-0	-22	46	225	Typ Dowel Conn	288	1.28
H12	35000-0	-18	46	225	Typ Dowel Conn	286	1.27
L12	35100-0	-7	46	225	Typ Dowel Conn	279	1.24
M12	35200-0	-3	46	225	Typ Dowel Conn	277	1.23
G13	35300-0	29	46	225	Typ Dowel Conn	256	1.14
H13	35400-0	32	46	225	Typ Dowel Conn	254	1.13
L13	35500-0	43	46	225	Typ Dowel Conn	247	1.10
M13	35600-0	46	46	225	Typ Dowel Conn	245	1.09
G14	35700-0	79	46	225	Typ Dowel Conn	223	0.99
H14	35800-0	83	46	225	Typ Dowel Conn	221	0.98
L14	35900-0	93	46	225	Typ Dowel Conn	213	0.95
M14	36000-0	97	46	225	Typ Dowel Conn	211	0.94
J14	36100-0	86	46	225	Typ Dowel Conn	218	0.97
K14	36200-0	90	46	225	Typ Dowel Conn	216	0.96
J13	36300-0	36	46	225	Typ Dowel Conn	251	1.12
K13	36400-0	39	46	225	Typ Dowel Conn	249	1.11

Results with Liquefied Soil

Pier 15 Pile Head Load Effects		Pier 15 Mpo Transverse Pushover (North) w/ Non-Liq Soil					
		(kip)	(kip)	(kip-ft)	P-M Connection	(kip-ft)	Moment
Pile	Member	Axial	Shear	Moment	Relationship	Mp	Mp/Mr
G1	30100-0	-576	46	199	Typ Dowel Conn	572	2.88
H1	30200-0	-574	46	199	Typ Dowel Conn	571	2.87
J1	30300-0	-571	46	198	Typ Dowel Conn	570	2.87
K1	30400-0	-568	46	198	Typ Dowel Conn	568	2.86
L1	30500-0	-563	46	199	Typ Dowel Conn	567	2.85
M1	30600-0	-558	46	199	Typ Dowel Conn	565	2.84
G2	30700-0	-524	46	199	Typ Dowel Conn	550	2.77
H2	30800-0	-522	46	199	Typ Dowel Conn	549	2.76
J2	30900-0	-519	46	198	Typ Dowel Conn	549	2.76
K2	31000-0	-516	46	198	Typ Dowel Conn	547	2.76
L2	31100-0	-511	46	199	Typ Dowel Conn	545	2.74
M2	31200-0	-506	46	199	Typ Dowel Conn	543	2.73
G3	31300-0	-472	46	199	Typ Dowel Conn	528	2.65
H3	31400-0	-469	46	199	Typ Dowel Conn	527	2.65
L3	31500-0	-459	46	199	Typ Dowel Conn	522	2.63
M3	31600-0	-454	46	199	Typ Dowel Conn	520	2.62
G4	31700-0	-420	46	199	Typ Dowel Conn	505	2.53
H4	31800-0	-417	46	199	Typ Dowel Conn	503	2.53
L4	31900-0	-406	46	199	Typ Dowel Conn	498	2.50
M4	32000-0	-402	46	199	Typ Dowel Conn	496	2.49
G5	32100-0	-368	46	200	Typ Dowel Conn	480	2.41
H5	32200-0	-365	46	199	Typ Dowel Conn	479	2.40
L5	32300-0	-355	46	199	Typ Dowel Conn	474	2.38
M5	32400-0	-350	46	199	Typ Dowel Conn	471	2.37
G6	32500-0	-318	46	200	Typ Dowel Conn	455	2.28
H6	32600-0	-315	46	200	Typ Dowel Conn	454	2.27
L6	32700-0	-304	46	200	Typ Dowel Conn	448	2.24
M6	32800-0	-299	46	200	Typ Dowel Conn	446	2.23
G7	32900-0	-268	46	200	Typ Dowel Conn	430	2.15
H7	33000-0	-265	46	200	Typ Dowel Conn	428	2.14
L7	33100-0	-254	46	200	Typ Dowel Conn	422	2.11
M7	33200-0	-395	48	218	Typ Dowel Conn	493	2.26
G8	33300-0	-219	46	200	Typ Dowel Conn	403	2.01
H8	33400-0	-215	46	200	Typ Dowel Conn	402	2.00
L8	33500-0	-204	46	200	Typ Dowel Conn	396	1.97
M8	33600-0	-317	49	218	Typ Dowel Conn	455	2.08
G9	33700-0	-170	46	201	Typ Dowel Conn	376	1.88
H9	33800-0	-167	46	201	Typ Dowel Conn	374	1.87
L9	33900-0	-156	46	201	Typ Dowel Conn	368	1.84
M9	34000-0	-240	49	219	Typ Dowel Conn	415	1.90
G10	34100-0	-121	46	201	Typ Dowel Conn	348	1.74
H10	34200-0	-118	46	201	Typ Dowel Conn	346	1.73
L10	34300-0	-107	46	200	Typ Dowel Conn	340	1.70
M10	34400-0	-103	46	200	Typ Dowel Conn	338	1.68
G11	34500-0	-72	46	200	Typ Dowel Conn	319	1.59
H11	34600-0	-69	46	200	Typ Dowel Conn	317	1.58
L11	34700-0	-58	46	200	Typ Dowel Conn	310	1.55
M11	34800-0	-54	46	200	Typ Dowel Conn	308	1.54
G12	34900-0	-23	46	200	Typ Dowel Conn	289	1.44
H12	35000-0	-19	46	200	Typ Dowel Conn	287	1.43
L12	35100-0	-8	46	200	Typ Dowel Conn	280	1.40
M12	35200-0	-4	46	200	Typ Dowel Conn	277	1.38
G13	35300-0	27	46	200	Typ Dowel Conn	257	1.28
H13	35400-0	31	46	200	Typ Dowel Conn	255	1.27
L13	35500-0	42	46	200	Typ Dowel Conn	248	1.24
M13	35600-0	45	46	200	Typ Dowel Conn	245	1.23
G14	35700-0	77	46	200	Typ Dowel Conn	224	1.12
H14	35800-0	81	46	200	Typ Dowel Conn	222	1.11
L14	35900-0	92	46	200	Typ Dowel Conn	214	1.07
M14	36000-0	95	46	200	Typ Dowel Conn	212	1.06
J14	36100-0	85	46	200	Typ Dowel Conn	219	1.10
K14	36200-0	88	46	200	Typ Dowel Conn	217	1.08
J13	36300-0	35	46	200	Typ Dowel Conn	252	1.26
K13	36400-0	38	46	200	Typ Dowel Conn	250	1.25

Results with Non-Liquefied Soil

7.5.2 P15/18 Investigation of Pile Axial forces from the Governing Results

The following represents the envelope of pile axial forces from the overstrength pushover cases for Piers 15 and 18. The typical prestressed pile doweled connections include 6 - #8 A615 dowels. The concrete filled damaged prestressed piles at Pier 15 were noted to include 8 - #8 A615 dowels; however, these piles were not found to govern.

Envelope of Pile Axial Forces (kip)			
	Pier 15	P18 Typ.	P18 Repair 36"
Tension	97	142	388
Comp.	-698	-316	-1311
	Repaired 36" Piles Govern P18		

An adequate dowel capacity was provided to develop the maximum pile tensile axial force for typical piles at Piers 15 and 18, as shown. Remember that the repaired piles of Pier 18 have a significant increased stiffness relative to the typical piles, drawing additional forces. Modifications to the doweled connections for the 36” x 0.75” concrete filled steel repair piles were not noted in the as-built plans. Elevations for the repair pile indicate limited embedment into the footing; therefore, partial axial development for the pile section was not considered.

Tensile Strength of the Doweled Connection with 6 - #8 A615 Bars

$Ast := 6 \cdot (0.79in^2) = 4.74 \cdot in^2$ area of mild longitudinal reinforcement

$fy_{rebar} := 60ksi$ nominal yield stress of the dowel reinforcement

$fye_{rebar} := 68ksi$ expected yield stress of the dowel reinforcement

$Tn_{nominal} := Ast \cdot fy_{rebar} = 284.4 \cdot kip$ strength with nominal properties for comparison

$Tn_{expected} := Ast \cdot fye_{rebar} = 322.32 \cdot kip$ tensile capacity of the doweled connection

$fue_{rebar} := 95ksi$ expected ultimate stress of the dowel reinforcement

$Tu_{expected} := Ast \cdot fue_{rebar} = 450.3\,kip$ for comparisononly, moment capacity significantly reduced for tensile axial forces beyond Tn.expected

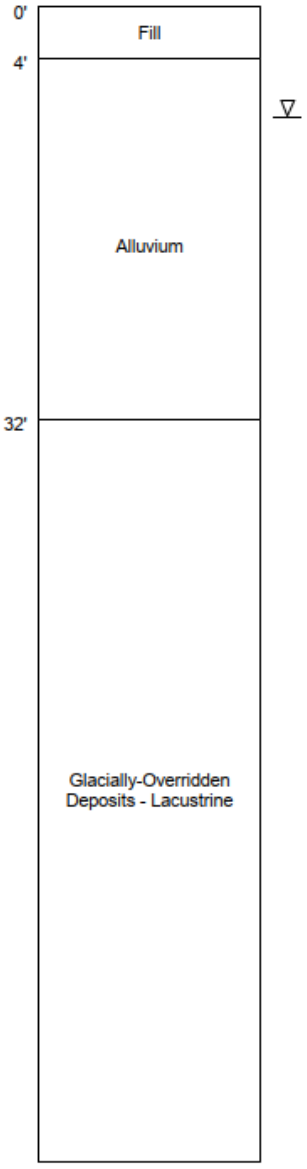
A comparison of the envelope pile axial forces and the unfactored axial resistance of the piles from recommendations provided by Shannon and Wilson appears on the following pages. Annotations have been made over their recommendations to determine the unfactored axial resistance of the piles at the average pile tip elevation. Communications with the Shannon and Wilson indicated that downdrag effects on piles due to settlement are only applicable to the post-earthquake evaluation; therefore, they were not considered to act with the overstrength demands. An adequate unfactored tension and compressive axial resistance exists in the piles relative to the maximum individual pile overstrength axial force demands.

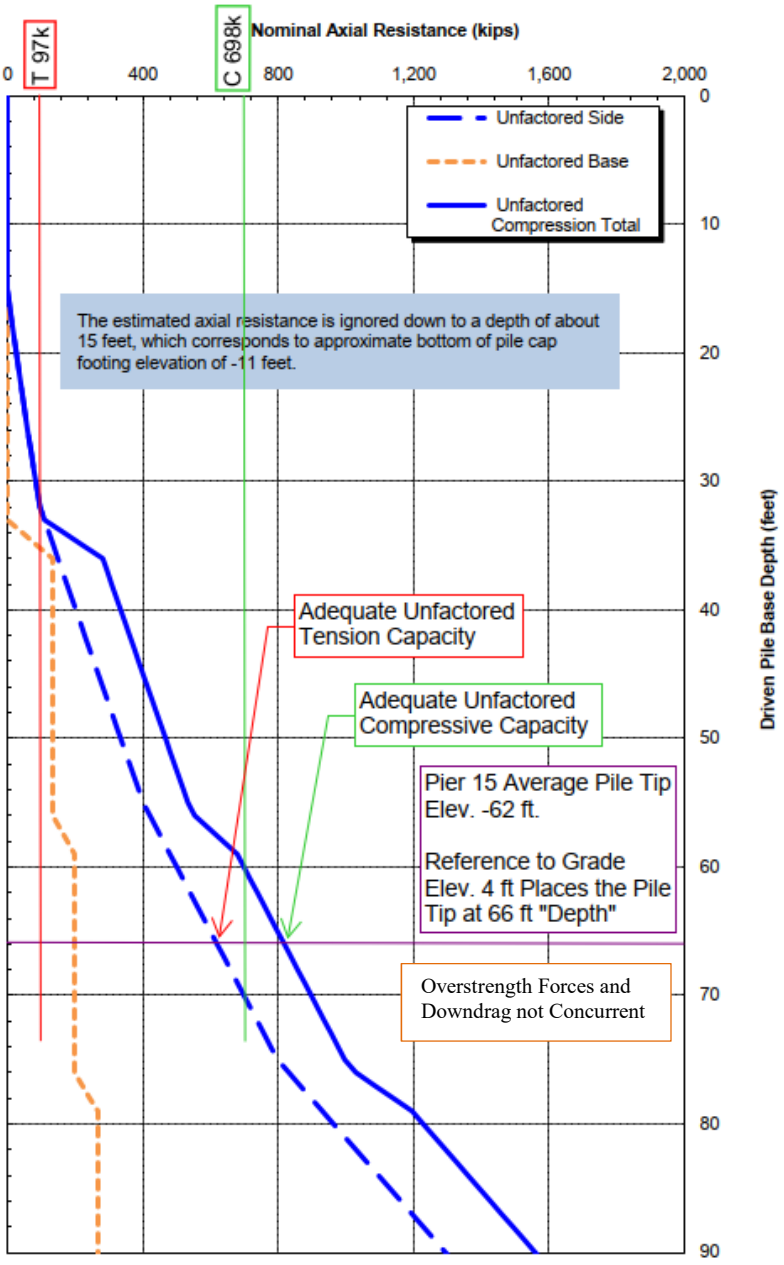
KDK

Envelope of Pile Forces (kip)		
	Pier 15	Pier 18
Tension	97	388
Comp.	-698	-1311
Repaired 36" Piles Govern P18		

ASSUMED SUBSURFACE PROFILE

Based on nearby explorations:
WS-2
(Reference Grade Elev. = 4 ft.)





NOTES

1. The analyses were performed for a single shaft based on guidelines included in AASHTO, and our experience.
2. The factored total shaft resistance is determined by adding the unfactored side and base resistances, multiplied by appropriate resistance factors per AASHTO.

West Seattle High Level Bridge
Seattle, Washington

ESTIMATED AXIAL RESISTANCE OF 24-IN DRIVEN CONCRETE PILE PIER 15

September 2020 105492-001

SHANNON & WILSON

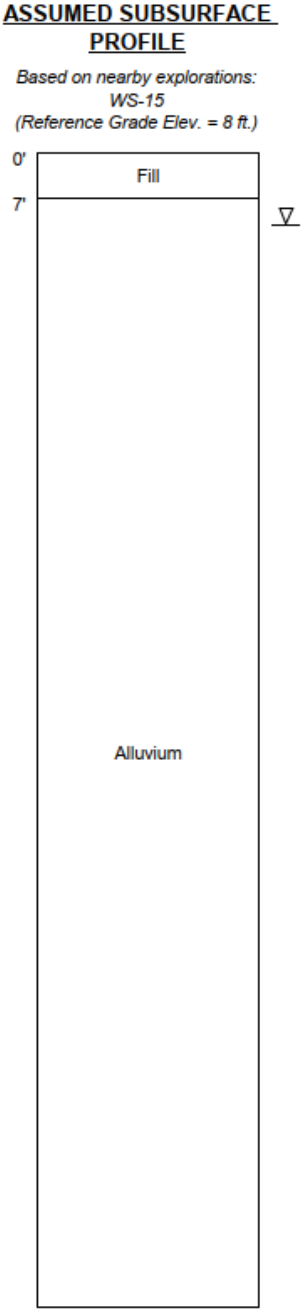
FIG.

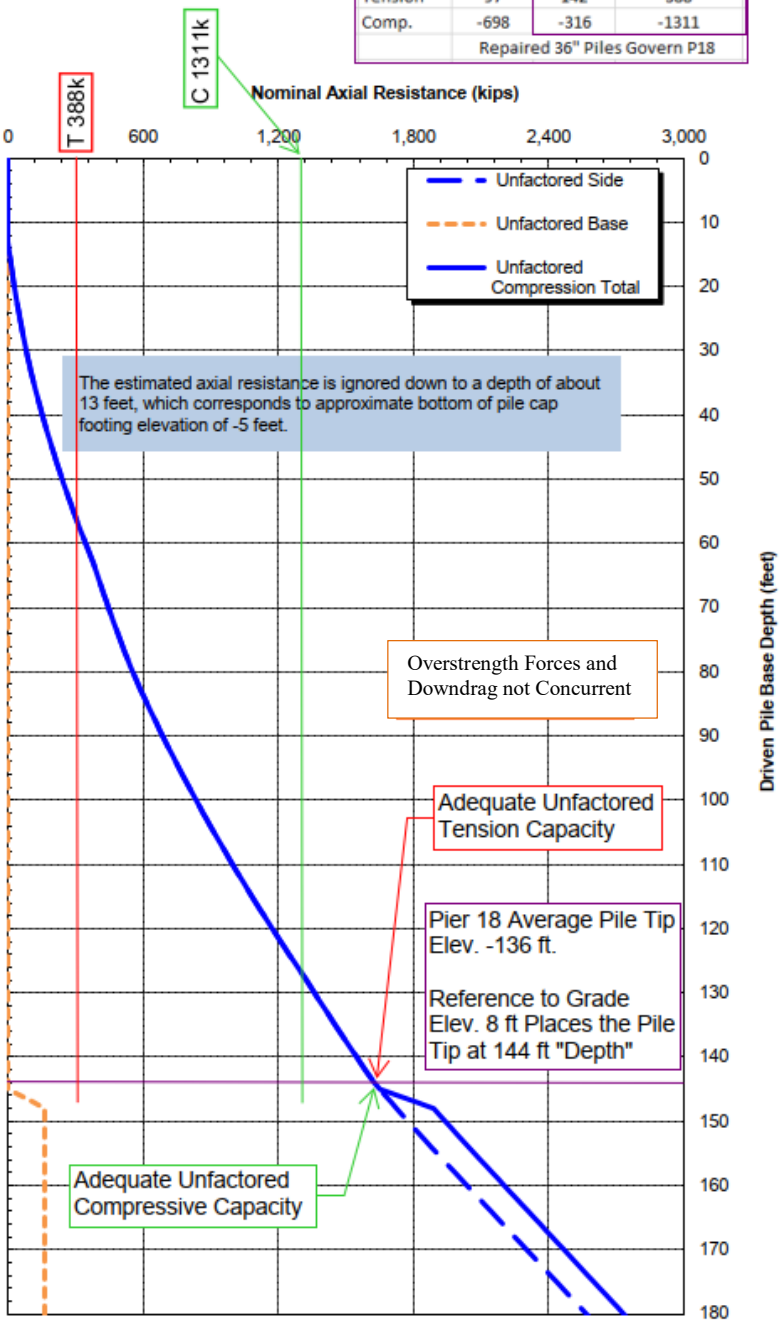
KDK

Envelope of Pile Axial Forces (kip)			
	Pier 15	P18 Typ.	P18 Repair 36"
Tension	97	142	388
Comp.	-698	-316	-1311
	Repaired 36" Piles Govern P18		

ASSUMED SUBSURFACE PROFILE

Based on nearby explorations: WS-15 (Reference Grade Elev. = 8 ft.)





NOTES

1. The analyses were performed for a single shaft based on guidelines included in AASHTO, and our experience.
2. The factored total shaft resistance is determined by adding the unfactored side and base resistances, multiplied by appropriate resistance factors per AASHTO.

West Seattle High Level Bridge
Seattle, Washington

ESTIMATED AXIAL RESISTANCE OF 24-IN DRIVEN CONCRETE PILE PIER 18

September 2020 105492-001

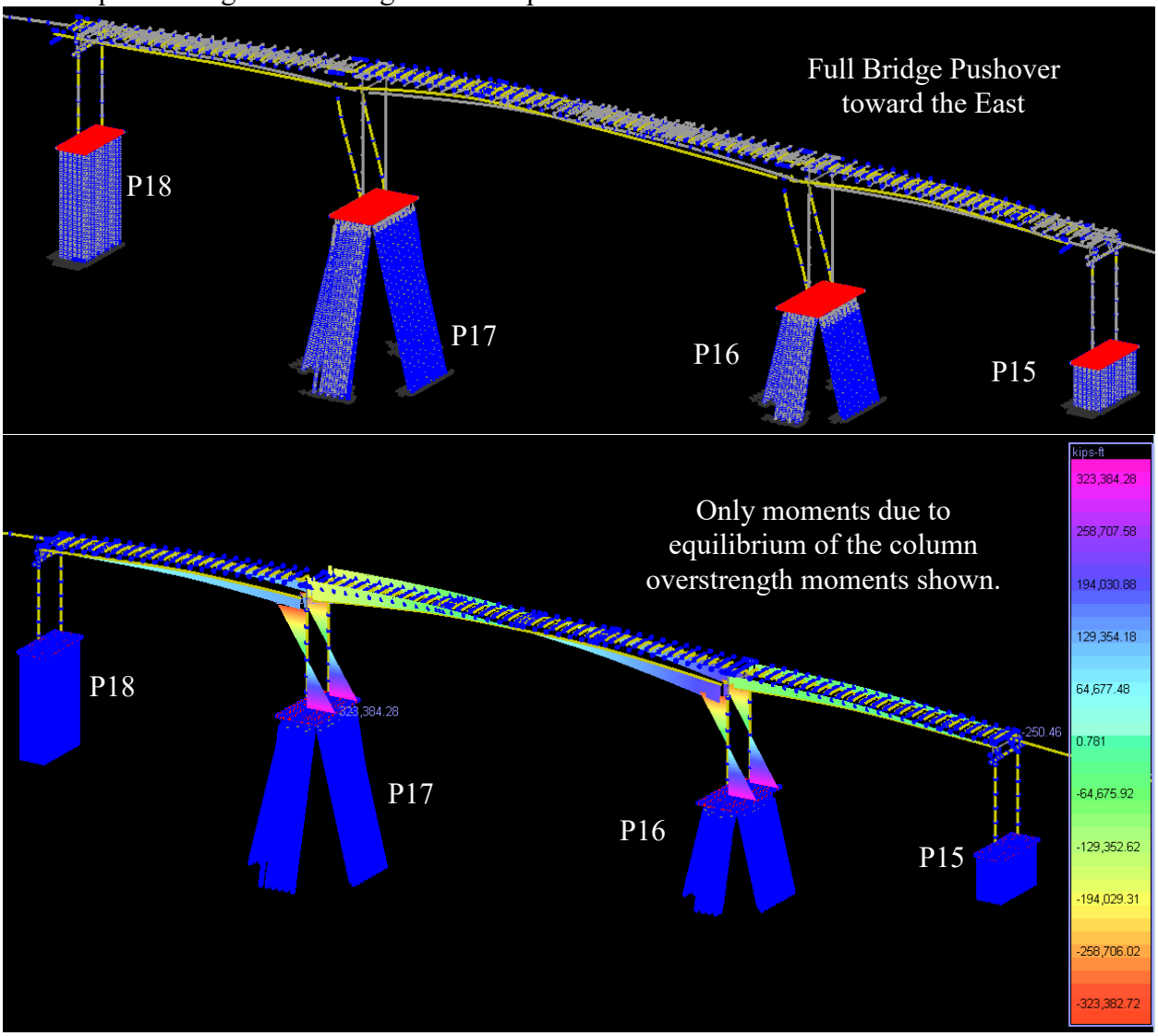
SHANNON & WILSON **FIG.**

8.0 EVALUATION OF SUPERSTRUCTURE SEISMIC DEMANDS

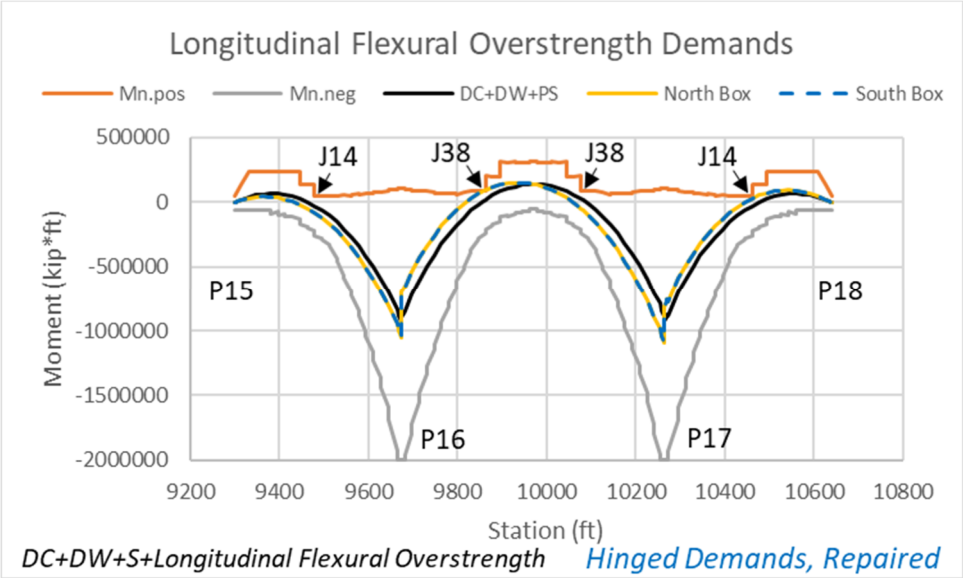
This section provides a preliminary evaluation for superstructure demands under longitudinal and vertical seismic response. As a capacity protected element, the superstructure must allow for the development of the intended substructure mechanism at flexural overstrength. The effects of vertical accelerations become important for the WSHB due to the long span lengths. Superstructure demands corresponding to the discussed preliminary vertical spectra are presented. Next, evaluations for the 100-30 combination rule are explored. The columns remain within the elastic range under 0.3L+0.3T+1.0V, thus the RSA forces are directly applicable. For 1.0L+0.3T+0.3V, the structure develops a longitudinal mechanism, therefore forces corresponding to the expected flexural mechanism are added to 30% of the vertical RSA forces. The vertical RSA forces are expected to be insensitive to the formation of a longitudinal mechanism, since the structure does not fuse vertically. Preliminary results are discussed.

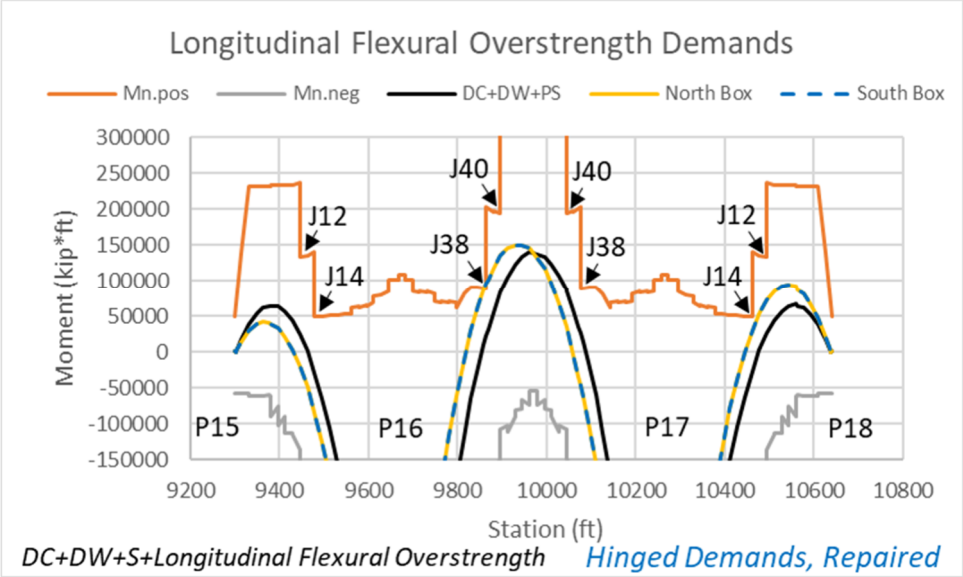
8.1 SUPERSTRUCTURE DEMANDS AT FLEXURAL OVERSTRENGTH

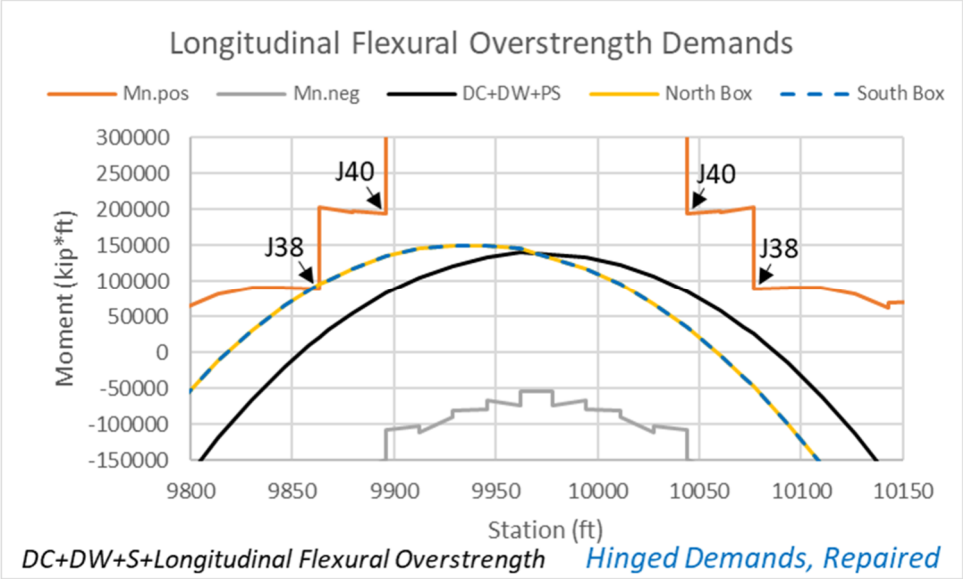
This section provides an evaluation for superstructure forces due to flexural overstrength of the column plastic hinges under longitudinal response. These forces are shown below.



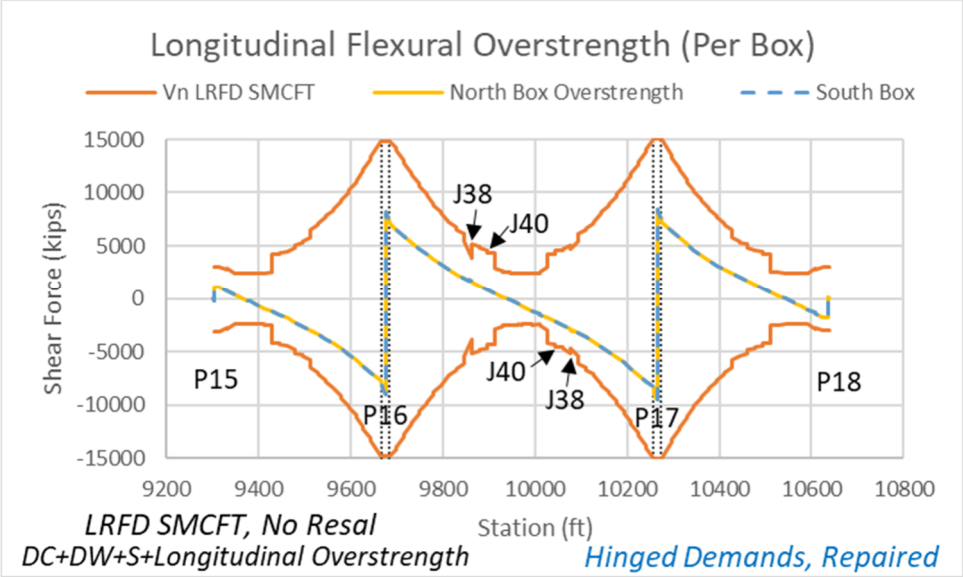
The following figures compare the superstructure overstrength demands to the provided moment capacity without phi-factors using the current work-in-progress Phase II PT strengthening layout. Note that this layout was initially sized for AASHTO strength and service loading only. In the figures, S represents the secondary PT moments. Note that only the more conservative Hinged Model superstructure load effects were considered. The Hinged Demands includes the effects of redistributed forces due to existing distress at Joints 38 that are locked in by the retrofit PT.

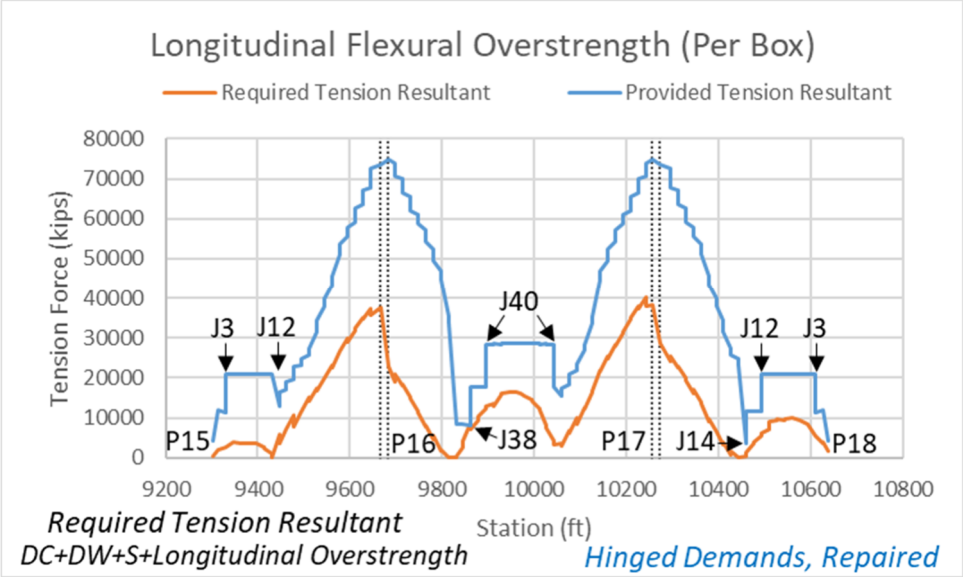


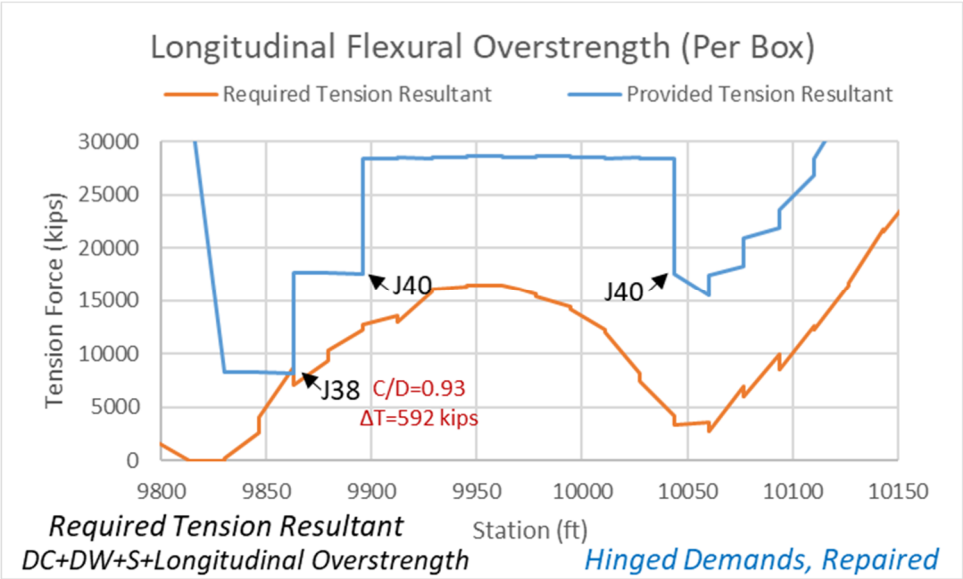


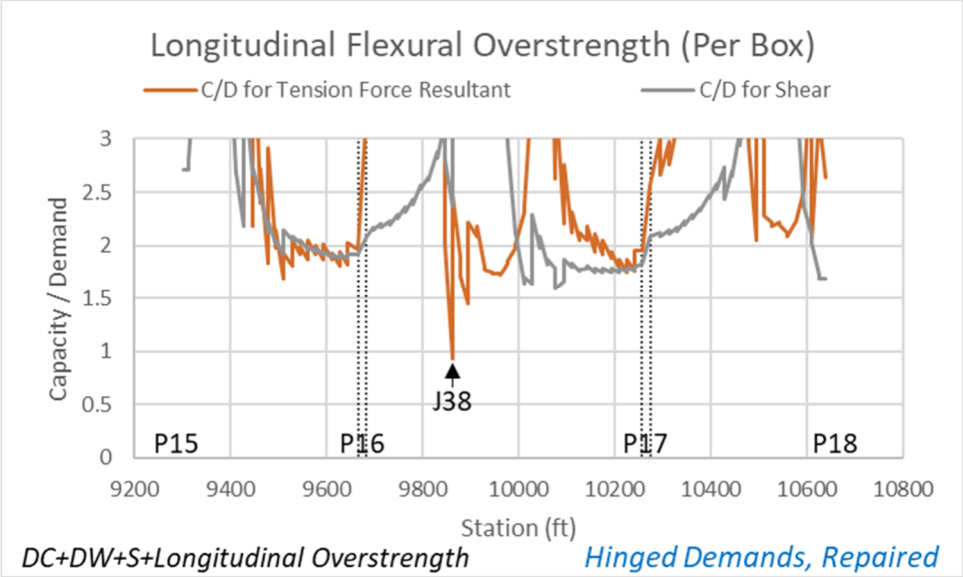


Joints 38 are right at capacity under the longitudinal flexural overstrength demands without including the contribution of the added soffit CFRP or any stress gain beyond fpe in the unbonded retrofit tendons due to deformation compatibility. These behaviors will increase the capacity. The moment capacity at Joint 14 currently exceeds the overstrength demands, but reductions in the side span PT are planned, so this remains an area of future interest.




Longitudinal Flexural Overstrength (Per Box)
Required Tension Resultant
Provided Tension Resultant
Tension Force (kips)
80000
70000
60000
50000
40000
30000
20000
10000
0
J3
J12
J40
J12
J3
P15
P16
J38
P17
J14
P18
9200
9400
9600
9800
10000
10200
10400
10600
10800
Station (ft)
Required Tension Resultant
DC+DW+S+Longitudinal Overstrength
Hinged Demands, Repaired

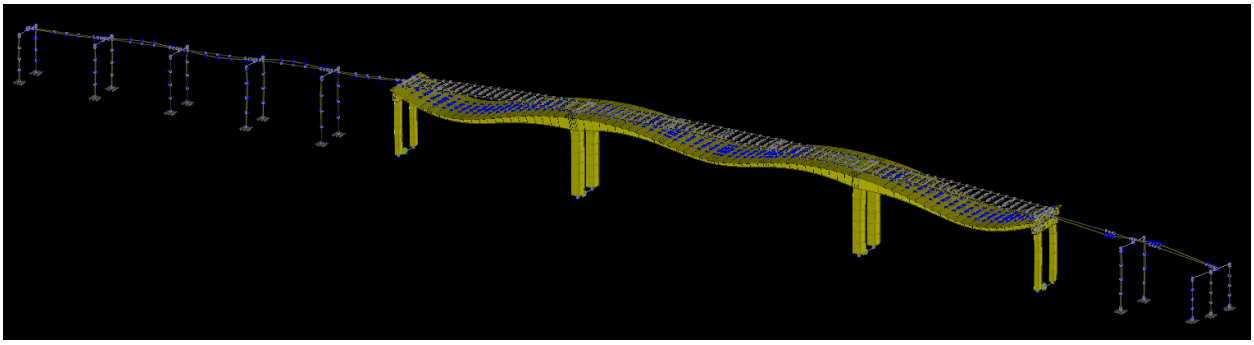

Longitudinal Flexural Overstrength (Per Box)
Required Tension Resultant
Provided Tension Resultant
Tension Force (kips)
30000
25000
20000
15000
10000
5000
0
J40
J40
J38
C/D=0.93
ΔT=592 kips
9800
9850
9900
9950
10000
10050
10100
10150
Station (ft)
Required Tension Resultant
DC+DW+S+Longitudinal Overstrength
Hinged Demands, Repaired

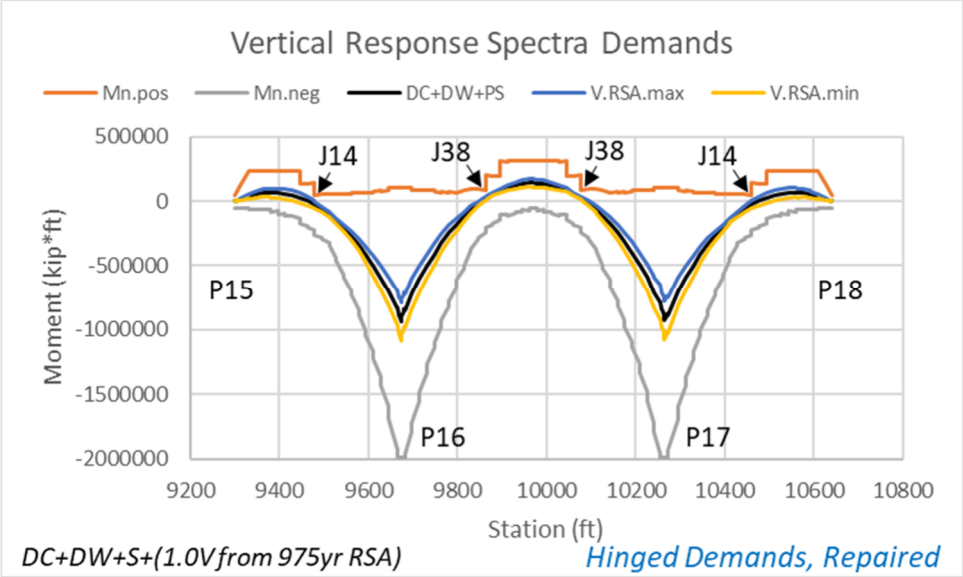


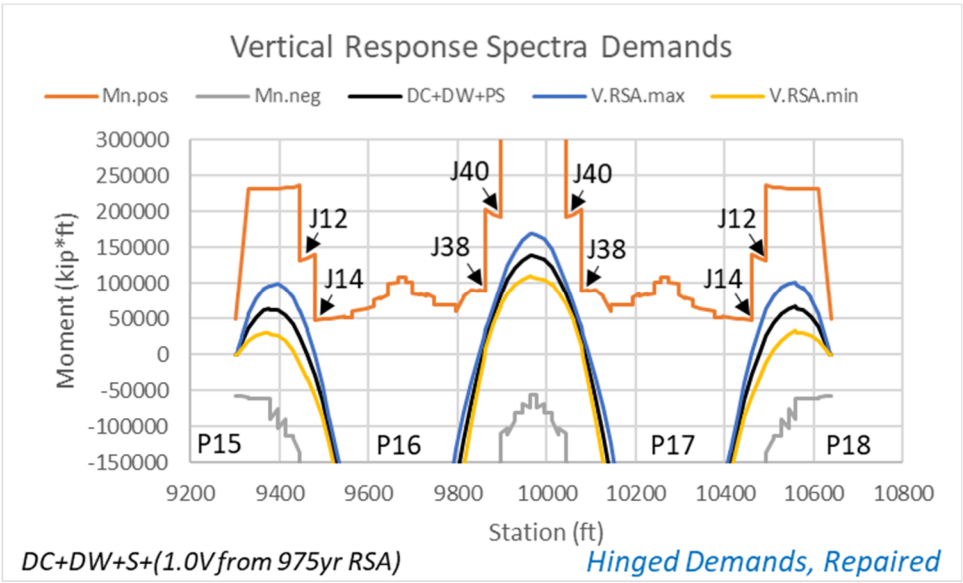
Flexural capacity just exceeded on the Pier 16 side Joint 38 for longitudinal pushover from west to east (ahead station). The flexural exceedance leads to an inadequate tension force resultant C/D = 0.93 at Joint 38, where the provided reinforcement is 592 kips short of the required value. The shear demands are generally low, so the consequences of exceedance are largely flexural in nature and localized to Joints 38 (both locations due to the cyclic nature of the earthquake). Note that no strength reduction factors were included in the moment and shear evaluation, consistent with seismic design practice.

8.2 SITE-SPECIFIC VERTICAL SPECTRA RSA DEMANDS

The following figures provide an evaluation for superstructure demands from a RSA analysis using the 975yr SEE site-specific vertical spectra. Note that RSA results are unsigned, therefore a maximum and minimum case is presented. The fundamental vertical mode shape, shown below, has a period of 0.84 seconds. As shown in the following figures, the vertical acceleration demands are large, particularly in the side spans near Joint 14.





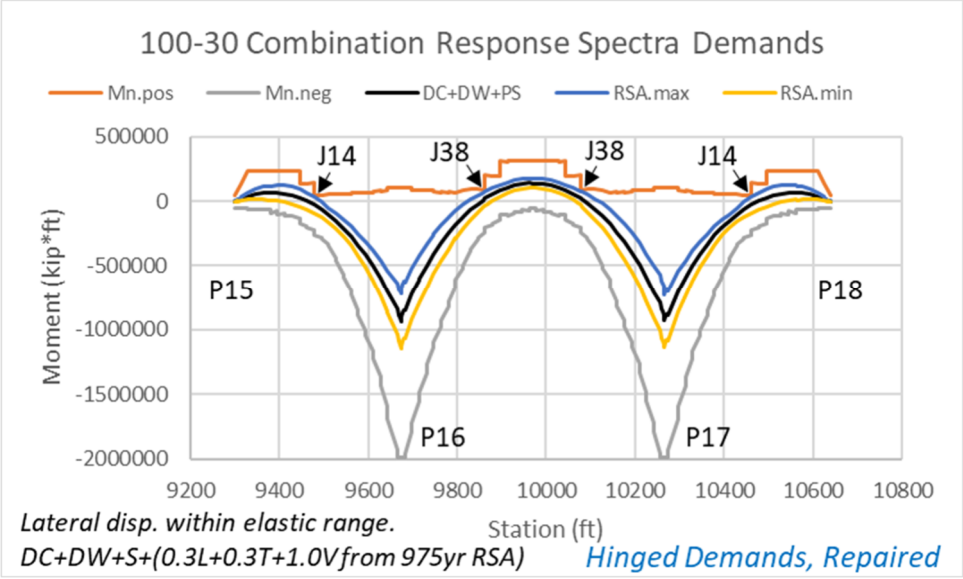


8.3 100-30 DIRECTIONAL COMBINATION RULE SUPERSTRUCTURE DEMANDS

This section evaluates superstructure demands corresponding to the 100-30 combination rule.

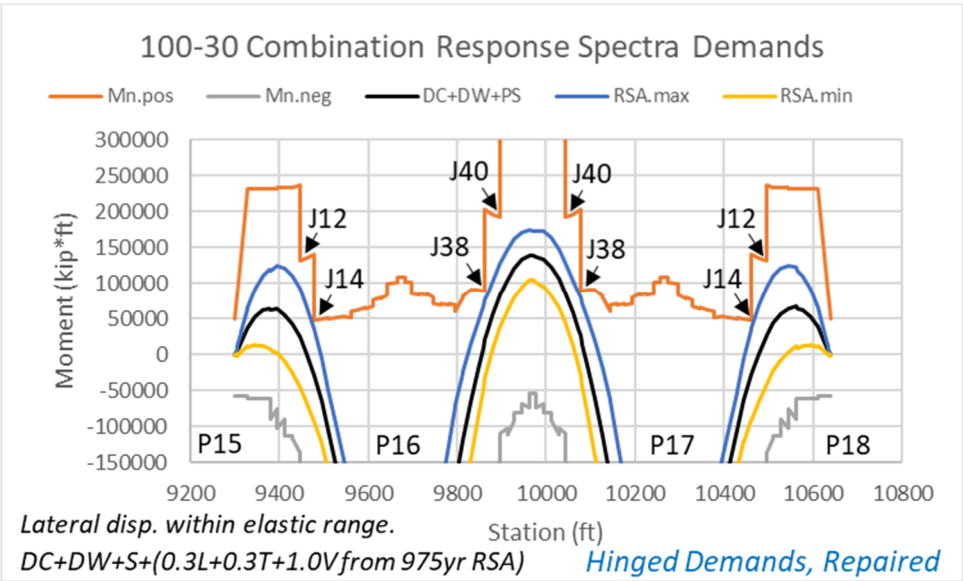
8.3.1 Superstructure Demands for 0.3L+0.3T+1.0V

The columns remain within the elastic range under 0.3L+0.3T+1.0V, thus the RSA forces shown in the following figures are directly applicable. This is somewhat conservative, as it presupposes that prior longitudinal hinging has not reduced the stiffness and thus force at 30% of the peak longitudinal demands.



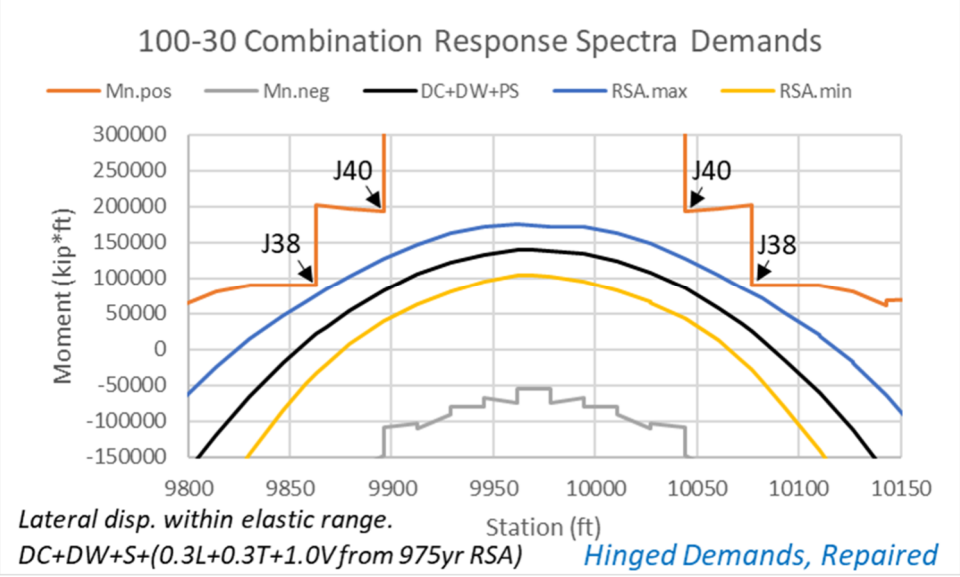
Lateral disp. within elastic range.
DC+DW+S+(0.3L+0.3T+1.0V from 975yr RSA)

Hinged Demands, Repaired



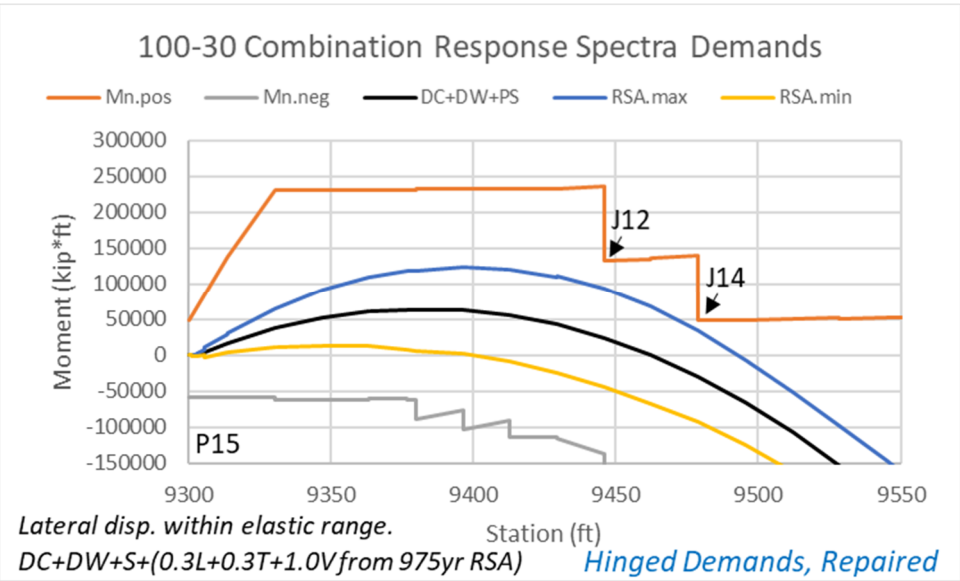
Lateral disp. within elastic range.
DC+DW+S+(0.3L+0.3T+1.0V from 975yr RSA)

Hinged Demands, Repaired



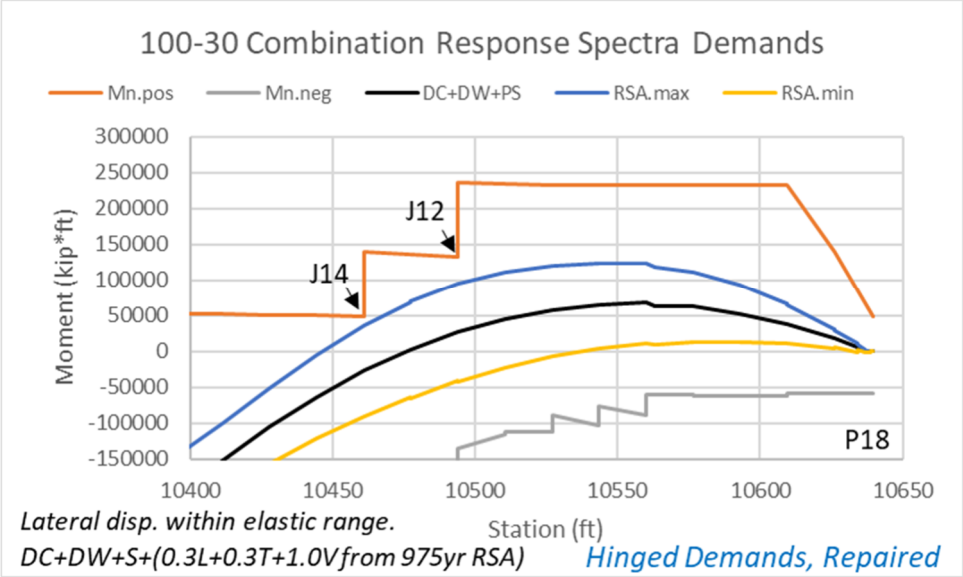
Lateral disp. within elastic range.
DC+DW+S+(0.3L+0.3T+1.0V from 975yr RSA)

Hinged Demands, Repaired



Lateral disp. within elastic range.
DC+DW+S+(0.3L+0.3T+1.0V from 975yr RSA)

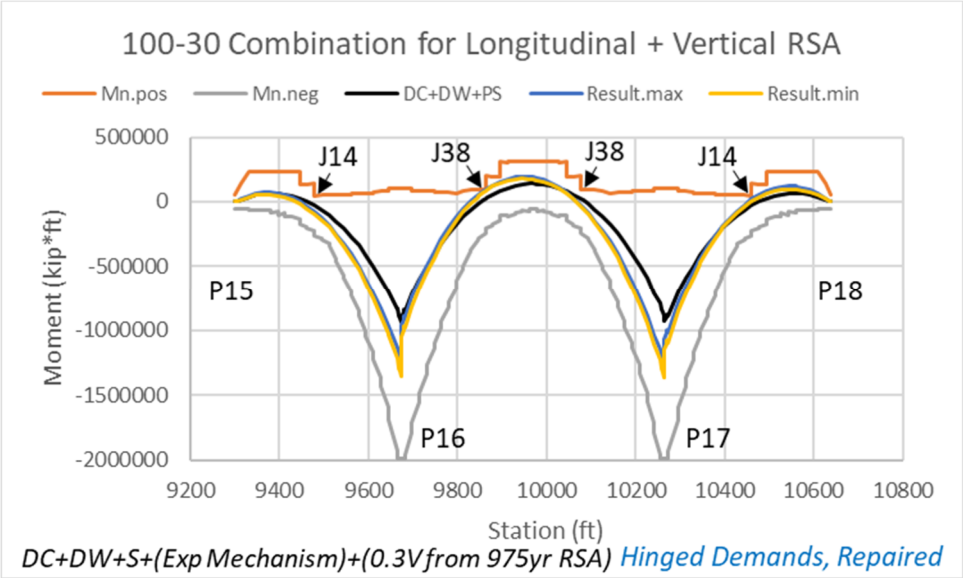
Hinged Demands, Repaired

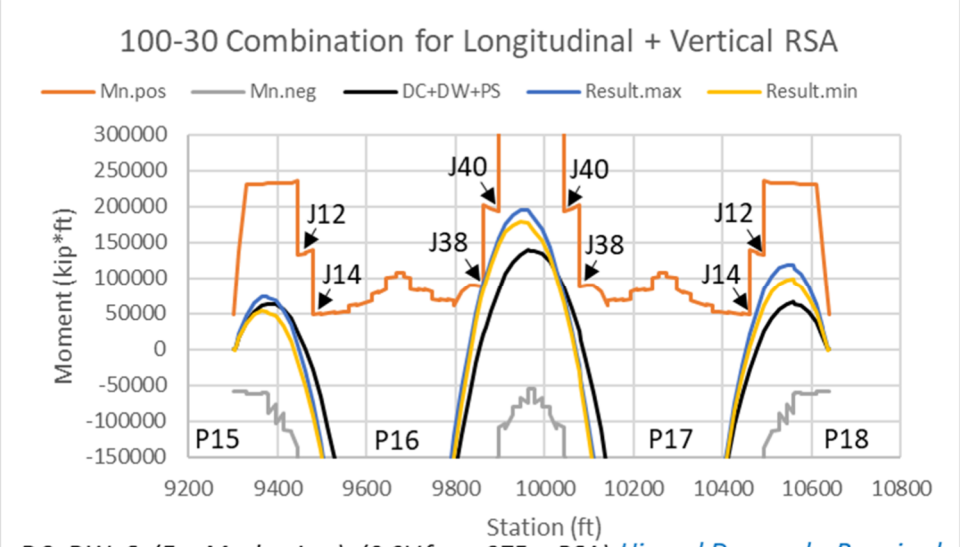


As shown, elastic superstructure forces corresponding to the 0.3L+0.3T+1.0V RSA case place Joints 38 and 14 just below their capacity. These locations align with the observed soffit and web shear cracking near the stressing blisters for the continuity tendons.

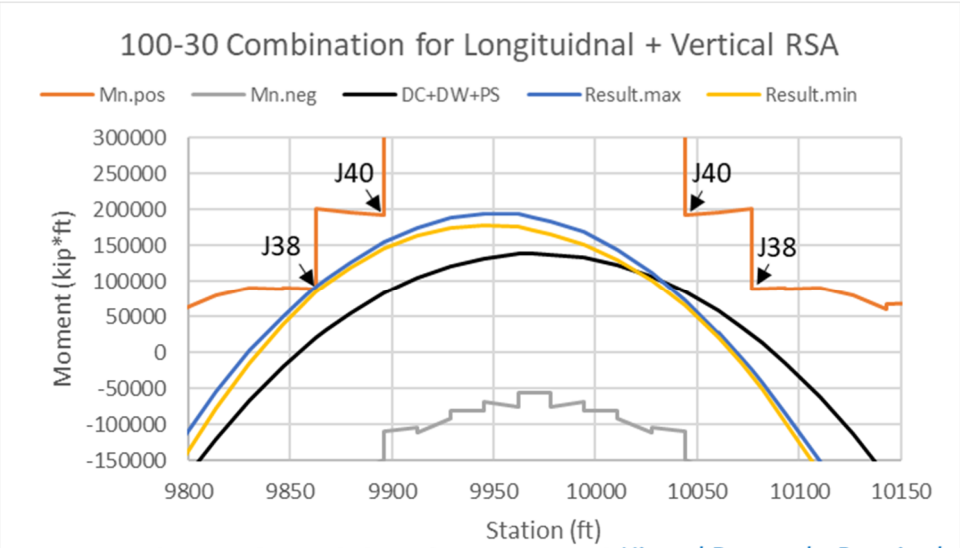
8.3.2 Demands for 1.0L (Expected Mechanism) and 0.3V (Vertical RSA)

For 1.0L+0.3T+0.3V, the structure develops a longitudinal mechanism, therefore forces corresponding to the expected flexural mechanism are added to 30% of the vertical RSA forces in the following preliminary evaluation. The vertical RSA forces are expected to be insensitive to the formation of a longitudinal mechanism, since the structure does not fuse vertically.

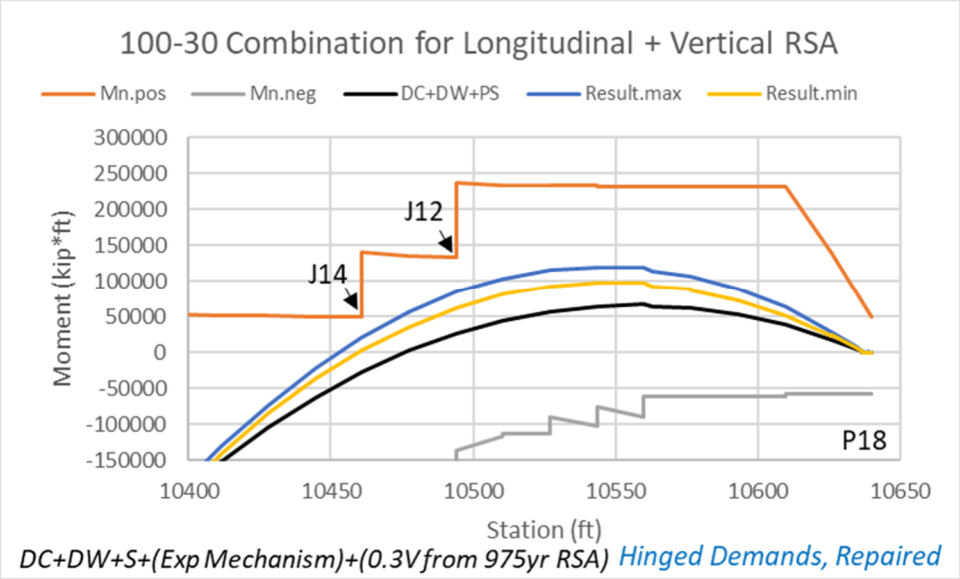




DC+DW+S+(Exp Mechanism)+(0.3V from 975yr RSA) *Hinged Demands, Repaired*



DC+DW+S+(Exp Mechanism)+(0.3V from 975yr RSA) *Hinged Demands, Repaired*



As shown, the evaluation for 1.0L (Expected Mechanism) and 0.3V (Vertical RSA) leads to demands that effectively match the moment capacity at Joints 38. Combinations of overstrength and vertical RSA forces were not considered, as this combination becomes increasingly unlikely. Calculations for the moment capacity conservatively ignore the contribution of the added soffit CFRP and any stress gain beyond fpe in the unbonded retrofit tendons due to deformation compatibility. These behaviors will increase the capacity.

Superstructure evaluations using the recommended site-specific spectra show that vertical accelerations are important, particularly at the locations of primary interests at Joints 38 and 14. In the prior figures, all results were compared relative to the superstructure capacity with the current work-in-progress Phase II PT layout initially sized for AASHTO Strength and Service loading. Regions that are at capacity with the retrofit PT would have a corresponding exceedance if the existing structural capacity had been investigated. Thus, it can be concluded that the existing superstructure is not adequately capacity protected, with regions of both moment and shear exceedance at the distressed Joints 14 and 38. Results also indicate that seismic demands may govern for the design of Phase II permanent strengthening PT, since regions at capacity may require additional strengthening pending results of the more refined estimate for the capacity of the retrofitted structure.

WEST SEATTLE HIGH-RISE BRIDGE
SUPERSTRUCTURE REPLACEMENT FEASIBILITY

Introduction

This memorandum is intended to be a brief summary of the considerations and take-aways identified when determining the apparent low cost alternative for superstructure replacement. This memorandum is not intended to be comprehensive of all the details of all the alternatives considered. There are many different superstructure types that could be utilized, however, for Cost-benefit Analysis (CBA) purposes, with limited scope and schedule, the apparent low cost alternative was identified and carried forward through the quantitative evaluation process.

The existing high-rise bridge was originally designed to be either balanced cantilever concrete box structures or steel tub box girder structures. Though the design of a superstructure replacement would incorporate modern code requirements and detailing practices, based on preliminary analysis and assessment of the existing bridge it was deemed to be structurally feasible that the bridge superstructure could be replaced with either one of these structure types with a very similar overall geometry to what was originally designed, allowing the bridge to remain on alignment and maintain the same approximate profile. Having determined the preliminary structural feasibility, the investigation focused largely on the constructability aspects of various options.

Assumptions

The following assumptions were used:

- It is assumed that a superstructure replacement maintains the existing alignment of the structure.
- It is assumed that a superstructure replacement maintains the existing profile and ties in at the existing approaches.
- No modification of approach structures was considered as part of the options investigated, and no assessment of existing approach structures was conducted.
- It is assumed that any options must maintain the operation of the navigation channel and adjacent low-rise bridge during and after construction, assuming only intermittent short-term closures.
- Maintain existing vertical clearance above the roadway
- The WSHB corridor will be closed during construction of a given alternative
- Preliminary geotechnical input was limited to pile axial capacities, preliminary lateral soil design parameters (including liquefaction considerations), and preliminary seismic hazard information.
- A preliminary corrosion assessment was used to define assumptions for the condition of the existing foundations.

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- If determined to be adequate for today's standards, the columns and main pier diaphragm reinforcement would be reused.

Demolition and Foundations

The on-alignment superstructure replacement involves demolishing the existing box girders between Pier 15 and Pier 18 and replacing the superstructure elements, reusing the current foundations and substructure. Various demolition concepts have been presented in Appendix G. Demo would have to be done extra carefully as to not overstress the columns and to preserve enough steel to tie in the new superstructure. Figure 1 below depicts how the superstructure could be demolished one segment at a time in manner like the way it was built.

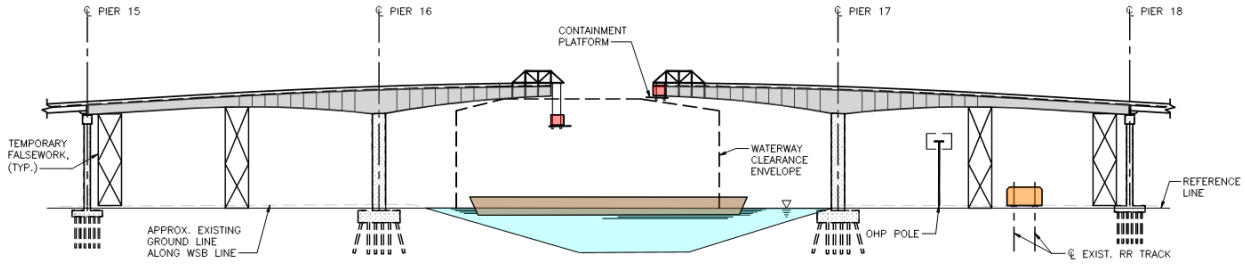

Figure 1: Demolition of the Existing Superstructure

Preliminary analyses indicate that the foundations would need some level of retrofit in order to support the new superstructure, which would need to be retrofit concurrent with superstructure replacement. Potential foundation strengthening concepts were developed based on limited geotechnical input as outlined in the assumptions. Foundation strengthening was assumed to be the same for all options for the purposes of this assessment, and conservative rough order-of-magnitude costs for the strengthening of existing piers 16 and 17 were included in the costs for a superstructure replacement alternative.

Balanced Cantilever Concrete Box Option

This option would essentially be an in-kind replacement of the existing bridge structure. A set of twin post-tensioned box girders would be constructed on top of the existing columns using balanced cantilever construction methods.

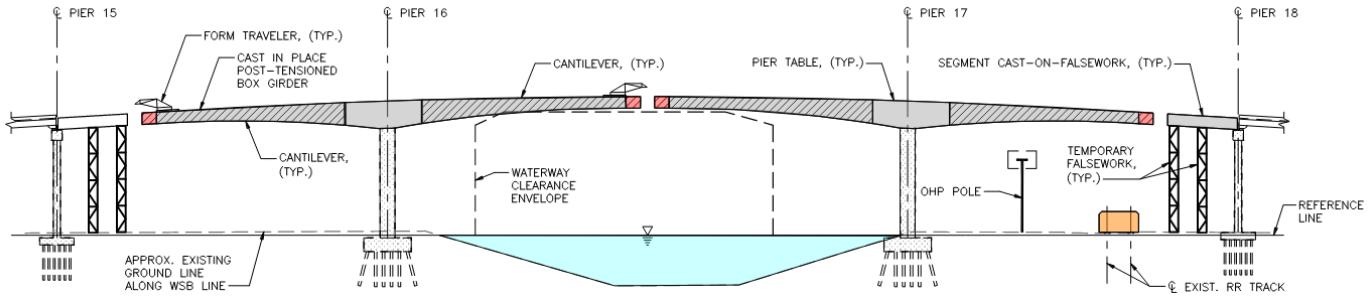

Figure 2: Construction of a Balanced Cantilever using Form Travelers

Basic demolition/construction sequence:

- Demolish existing superstructure, taking care to preserve existing column bars.
- Construct new pier tables to support segmental box girder construction.
- Construct new concrete box girders as balanced cantilevers from each main pier.

Advantages of this option:

- Balanced cantilever construction requires no temporary works within the river, aside from what is needed along the banks to construct the pier table.
- Requires no shoring towers in adjacent roadways or private properties, however end segments could be cast on falsework where space exists.
- A concrete option is likely to provide lower maintenance and operations costs
- Due to construction methods, this option would have the fewest impacts to existing development beneath the bridge.

Challenges associated with this option:

- Requires specialized equipment (form travelers), which add significant cost to the project
- Cantilever construction can impose large out-of-balance moments, which are a risk for the existing columns and foundations.
- Heavier structure may require more significant foundation strengthening (to be assessed in detail beyond this study).
- Transverse fit up between boxes may pose challenges due to time-dependent effects in the concrete. The original construction addressed this using (4) form travelers, such that boxes could be segmentally cast simultaneously.

Costs:

Rough order-of-magnitude costs for the balanced cantilever box girder were based on the assumption of a similar concrete outline to the existing structure, with increased reinforcement and post-tensioning allowances based on existing knowledge about the structure gained as part of the current bridge stabilization, as well as comparison with similar structure types designed and constructed recently in the area. Costs assumed only 2 form travelers. An additional 20%

contingency allows for potential costs associated with additional form travelers and/or other increases in structural cost which may be determined based on a more robust investigation.

Steel Box Girder Option

This option assumes replacement of the existing bridge structure with a set of twin steel box girders constructed on top of the existing columns, erected in segments using falsework towers.

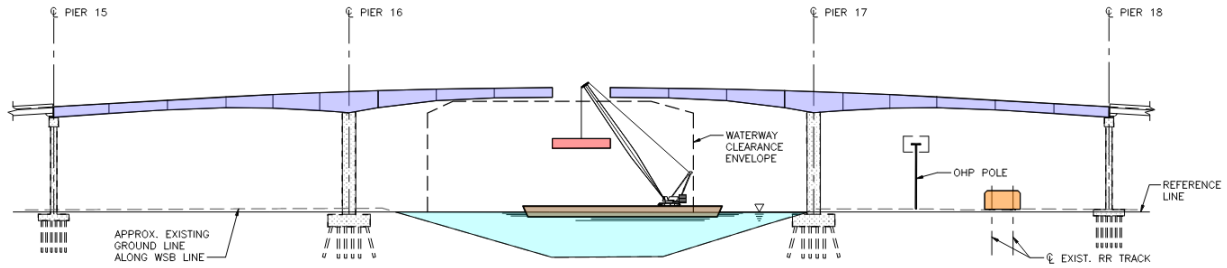

Figure 3: Construction of Steel Box Girder Main Span

Basic demolition/construction sequence:

- Demolish existing superstructure, taking care to preserve existing column bars.
- Build up existing piers to the elevations required to support steel box girders, trimming existing column bars down to that elevation.
- Erect new girders, using splice/shore towers as required.
- Girder segments would likely need to be erected in pairs for stability, with lateral bracing between them already installed. Erection could progress segmentally.

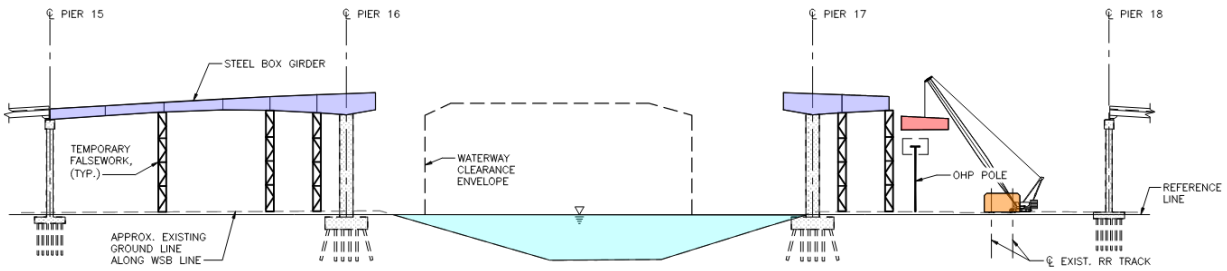

Figure 4: Construction of Steel Box Girder End Spans

Advantages of this option:

- Erection of steel superstructure likely faster than concrete, meaning less construction time on-site.
- Steel girders provide a lighter structure, which may produce less demand on the existing foundations.

Challenges associated with this option:

- Splice towers will be significant and expensive.
- In-water work will be required in a location with significant marine traffic.
- Foundation strengthening is likely required.
- Steel is a long lead time item, which is often subject to requirements that it be produced within the United States, increasing risk for cost and schedule delays.

Costs:

Rough order-of-magnitude costs for the twin steel box girders were estimated based on the assumption of similar steel quantities as were provided in the original design. Costs for steel plate are based on experience with similar structure types designed and constructed recently in the area. The majority of structure costs are in this material, with the next large contributor to cost being the temporary works, assumed to be about 10% of the total structure cost.

Other Superstructure Options

Various other options and construction methods were investigated as potential superstructure replacements. Among these structure types are extradosed, cable-stayed, or network tied-arch superstructures. An extradosed option utilizing either a steel or concrete box was considered one of the more viable of these options. An extradosed option was preferred to a traditional cable-stayed due to the shorter towers required, which have less impact on vertical clearance above the roadway which could affect air traffic. Cantilever erection would minimize temporary works on the ground and keep construction out of the river. Due to the cables, the superstructure depth could be reduced, whether made of steel or concrete, which could offset the weight added to the foundations due to the towers.

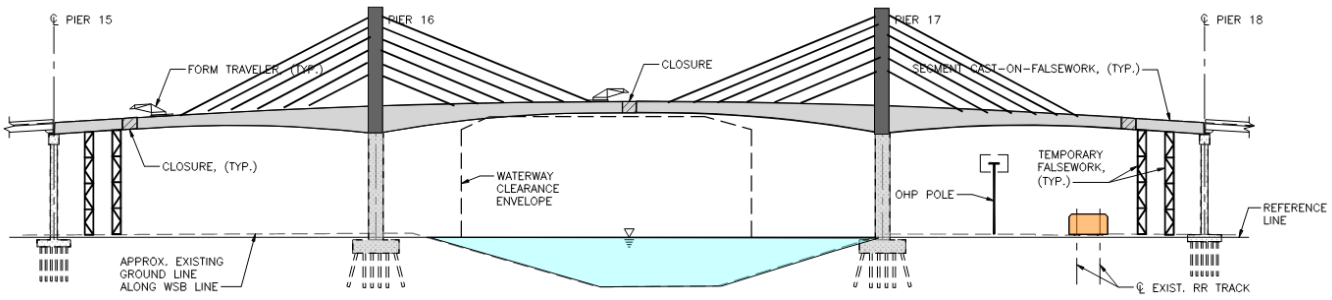

Figure 5: Extradosed Superstructure Option (Reduced Superstructure Depth not Depicted)

Basic demolition/construction sequence:

- Demolish existing superstructure, taking care to preserve existing column bars.
- Extend piers upward to provide for an extradosed bridge. Erect new superstructure (concrete or steel boxes) using the balanced cantilever method.

Advantages of this option:

- Preferred to a traditional cable-stayed due to the shorter towers required, which have less impact on vertical clearance above the roadway which could affect air traffic.
- Cantilever erection would minimize temporary works on the ground and keep construction out of the water.
- Due to the cables, the superstructure depth could be reduced, whether made of steel or concrete, which could offset the weight added to the foundations due to the towers.

Challenges associated with this option:

- Unique structure type and construction approach will require specialized knowledge.
- Potential impacts to vertical clearance above the roadway are a significant risk.
- Foundation strengthening likely needed.

Costs:

Rough order-of-magnitude costs for the extradosed superstructure were not calculated as it was determined that this structure type would not be competitive in cost with the box girder options.

Findings

The steel box girder option was estimated to be more expensive than the concrete option by about 20%, however based on the uncertainty inherent in such preliminary costs, this gap could be quickly closed with the progression of design and the two options can be considered roughly equal as far as cost impacts, both performing better from an initial capital cost perspective than additional superstructure types considered.

A superstructure replacement option was incorporated into the CBA as Alternative 4. This superstructure replacement is assumed to be a girder structure composed of either steel tub girders or concrete box girders. Following an apparent low-cost approach, WSP eliminated other structure types, such as extradosed, cable-stayed, or network tied-arch superstructures.

Key considerations & Take-Aways

The key considerations and take-aways are not necessarily specific to the apparent low cost alternative. The key considerations and take-aways within this appendix acknowledge that the actual superstructure replacement may be different than what was considered in the CBA, and intend to highlight project specific conditions that could influence such decisions, with the objective of minimizing repeat work with what has been considered to-date in order to maximize efficiency in future phases of work.

If a girder type superstructure replacement, shown in figures 2 through 4 above, is to be further pursued, the following should be considered:

- Feasibility of reuse of existing piers, including how to demolish while preserving columns
- Cost and schedule implications of steel vs concrete box girders

No matter which structure type is chosen for superstructure replacement, any will have to carefully balance the needs of many stakeholders, including but not limited to:

- Environmental (in-water work)
- FAA (overall structure height)
- BNSF (foundation work around railroad)
- USCG (navigable waterway height)
- Port of Seattle (access to harbor island during and after construction)

Figure 6: Examples of Stakeholders

APPENDIX K

BRIDGE REPLACEMENT CONCEPTS MEMORANDUM

WEST SEATTLE HIGH-RISE BRIDGE
BRIDGE REPLACEMENT TO RESTORE TRAFFIC

This appendix is only intended to provide a brief summary of key aspects considered when looking at the various ways to replace the bridge on-alignment, and restore live load on the West Seattle High-Rise Bridge (WSHB). This is described in the main report as *Alternative 5*.

All New Pier Concepts

Since the cost-benefit analysis is based on the assumption that all bridge alternatives are built on the existing alignment, new piers placed water-side of the existing main piers would be in conflict with the West Seattle Low-Bridge (WSLB) as shown in figure 1 below. Because of this conflict and the location of the navigational channel, all new piers must be placed outside of the existing main piers, forcing the main span to grow in length.

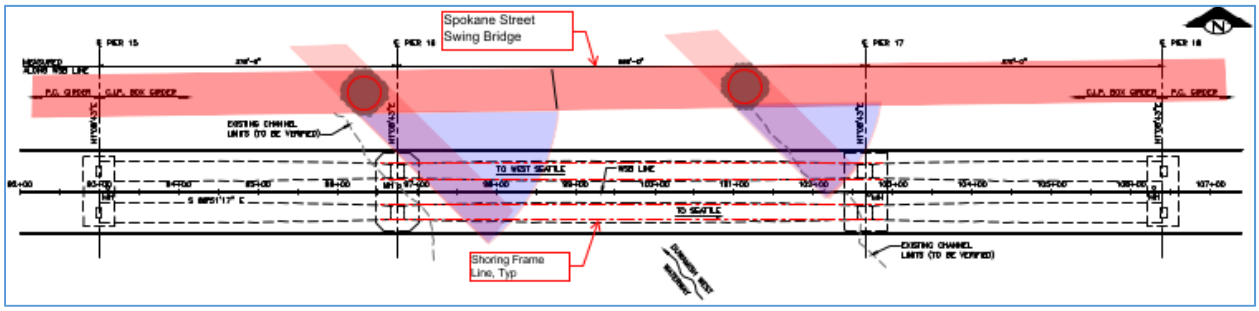

Figure 1: Envelope of Spokane Street Swing Bridge Under WSHB (Plan View)

Other limiting site constraints for placing new bridge foundations include:

- Existing battered pile extents (East and West)
- BNSF tracks (Most severe limitations East)
- Existing ground-level streets (East and West)
- Waterway and shoreline (East and West)

In looking at possible places to put new foundations near the existing main piers (16 and 17), it became apparent that the above listed constraints make it very difficult to place a foundation on the east side of the waterway, east of existing pier 17. The existing pier 17 battered pile foundations, shown in figure 2 below, are approximately 155 feet wide at their base and, assuming the same footprint or larger is required for a new foundation, there is not enough space to fit a new foundation between existing pier 17 and the BNSF railroad tracks.

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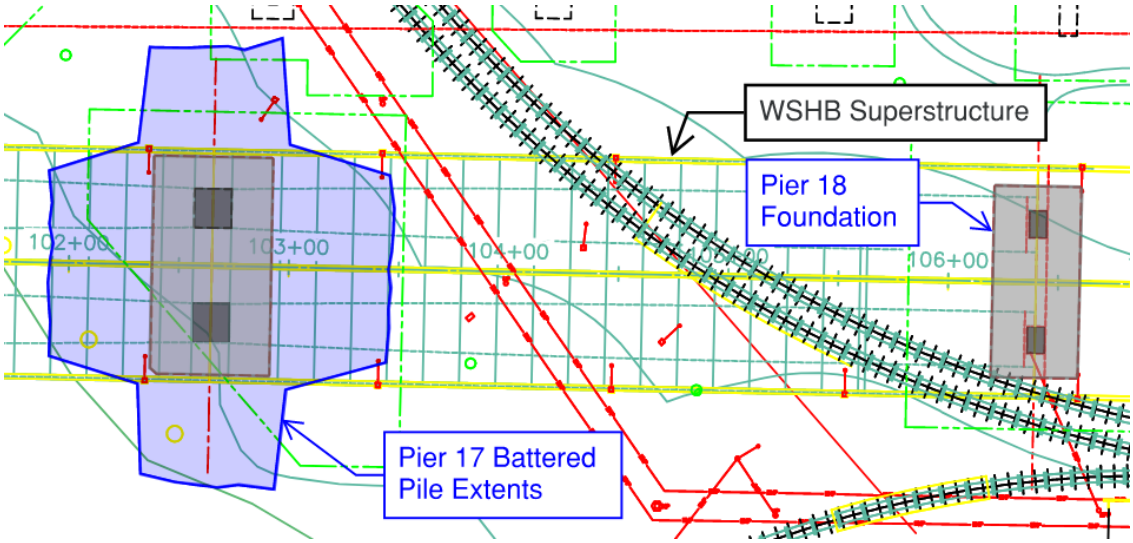

Figure 2: Site constraints (East of waterway)

The only feasible place to place a new foundation without realignment of the BNSF railroad tracks is shown below in figure 3, where isolated foundations are placed on either side of the tracks about 240 feet east of the existing pier 17.

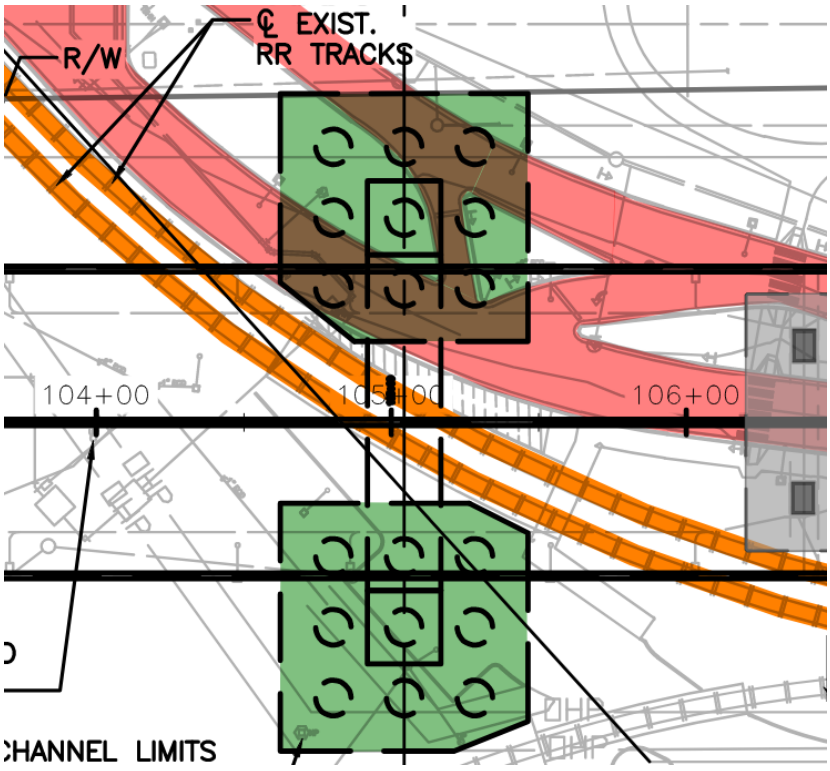

Figure 3: Conceptual foundation locations (East of waterway)

This foundation location, while potentially feasible, has many issues:

- Greatly increases the main span length
- Would require excavation across the entire width of SW Manning Street, a major issue for harbor island traffic
- Clipping corners of foundations to avoid conflicts with railroad
- Excavations right next to BNSF railroad tracks

If this foundation system was able to be built, with the above listed issues resolved, figures 4 and 5 below show examples of bridge types which could be used to bridge the greatly increased span:

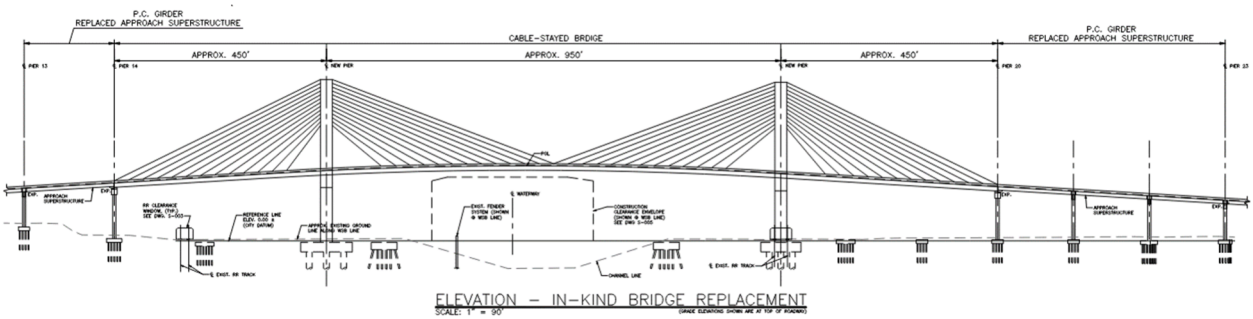

Figure 4: 950-ft main span cable-stayed bridge

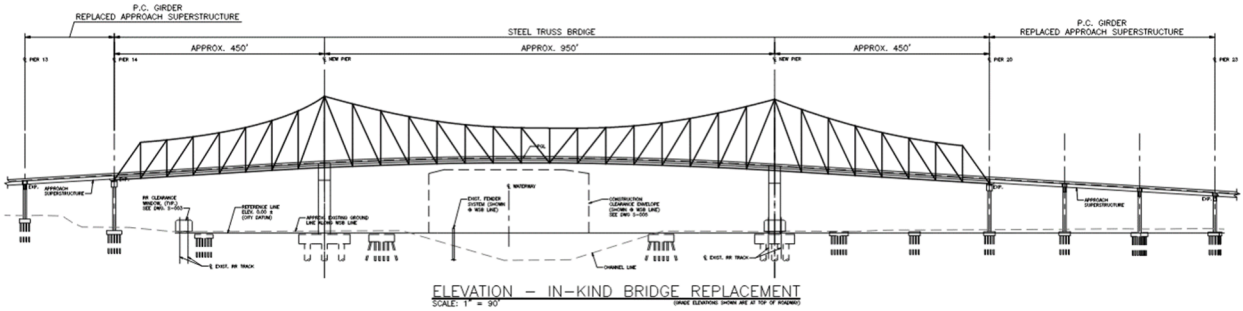

Figure 5: 950-ft main span steel through truss bridge

Also considered, but not shown, is a steel tied arch bridge. While advantageous because it does not require long back-spans like the previous two options, 950 feet is outside of the typical span length where steel arch spans are economical options. In addition, the horizontal tie portion of such a bridge would present challenges with maintaining the existing waterway clearance as the tie does not follow the same arched profile as the bottom of the existing box girder superstructure.

Partial Reuse of Existing Piers

Due to the previously listed issues with a new foundation east of the waterway, the possibility of reusing the existing Pier 17 was explored. If instead of two cable-stayed pylon piers were built as shown in figure 4, a single, larger pylon pier was built, it may be feasible to use the existing pier

17 (likely with retrofits) as a landing point for the cantilever end of a cable-stayed span. Figure 6 below illustrates this concept. The larger pylon pier west of the existing pier 16 would necessarily larger than that required for the shorter cable-stayed bridge option shown in figure 4, but the location shown has very few limiting constraints and is the ideal location for new foundations.

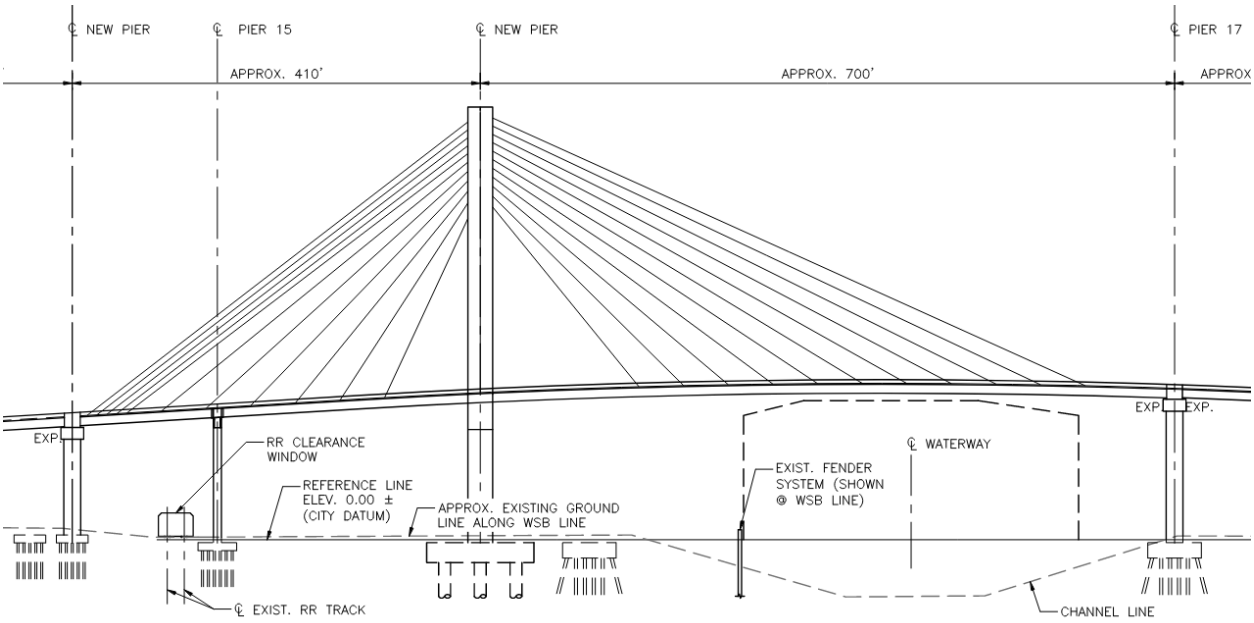

Figure 6: Alternative 5 Cable-stayed Bridge

The cable-stayed bridge shown in figure 6 above is also advantageous because it minimizes the need for approach span replacement. On the west side, similar to the cable-stayed option shown in figure 4, approach spans are replaced back to pier 13. However, where the option shown in figure 4 requires some east side approach spans to be replaced, this figure 6 option only necessitates replacing the existing Pier 17-18 box girder span with two spans of precast girders.

Precast girder spans are significantly cheaper to construct than cable-stayed spans, so the reduced length of cable-stayed bridge (approximately 1110 feet) of the figure 6 bridge is the apparent low cost compared to the 1850 feet of cable-stayed bridge for the bridge shown in figure 4.

Alternative 5 considerations for future study

If a cable-stayed bridge as shown in figure 6 is to be further pursued, the following should be further considered:

- Locations for tie-down piers of back-span (shown as new pier and Pier 15)
- Height restrictions imposed by FAA
- Feasibility of reuse of existing piers, especially Pier 17

Whether or not a cable-stayed bridge or any of the other bridge types presented here are chosen for replacement, any replacement bridge will have to carefully balance the needs of many stakeholders, including but not limited to:

- Environmental (in-water work)
- FAA (overall structure height)
- BNSF (foundation work around railroad)
- USCG (navigable waterway height)
- Port of Seattle (access to harbor island during and after construction)

Figure 7: Examples of stakeholders

APPENDIX L

PRELIMINARY TUNNEL CONCEPT MEMORANDUM

Preliminary Tunnel Concept Development

West Seattle High-Rise Bridge (WSHB) Repair and Strengthening

Prepared for:

City of Seattle

Prepared by:

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October 6, 2020

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Preliminary Tunnel Concept Development
West Seattle High-Rise Bridge (WSHB) Repair and Strengthening

1. Introduction

1.1. Project Overview

The West Seattle High-Rise Bridge (WSHB; Bridge No. 131) was originally brought into service in 1984 and serves as one of the primary east-west throughways in the City of Seattle, carrying thousands of vehicles per day across the Duwamish River. The bridge has been monitored and inspected throughout its service life, including regular monitoring of cracks in the main-span superstructure that were first discovered in 2013. Preliminary assessments of structural integrity and capacity have been conducted since the cracking was first noted in 2013; however, recent inspections indicate that the apparent distress, evidenced by crack growth, has been increasing. In light of the recent inspection findings, WSP USA, Inc. (WSP) was contracted by the City of Seattle (City) to perform a structural evaluation and load rating of the WSHB between Piers 15 and 18 with consideration of the existing conditions. Following this structural evaluation, the City expanded the scope of study to include a preliminary seismic assessment as well as concept development alternatives, including the following: superstructure strengthening concepts, bridge replacement concepts and a preliminary tunnel concept.

1.2. Purpose and Scope

This report has been prepared to document the development of a preliminary tunnel concept, as described in Section 1.1, and to feed into a cost-benefit analysis being conducted to evaluate repair and replacement alternatives. The scope of work associated with this exercise included the following preliminary activities:

- Development of a concept level tunnel alignment, profile and cross section;
- Documentation of the preliminary bases of the concept design, including reference geotechnical and geological conditions, structural design requirements, fire and life safety requirements, civil design requirements and constraints, and local and regional facility operation and maintenance requirements (as available);
- Estimation of dredging and excavation volumes as well as a summary of construction considerations, including potential excavation support and/or stabilization measures;
- Review of potential casting facilities and haul routes for immersed segments; and
- Development of an order-of-magnitude cost estimate, including identification of risk items and items recommended for further study.

1.3. Assumptions and Limitations

As noted in the scope of work, the development of an immersed tunnel concept is, at this stage, preliminary. In order to advance the concept design without reference to detailed calculations or studies, general assumptions were adopted that are consistent with experience and practice, conservative in nature, explicit in identification and, as required, specifically communicated by

the City. Assumptions and limitations particular to analytical frameworks adopted on a discipline-by-discipline basis are identified in the relevant report sections, and where risks may be associated with these assumptions and/or limitations they are identified in Section 12 (Risk). Assumptions specifically communicated by the City are, in summary form, provided below:

- The preliminary tunnel concept includes an immersed tunnel crossing under the Duwamish [West] Waterway, daylighting on Harbor Island to the east and Delridge to the west;
- The design of civil tie-ins with existing facilities is beyond the scope of this study;
- Any role and inclusion of rail facilities in the tunnel is purely speculative and is provided for informational use only;
- Known high level risks and cost drivers will not be specifically quantified; however, they will be identified, and cost allowances will be provided for them based on recent, relevant experience; and
- Aside from the tunnel, specific project elements will not be quantified; however, they will be identified, and cost allowances will be provided for them based on recent, relevant experience.

2. Overview of Immersed Tunnels

2.1. A Brief History

Knowledge of the fundamental concepts underlying the design, construction and operation of immersed tunnels has existed for several centuries, but it wasn't until 1910 that the first immersed transit tunnel was completed across the Detroit River. The decades following included an early flourishing of the technology, with immersed tunnels constructed in the United States and Europe, including the Holland Tunnel in New York (1927) and the Maastunnel in Rotterdam (1942). The technology eventually migrated to the Pacific Coast of North America, with the George Massey Tunnel in Vancouver (1959) and Bay Area Rapid Transit's Transbay Tube in San Francisco (1973). Recent decades have seen a resurgence of interest in immersed tunnels, ranging from minor channel crossings like the Tomassentunnel in Rotterdam (2002) to major recent and upcoming undertakings like the Elizabeth River Tunnels Project in Virginia (2017) and the Fehmarnbelt Tunnel between Denmark and Germany (construction is scheduled to *start* in early 2021).

2.2. Suitable Settings

Though immersed tunnels have been constructed around the world and the range of technical approaches are well understood, the technology is limited in its breadth of application. Specifically, immersed tunnels are best suited to low energy aquatic environments that are shallow in depth and possess sufficient sediment depths to allow for gentle entry and exit grades and ease of dredging/excavation. Immersed tunnels may prove economically viable in settings where ground conditions prove challenging for support of long-span bridge structures and the shallow depth requirement allows for a shorter overall length compared to a bored tunnel option. Immersed tunnels are typically not used for crossings where specially sensitive habitats or contaminated sediments are encountered at the crossing, as habitat protections may

prove too onerous or unfeasible, and mitigation measures and disposal of contaminants may increase the cost to such a level that an immersed tunnel is no longer a competitive solution.

2.3. Construction Considerations

2.3.1. Unit Fabrication and Delivery

The fabrication of immersed tunnel segments varies based on the immersed tunnel type, with differing fabrication requirements for steel shell type segments versus precast concrete segments. Precast concrete segments, which are becoming the most frequently used type, are often fabricated at casting basins developed specifically for the project, but when existing dry-dock facilities are available in the region and can be utilized as-is or with some minor modifications then these may prove more economical. The tunnel segments are constructed in the dry and sealed, using bulkheads slightly set back from the end of the segment, prior to slowly flooding the basin/dock. After the basin/dock has been flooded, the gate is opened to the adjacent navigational channel and the tunnel segment is floated to the construction site.

The delivery of the tunnel segments requires coordination with private and public entities in the region to ensure navigation requirements are adhered to. Additionally, depending on the location of the casting facility and the construction site, seasonal constraints may need to be considered, such as ice, high seas, etc. Due to the challenges associated with transport of immersed tunnel segments, and in particular concrete immersed tunnel segments, the Federal Highway Administration noted that “concrete elements are usually cast in a basin constructed close to the site.” (FHWA, 2009) For instance, the Fort Point Channel tunnel in Boston was cast in an on-site facility while the Midtown Tunnel in Virginia was cast in Baltimore, Maryland, nearly 200 miles from the project site.

2.3.2. Dredging and Sediments

Prior to placement of immersed tunnel segments, a trapezoidal channel is typically dredged, with the base of the trapezoid wider than the base of the immersed tunnel segment and side slopes laid back to prevent excessive sloughing or raveling. Some of the dredged materials are typically reused during the final stage of backfilling over the immersed tunnel; however, much of the material is replaced by engineered materials designed to protect the tunnel from scour and potential impacts (i.e. dropped anchor) as well as to provide tunnel support both during construction and during a potential seismic event. Dredging costs vary from project to project, but they can be expected to increase dramatically if contaminated sediments are encountered.

2.3.3. Unit Placement and Connection

After dredging is completed a levelling layer, typically gravel, is placed and screeded to provide a level surface for immersed tunnel segments. The immersed tunnel segments are then carefully lowered, as ballast is filled within the segment, and adjustments to line and grade using one of several methods that have been adopted, though adjustable jacks are amongst the most common. Once the immersed tunnel segments are aligned and set at the proper grade, any void space between the base of the segment and the levelling layer may be filled using grout bags, sand jetting or some other method. When these phases are complete and an initial seal is established, the water within the sealed zone between segments is pumped out to allow personnel access for construction of a final seal and structural connections.

2.3.4. Cut-and-Cover and Open Cut Connections

The immersed tunnel segments are connected the to the above ground roadway network via cut-and-cover and open cut connections at either end. Construction of these segments are done in the dry and typically require watertight excavation support and dewatering systems. Depending on the suitability of ground conditions, the excavation support may consist of anchored and/or braced sheetpile walls for smaller immersed tunnel applications or more robust systems, such as anchored or braced slurry walls or secant pile walls. Because much of the work to construct the cut-and-cover and open cut segments is often completed below the groundwater table, it is important that water intrusion is kept to a minimum. Ground improvement measures may be incorporated to decrease the potential for seepage through the base of the excavation and to provide long-term support for the constructed tunnel segments.

3. Topography and Geology

3.1. Regional Setting

The proposed tunnel site is located near the southern terminus of Harbor Island, which was created by infilling historical tidal flats at the confluence of the Duwamish River with Elliott Bay within the Duwamish Valley. The Duwamish Valley resides in the greater Puget Lowland, a topographic and structural depression bounded by the Olympic Mountains to the west and the Cascade Mountains to the east. The region is characterized by a dynamic geologic history, including cycles of glacial advance and retreat as well as local and regional seismic events.

A detailed seismic assessment was not conducted for this conceptual stage of the project; however, hazard mapping and recent project experience indicate that the seismic hazard in the vicinity of the site is primarily derived from two sources: the Seattle Fault Zone and the Cascadia Subduction Zone. The site is located within the mapped northern boundary of the Seattle Fault Zone, an east-trending zone of reverse faulting, which is considered capable of producing a M7.0 event. The Cascadia Subduction Zone is a convergent plate boundary spanning from Northern California to Southern British Columbia that dips from west to east as the Explorer, Juan de Fuca and Gorda Plates are subducted under the North American Plate. The Cascadia Subduction Zone is thrust fault considered capable of producing a M9.0 or greater mega-thrust event or smaller, more frequent intraslab events like the M6.8 Nisqually earthquake.

3.2. Site Topography

3.2.1. Surface Topography

Harbor Island is bounded on the east and west by branches of the Duwamish Waterway, a dredged channel conveying water to Elliott Bay to the north. The proposed tunnel alignments would cross the West Waterway in an east-west orientation, with portals developed on either side of the waterway. Surface topography on Harbor Island and east of the East Waterway is relatively flat, consistent with the historical infilling of tidal flats in a bay margin. The topography west of the West Waterway is also relatively flat aside from Puget Ridge, which is located approximately 700 feet west of the West Waterway (along the alignment of the West Seattle High-Rise Bridge [WSHB]) and rises to an elevation of about 150 feet. The northern terminus of

Puget Ridge is known as Pigeon Point, which roughly lines up with the current alignment of the WSHB.

3.2.2. Bathymetry

Locally, the West Waterway is approximately 400 feet in width as measured perpendicular to its dominant orientation, and approximately 500 feet as measured along the axis of the proposed tunnel alignment. The West Waterway is a regularly used shipping channel that requires frequent dredging due to the accumulation of sediment that is carried downstream (south to north). The maximum, existing channel depth in the vicinity of the proposed tunnel alignment corresponds to the minimum channel elevation, which was estimated to be approximately 42 feet below Mean Lower Low Water following dredging completed in January 2020[1].

3.3. Geologic Units

The subsurface soils have been divided into four general units: fill, alluvium and fluvial-marine deposits, recent lacustrine deposits and glacially overridden deposits. The following subsections provide a brief description of each unit.

3.3.1. Fill

The fill materials include loose to medium dense, clean to silty, sand to gravel with some wood and other debris, consistent with the extensive historical development on the island, and frequent, local modifications to facility use type and operation. Fill materials used to create Harbor Island were hydraulically conveyed from Jackson Hill and Dearborn Street during regrading operations as well as dredged from the Duwamish Channel, and can be expected to be uncompacted to poorly compacted based on the identified methods of placement.

3.3.2. Alluvial / Fluvial-Marine Deposits

The alluvial and fluvial-marine deposits consist of loose to very dense, clean to silty, fine to medium sand with scattered layers of very soft to hard sandy and/or clayey silt. These deposits contain organic materials, such as wood and shell fragments.

3.3.3. Recent Lacustrine Deposits

The recent lacustrine deposits consist of stiff to hard clay and silt and very dense silty sand. These materials were deposited during a period of glacial retreat following peak glacial advance and prior to subsequent sea-level rise. These materials are typically not highly over-consolidated like older fine-grained deposits that have been glacially overridden.

[1] US Army Corps of Engineers (Seattle District), 2020. Seattle Harbor, Duwamish Waterway, E-12-2-1-190, Sheet No. 1 of 6. Survey after dredging on January 21, 2020.

3.3.4. Glacial Deposits

All glacial deposits have been overridden by glaciers and are typically very dense and hard sand, silt and clay. These materials are a mixture of reworked sediments, lacustrine sediments and till and till-like sediments that are frequently interbedded.

3.4. Geologic Hazards

A high-level overview of regional seismicity and faulting is provided in Section 3.1; however, a detailed geologic hazard review was not included as part of this preliminary study. In general, a hazard review would include, at a minimum, assessments of the following:

- Seismicity and Faulting;
- Tsunami and Inundation Hazards;
- Liquefaction/Cyclic Strength Loss;
- Marine Slope Stability; and
- Contaminated Sediments.

4. Alignment – Horizontal and Vertical

4.1. Horizontal Alignment

Due to the potential range of impacts initially identified for tunnel alignments north and south of the WSHB, two tunnel concept locations were considered for continuing discussion in this report.

Allowance has been made for light rail tracks in the mainline cross section; however, the origin, destination, east and west tie-ins, and even if a rail line would be proposed in the mainline tunnel, is yet to be determined.

The design criteria for the respective alignments allow for approximately design speeds of 55 mph for trains (Sound Transit Design Criteria Manual [STDCM], Feb. 2020, Section 4.4.1) and approximately 50 mph for vehicles, with horizontal radii ranging between approximately 2,000 and 3,000 foot. The vehicle posted speed on the existing WSHB is 45mph.

The south alignment runs parallel to and approximately 70 feet south of the WSHB. (see Figures A-2 and A-3 in Attachment A).

The south alignment would impact the existing rail line and bridge south of the WSHB and require both its realignment and a new bridge structure to span the West Waterway. This alignment would tie in to the existing mainline to the west along a steep hillside (locally known as Pigeon Point) near the Delridge Way SW vicinity, and in the vicinity of SR99 to the east.

The north alignment runs mostly parallel to and approximately 200 feet north of the WSHB, 100 feet north of the existing swing bridge foundations, and directly north of the 250-foot-high electrical line towers. An alignment closer to the swing bridge was not advanced due to the risk

of impacts to swing bridge foundation elements. The north alignment would tie into the existing mainline north of Pigeon Point to the west and in the vicinity of SR99 to the east.

4.2. Vertical Alignment

The maximum mainline grade considered is 6%, as this is a typical maximum grade for extended rail lengths (STDCM Table 4-4) and is the same as the existing WSHB aerial structure. The 6% grades outside the waterway are connected by a 1,200-foot vertical curve. The vertical curve length is determined based on STDCM Table 4-8 where LVC = 100A (A being the algebraic difference in grades).

If the train design speed were reduced, or if the trains were eliminated from the cross section, this curve length could likely be considerably reduced. For example, if the vertical profile design was based on vehicular criteria a 650-foot curve would be appropriate instead of the 1,200-foot curve noted above. This would shorten the overall tunnel length by approximately 600 feet and allow the tunnel's open cut sections to shift in, reducing surface impacts and lessening overall construction quantities.

The vertical layout of the south alignment is based on providing 23.5-foot clearance (WSDOT Design Manual Exhibit 720-2) over the railroad tracks crossing the East Waterway, a minimum tunnel structure clearance of 45 feet under MLLW elevation in the West Waterway navigation channel, and then rising at the maximum 6% grade to the west. There is a possibility that some amount of fill material may be needed in order to cross the East Waterway. Further design and analysis will be required to determine impacts in this area.

The vertical layout of the north alignment does not contend with the railroad tracks crossing the East Waterway; however, it still adheres to the same maximum tunnel structure elevation in the West Waterway navigation channel of 45 feet under MLLW, will likely require relocation of rail lines and vehicle access to Harbor Island and T-18. This tunnel concept also rises with the maximum 6% grade to both the east and the west.

The vertical curve under the West Waterway also allows for the change in roadway profile elevation and roadway superelevation along the curve to be designed into the tunnel ballast so that vertical curves are not required to be cast into the structural elements of the tunnel box.

4.3. Tie-In Considerations

After crossing under the West Waterway, the mainline is also assumed to maintain similar vehicular connections to the local street network in the Delridge area to the west and in the vicinity or SR 99 to the east. It is expected that significant reconstruction of ramps and intersections will be required. Rail connections are not known at this time and will need further consideration beyond the scope of this report.

4.3.1. Street Tie-Ins

The Delridge Way SW ramps are assumed to connect similarly to the existing connections, except coming up from below grade instead of down from above grade. The intersection at Chelan Ave SW and SW Marginal Place would likely need to be reconstructed and reconfigured to accept the new open cut ramp section required approaching the intersection. The east side

tie in will need to clear rail tracks while crossing over the East Duwamish waterway, and then reconnect ramps to/from SR99 and E. Marginal Way S.

4.3.2. Rail Tie-Ins

A space allowance in the mainline submerged tunnel has been made for two railway passages, however the tie-ins to the east and west are unknown. A rail alignment would likely need to separate from the mainline vehicle tunnel immediately after the submerged crossing to allow vehicle ramp connections to be made with the local street network on the west, and SR99 on the east.

5. Right-of-Way Impacts

Temporary and permanent easements, and permanent acquisitions will be required to construct the facility, and to allow for permanent operation and maintenance of the final roadway network. Figure B-1 in Attachment B depicts the two concepts in relation to right of way and parcel lines.

5.1. Easements

An easement is a vested or acquired right to use land for a specific purpose. It is an interest in land owned by another that entitles the easement holder to a specific, limited use. Easements can be either temporary or permanent. Temporary easements typically are related to construction activities for a designated and agreed upon length of time.

5.1.1. Temporary Construction Easements

A temporary construction easement is typically an easement granted by a property owner allowing the contractor right of entry over a designated area of their property for the purpose of constructing a project. It is anticipated that temporary construction easements extending up to 50 feet on both the north and south side of the below grade alignments may be required for tunnel construction, not providing for contractor staging and stockpiling requirements.

5.1.2. Permanent Easements

A permanent easement is typically an easement granted by a property owner allowing right of entry over a designated area of their property for the purpose of accessing built infrastructure, whether for inspection or operations and maintenance. It is anticipated that permanent easements will be required in the vicinity of the open-cut sections near the portals and the cut and cover sections, both for access into the tunnels as well as for access to operational facilities above the tunnel portals.

5.2. Impacts

Detailed right-of-way impacts and coordination requirements with property owners were not determined as part of this study. Any further development of the concepts identified in this study should define the particular implications of these impacts and requirements, including costs, operational impacts (i.e. Port), and temporary and permanent easement agreements.

6. Tunnel Spaceproofing

Since certain elements of the fire protection and life safety systems can have significant spatial requirements that can impact the tunnel structure, early definition of feasible concepts for these systems is necessary to allow confident development of the tunnel structure(s) to proceed. Fire protection and life safety system elements having spatial impacts include:

- Emergency ventilation system (roadway/trainway)
- Fixed fire suppression system (roadway)
- Tunnel drainage system (roadway)
- Emergency egress (roadway/trainway)

Feasible concepts for these systems have been developed to a point to provide assurance that the tunnel structure, as proposed, has been adequately space-proofed to accommodate them. Descriptions of these specific systems are provided in more detail in Section 7.

Other fire protection and life safety systems that will need to be incorporated within the defined limits of the structure including the following:

- Traffic control and monitoring (roadway)
- Roadway/trainway lighting
- Fire alarm and detection (roadway/trainway)
- Fire Protection Standpipe (roadway/trainway)
- Electrical power distribution (roadway/trainway)
- Emergency communications (roadway/trainway)
- Structural fire protection (roadway/trainway)
- Exit and other special signage (roadway/trainway)
- Intrusion detection/access control (roadway/trainway)
- Emissions monitoring (roadway)

6.1. Vehicular Requirements

The WSHB is owned, operated, and maintained by the City of Seattle. It has a roadway classification of Principal Arterial and listed as a “Major Truck Street and Transit Way” in the City of Seattle Transportation Strategic Plan (2005). Several King County Metro bus routes typically travel the WSHB including the RapidRide C Line, 21 and 21X, 37, 50, 55, 56, 57, 116X, 118X, 119X, 120 and 125.

Due to the Limited Access condition created by the grade separated structure, high posted and operating speed, and the design and context of the proposed tunnel, the roadway design falls most appropriately under the guidance of the WSDOT Design Manual or the AASHTO *A Policy on Geometric Design of Highways and Streets*. Lane widths for this highway-type facility are

typically 11-12 feet (WSDOT DM Exhibit 1231-1). Due to the high truck and bus volumes, higher speed, and enclosed structure, it is recommended that a minimum of 11-foot lanes be used, with 12 feet being desirable.

6.1.1. Shoulder Width Considerations

Shoulder widths for this highway-type facility can vary significantly, but for new facilities they typically vary from 2-10 feet on the left and right (WSDOT Exhibit 1239-1). Considerations given to specifying widths should include emergency pullovers, minimum distance to barriers, part-time/peak/transit use potential, maintenance operations, law enforcement use, emergency services and incident response use, slow vehicle shoulder driving, and sight distance requirements, drainage structure locations, among other factors.

6.1.2. Clearance Envelope

The vehicular clearance envelope should take into consideration all potential vehicles traversing the entire width of the traveled way. A clearance diagram should at a minimum include static conditions, dynamic conditions, construction tolerances, and tunnel equipment requirements (such as ventilation, lighting, signing, fire/life/safety, etc). Typical vehicle design clearance for new non-freeway bridge structures is 16.5 feet (WSDOT DM Exhibit 720-3).

6.1.3. Ventilation

Longitudinal ventilation using jet fans will likely be the most economical ventilation solution for the roadway and transit bores. It is anticipated that banks of jet fans, constructed into ceiling niches, will be required in the cut-and-cover sections of each bore. Utilizing this concept will allow for the elimination of jet fans within the immersed tube section, thereby allowing for the overall height reduction of the immersed tubes. Preliminary details of the fan layouts are provided on drawings. Refer to Section 7.1.3 for further discussion.

6.1.4. Emergency Egress

Emergency egress will be provided by corridors and doors between each roadway bore. In the event of an emergency, motorists can utilize the egress passageways/doors to move from the incident bore to the adjacent, non-incident bore. The egress doors are fire-rated and have an assisted, sliding opening. Refer to Section 7.1.2 for further information.

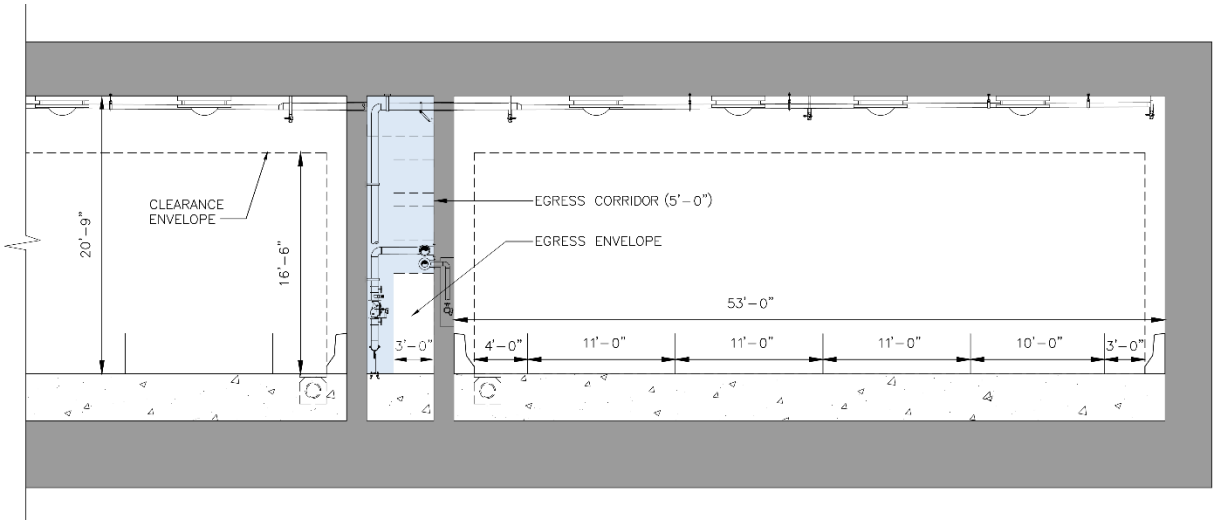

Figure 1 – Egress Corridor in Roadway Section

6.1.5. Other Systems

Other critical systems for tunnel spaceproofing are discussed in Section 7.1. A clear space above the vehicle envelope is required to make allowance for lighting, ITS signs and other devices. A minimum clearance of 3 feet is recommended for these items, with further confirmation required at a design phase depending on the actual placement and layout.

6.2. Rail / Transit Requirements

6.2.1. Ventilation

Similar to the roadway bore, longitudinal ventilation using jet fans will likely be the most economical ventilation solution. It is anticipated that banks of jet fans, constructed into ceiling niches, will be required in the cut-and-cover sections of each bore. Utilizing this concept will allow for the elimination of jet fans within the immersed tube section, thereby allowing for the overall height reduction of the immersed tubes. Preliminary details of the fan layouts are provided on Figure A-6 in Attachment A. Refer to Section 7.2.3 for a further discussion.

6.2.2. Egress

Emergency egress will be provided by corridors and doors between each roadway bore. In the event of an emergency, riders can utilize the raised egress walkway to move to an egress passageway/door that will provide access to the non-incident bore; maximum spacing 800 feet. The egress doors are fire-rated and have an assisted, sliding opening. Refer to Figure 2 for a view of the egress walkway and Section 7.2.2 for additional details.

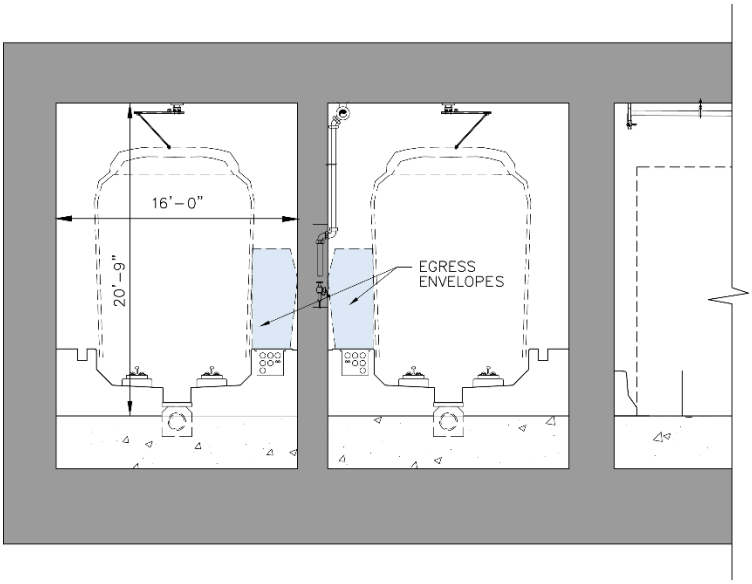

Figure 2 – Egress Walkway in Rail Section

6.2.3. Other Systems

Other critical systems for tunnel spaceproofing are discussed in Section 7.2. Space in the section will be needed for systems such as lighting, a standpipe and hose connections, conduits.

7. Tunnel Systems

7.1. Vehicular Requirements

7.1.1. Code Requirements

The primary national standard for fire protection systems in road tunnels is NFPA 502, Standard for Road Tunnels, Bridges, and Other Limited Access Highways. The City of Seattle Fire Code has adopted NFPA 502 as a code requirement with various amendments. It requires the installation of both fixed fire fighting systems (FFFSs) and standpipe systems in road tunnels. The standard provides performance goals and general guidance for system design. Chapter 33 of the City of Seattle Fire Code identifies requirements for road tunnels based on NFPA 502 2011 edition (with amendments). Some of the headline amendments with respect to this initial concept design phase are summarized as follows:

- NFPA 502:
    - The tunnel, at the length of approximately 3,000 feet, is a category C/D tunnel (cutoff length is 3,280 feet) and all provisions of the standard apply.
    - The Authority Having Jurisdiction is nominated as the fire chief or other designated authority charged with the administration of the fire code.
    - Water-based fire-fighting systems are required for roadways and shall be installed to a minimum hazard classification as Extra Hazard Group 2 (per NFPA 13). If

flammable liquids/hazardous materials are likely to be present, an engineering analysis must be developed and approved by the authority having jurisdiction.

- Fire hose valve connections are required in a tunnel so that no location on the roadway is more than 150 feet from the hydrant.
- An emergency power system is required, and it shall provide power to all major tunnel systems, including the ventilation system.
- Annex material for design fires lists the following guidance: passenger car (10 MW), bus (30 MW), heavy goods truck (200 MW), tanker (300 MW).

The tunnel will need to accommodate a variety of operational systems and features in order to support safe traffic operations and to provide the necessary level of fire protection and life safety, including:

- Emergency egress.
- Tunnel ventilation.
- Fixed firefighting system and fire standpipe system
- Fire detection and emergency communications.
- Tunnel drainage.
- Electrical system.
- Tunnel lighting.
- Traffic control and monitoring system.
- Tunnel finishes and special signage.
- Operations and maintenance.

7.1.2. Egress

The egress strategy from the tunnels will be via the non-incident tunnel bore where cross-passageway doors lead to the adjacent, non-incident bore. Per NFPA 502 2011, Section 7.15.7, the door spacing must be 656 feet or less. A nominal spacing of 600 feet or less is recommended. The egress walkway and doors will need to have a minimum clear width of 3.6 feet. The passageway and doors will need a clear egress height of at least 7.6 feet.

Fire-rated doors are required to separate the egress pathway from the tunnel. Sliding egress doors are typically used to limit the impact of air pressures on door operation and to prevent door encroachment into the roadway space. Suitable emergency signage, lighting, and pressurization of the cross passageways/doors are also required.

Access to the doors would be provided at roadway level from both roadway bores.

7.1.3. Ventilation

Ventilation is required for normal operations (management of vehicle emissions) and emergency operations (management of smoke). A key requirement in NFPA 502 is the provision of tenable

conditions for egress and facilitation of conditions for firefighting. Achieving these goals relies on ventilation, means of egress and fire control. Ventilation is particularly integral with fire-life safety because it is essential to smoke management. Based on the tunnel length and traffic configuration, a longitudinal ventilation system is recommended for this facility.

Portal emissions and achieving air quality compliance in surrounding areas will be critical with a longitudinal system. At a length approximately 3,000 feet, it is unlikely that any special treatment, such as portal exhausts, will be necessary but this should still be checked at a more advanced concept development phase. An ambient air quality analysis of the emissions from the tunnel portals will be necessary with respect to any sensitive receptors in the surrounding areas near to the exit portals. This ambient air quality analysis will need to incorporate the expected tunnel traffic on an hourly basis, the subsequent vehicle emissions, the expected airflow in the tunnel, and the impact of external meteorological conditions. If acceptable air quality cannot be achieved, then ventilation buildings at each portal may be required to exhaust and disperse vitiated air away from sensitive receptors, although it is noted that this is considered an unlikely outcome of the analysis.

A preliminary analysis of the ventilation system performance for smoke management was conducted to determine the necessary system capacity to manage the smoke from a fire. The ventilation concept is based on one-way traffic and directing smoke downstream of the fire site (vehicles downstream drive out ahead of the fire), refer to Figure 3.

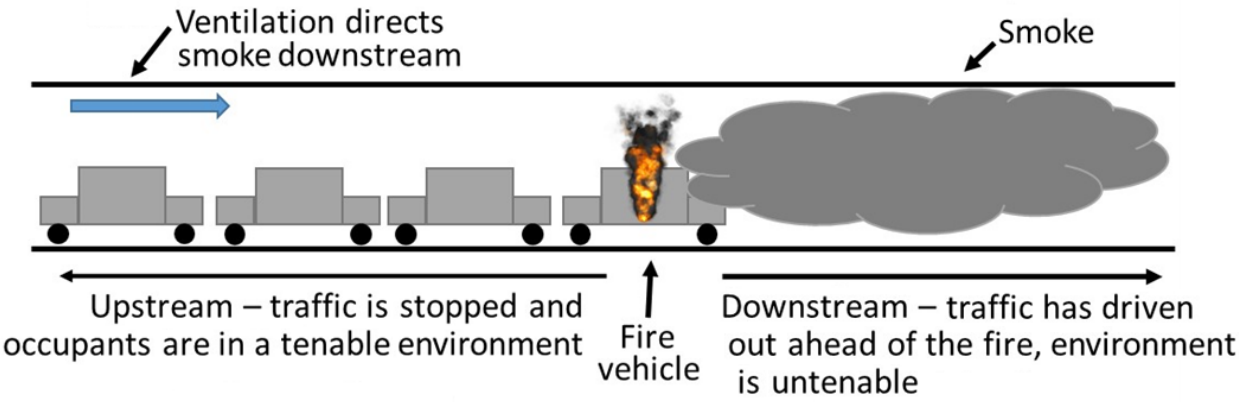

Figure 3 – Longitudinal Ventilation Concept

Jet fans were placed in the tunnel to generate the longitudinal airflow. Jet fans are going to be a key component for space proofing due to their large dimension relative to other tunnel systems. Fans were placed in the portal area on the entry side, in the cut-and-cover segments, in order to avoid placing fans in the immersed tunnel sections. A bank of eight fans was located near to the entry portal. The general set up in longitudinal profile is shown in Figure 4.

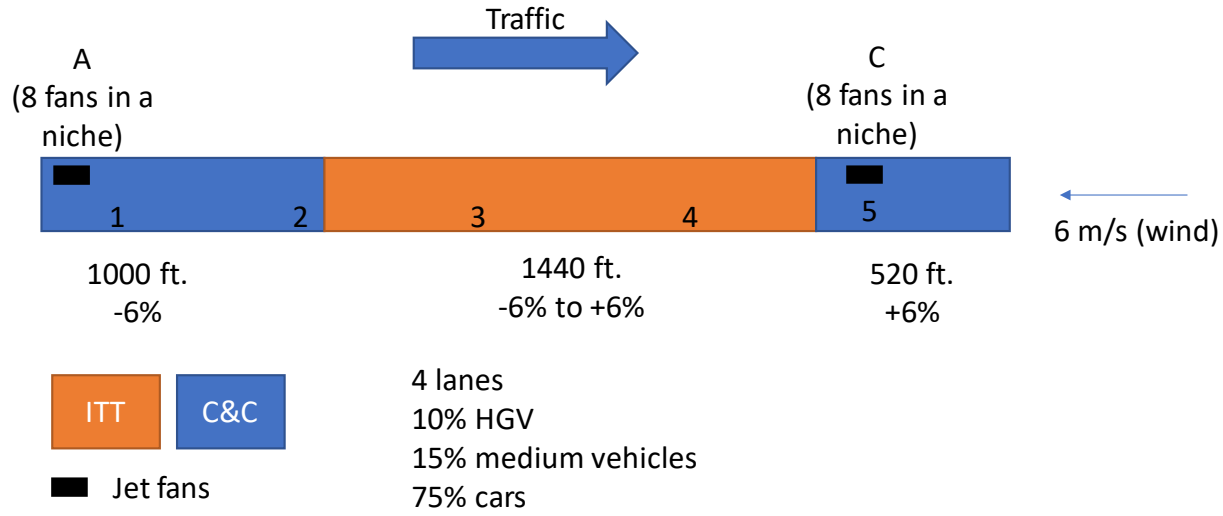

Figure 4 – Jet Fan Longitudinal Layout *(Without fans installed in the immersed tunnel section)*

The results of the analysis showed that the ventilation system could manage a 100 MW fire heat release rate (FHRR) at all locations except near the entry. A 100 MW FHRR corresponds to a heavy goods vehicle fire (NFPA 502 notes the range of the FHRR is 20 MW to 200 MW and the City of Seattle Fire Code only recognizes 200 MW). When jet fan banks were located near to the fire they were not operated, assuming the fire would damage the fans. The City of Seattle Fire Code modifies NFPA 502 to state that the heavy goods vehicle FHRR is 200 MW. Analysis was conducted on the fan layout proposed here to determine the maximum FHRR that could be managed with the fan arrangement proposed. Refer to Table 1 for the results for a 100 MW fire.

Table 1 – *Analysis for Road Tunnel Ventilation (Base-Case)*

Fire Location	FHRR (MW)	Operational Jet Fan Banks	V (feet per minute) (DW-01-09)	V_c (feet per minute) (NFPA 502 217)
1	100 (*95)	A (7 of 8), C	669*	676
2	100	A (7 of 8), C	855	676
3	100	A (7 of 8), C	844	592
4	100	A (7 of 8)	832	566
5	100	A (7 of 8)	691	566

To manage a larger FHRR than those provided in Table 1 it would be necessary to install additional jet fans in the tunnel. Another analysis was conducted where fans were installed in niches at three locations along the length with eight fans per niche; refer to Figure 5. Even with this many additional fans, the system still cannot manage a 200 MW FHRR at all locations throughout the tunnel; refer to Table 2. The arrangement depicted here is at the limits of what can practically be done with a ventilation system; fans cannot be placed closer together as their efficiency will drop dramatically and the niches locally increase cross sectional area of the immersed tunnel segments. An alternative measure would be to rely on the fixed fire-fighting system to reduce the FHRR to 100 MW and utilize fans at the cut-and-cover areas only. The

FHRR reduction with FFFS operation is reasonable based on full-scale testing; however, it would be subject to approval by the Authority Having Jurisdiction.

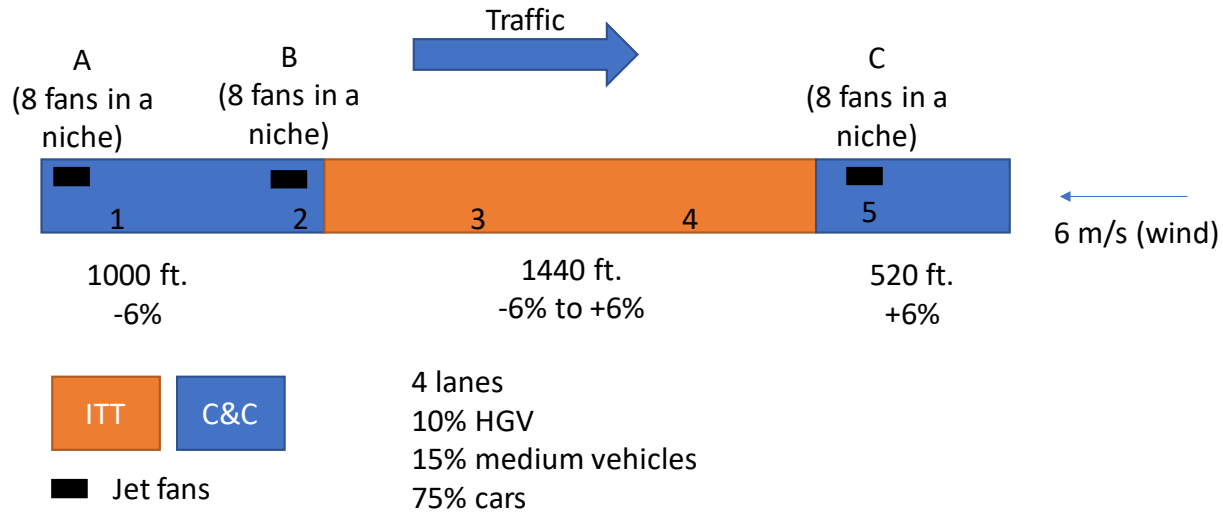

Figure 5 – Jet Fan Longitudinal Layout *(With fans installed in the immersed tunnel section)*

***Table 2** – Analysis for Road Tunnel Ventilation (32 Fans Installed)*

Fire Location	FHRR (MW)	Operational Jet Fan Banks	V (feet per minute) (DW-01-10)	V_c (feet per minute) (NFPA 502 217)
1	100 (*95)	A (7 of 8), C	669*	676
2	100	A (7 of 8), C	855	676
3	300	A (7 of 8), B, C	939	696
4	300	A (7 of 8), B	931	663
5	300	A (7 of 8), B	789	663

Lastly, the analysis herein has assumed that tanker vehicles do not use the tunnel. The FHRR from one of these incidents could be as large as 300 MW and there is less evidence with a gasoline fire that the FFFS would reduce the FHRR. If the tanker fire were to be made a design case, then it would be necessary to make allowances for fans in the immersed tunnel segments also, though even with the addition of fans in the immersed tunnel segments the ventilation system would not be able to manage to full FHRR throughout.

The fans assumed in the analysis had a thrust of 365 lbf per fan, a motor power 75 hp, an outer diameter of 4.8 feet and a length of 20 feet. The fans would need additional vertical clearance for installation; an allowance of approximately 6 feet has been provided in this study and is recommended. Figure A-6 in Attachment A depicts the typical fan layout in the tunnel.

7.1.4. Fire Protection Systems

NFPA 502 references various other NFPA standards for the specific system requirements pertaining to fire protection systems. The primary of these is NFPA 13 which describes system

requirements including material properties, hydraulic performance, functional provisions for fire department interaction, system approval and occupancy classification etc. The requirements of these standards must be incorporated into the tunnel design.

Roadway Fixed Fire Fighting System Configuration

The FFFS is an automatic deluge sprinkler type providing water spray capability over predetermined roadway zones and covering the entire roadway surface area. Delivery of water to the roadway is controlled by deluge valves for each zone. The valves are located in the tunnel utility corridor. In response to a fire incident within the roadway, two adjacent zones are activated to discharge water onto the fire area. The FFFS is supplied with a permanent water supply from the local water utility providing sufficient flow and pressure to meet system demand. It is not anticipated that fire pumps or water storage tanks will be required. The FFFS is also provided with fire department connections (FDCs) to allow the responding fire department to boost system pressure if needed. To the extent practical, the system will incorporate standard sprinkler system components, piping, valves, and appurtenances. Materials incorporated into the system will be UL/FM listed-approved. If exceptions are required, they will be submitted to the AHJ for approval. The system will be designed with careful consideration of maintenance requirements, longevity and ease of access for testing and inspection.

Standpipe System Configuration

A fire standpipe system will be provided to supply fire hose valve outlets located in cabinets throughout both roadways. The standpipe system is supplied with a permanent water supply pressurized from the local utility. Per code, system demand is 1,000 gpm at 130 psi residual pressure. Utility pressure is generally not sufficient to provide required residual pressure at hose valve connections. As such, fire department connections (FDC's) and fire hydrants are provided at both portals allowing the system to be manually pressurized by the responding fire department. In accordance with the standards and Seattle Fire Code requirements, roadway hose valves will be provided at maximum 275-foot intervals. Roadway hose valves are located in hose valve cabinets with fire rated doors and accessible from the roadway.

7.1.5. Drainage

Tunnel drainage systems normally consist of two independent systems: a storm water control system and a tunnel drainage system. Storm water control systems are required at the tunnel portals to intercept storm water flows that accumulate on the open approaches and transition roadways leading into and out of the tunnel. A separate tunnel drainage system, designed to be independent of inflow from sources outside the tunnel, is required to collect and discharge water and effluents generated within the tunnel. These effluent flows result from tunnel washing, use of fire suppression systems, vehicle carryover, and some groundwater seepage. The tunnel drainage system must also be designed and equipped to accommodate a potential fuel spill. System capacity is based on maximum combined FFFS and fire standpipe demand.

The profile of the selected tunnel alignment will dictate the location of the tunnel low point, which is where a drainage pump station will be required. The low point pump station is located in the egress corridor and will serve for both roadway and trainway drainage. To provide adequate volume for sump retention, no tunnel ballast is applied in the area of the pump station. Effluent catch basins spaced at ~75 feet and underground drainage piping are required along both

roadways. A similar arrangement is required for trainways and located in the center of trackways.

The storm water collected at the tunnel portals is considered to be clean and therefore does not require special treatment prior to discharge. However, the tunnel drainage effluent may require some form of pre-treatment prior to discharge depending on local permitting requirements.

7.1.6. Electrical

Electrical systems will be necessary to support safe traffic operation. The required installation methods and performance criteria of these various electrical systems for road tunnel application have been generally defined in within applicable codes and standards including NFPA 502 and the National Electrical Code. The required tunnel electrical systems include:

- Power distribution
- Fire alarm and detection
- Emergency communications
- Security
- Supervisory control and monitoring (SCADA)
- Communication systems

7.1.7. Lighting

The tunnel lighting system purpose is to provide the required illumination so that a motorist can safely navigate and maintain speed while in the tunnel. This objective must be met during daytime, nighttime, and during an emergency. Daylight conditions require high levels of illumination at the entry portal avoiding the “black-hole” effect. Nighttime levels are significantly lower and consistent throughout the tunnel. During an emergency, light levels are to be maintained at the nighttime level to allow for egress. It is recommended to use Light Emitting Diode fixtures throughout the tunnel and egress facilities.

7.1.8. Traffic

Roadway tunnels are required by NFPA 502 to be provided with a means for control of traffic within the tunnel, as well as traffic on the approach roadways leading into the tunnel These systems are necessary to control traffic within the tunnel and/or to prevent vehicles from entering the tunnel in the event of an traffic incident or emergency and also for purposes of tunnel maintenance. The types of traffic control systems and devices likely to be required include:

- Automatic Incident Identification, based on an intelligent, programmable, CCTV video stream
- Closed Circuit Television (CCTV), for general surveillance, would typically be monitored from a dedicated tunnel operations control center.

- Dynamic (Variable) Message Signs are typically provided in the tunnel and tunnel approaches at regular intervals above the travel lanes to display instructions and emergency messages to motorists.
- Lane Use/Control Signals are typically located along the tunnel walls or ceiling, and over the roadway at the tunnel portal approaches, at regular intervals to indicate the status of each travel lane as either opened or closed.
- Over-Height Vehicle Detection/Protection.

7.1.9. Finishes

Tunnel Finishes, which are further described typically coordinate with various roadway and tunnel elements, including lighting, architectural appearance, cleaning, and fireproofing. Fixed signage other than highway signage directs motorists, maintenance workers and first responders to emergency exits, cabinets, standpipe valves and similar elements.

Roadway Structural Thermal Protection

Structural protection systems are to be designed in accordance with the applicable standard, National Fire Protection Association (NFPA) 502 Standard for Road Tunnels, Bridges and Other Limited Access Highways. The extreme temperatures possible in a tunnel fire can do extensive damage to a tunnel's structural elements. This is recognized in NFPA 502 which requires that the structure be protected against progressive structural collapse and if provided with fire proofing, concrete temperatures are not to exceed specified limits. A structural durability analysis is required to determine if those conditions are met. The analysis looks at the heat transferred from the design fire products of combustion through the structure over time. Unless otherwise stipulated, the default input near-ceiling temperatures are described by the Rijkswaterstaat Curve (RWS). The requirement is quite rigorous and can result in the need for some form of temperature mitigation, often the installation of a thermal protective board or other insulation type. The analysis will present predicted ceiling concrete surface and reinforcement depth temperatures and conclude as to whether passive fire protection on roadway ceilings and walls is required. It considers the benefit of the fixed firefighting system in protection of structure and system components from the extreme temperatures in the design fire scenario. The ability of water sprays to mitigate solid fuel fires is well-understood and experience has shown that for heavy goods fuel loads the addition of costly thermal insulation might be able to be avoided. Other factors may play a role in this decision such as tolerance for risk of events occurring that are outside the anticipated design parameters. In the event that liquid fuels tankers are allowed passage through the tunnel and considering the greatly increased fuel load and resulting temperatures, it is probable that structural thermal protection will be required.

7.1.10. Operation and Maintenance

A dedicated and well-planned tunnel operations and maintenance program is necessary to ensure a safe, well maintained, and reliable tunnel facility which maximizes public safety and roadway availability. A tunnel Operations and Maintenance Plan is needed that fully considers the future operations and maintenance needs of the facility and adequately identifies all ancillary facilities, operating systems, infrastructure, staffing, maintenance equipment, and related items necessary to operate and maintain the facility.

Ancillary facilities that will be required to support operation of the tunnel and will include provision of an operation and control center for tunnel operations staff who will be responsible for the operation and monitoring of the mechanical, electrical, and traffic control systems on a 24/7/365 basis.

Maintenance related facilities may include maintenance shops, garage facilities, and other storage spaces to house equipment and spare parts that are needed to maintain the tunnel. Appropriate maintenance requires a mix of personnel including electricians, mechanics/millwrights, and general maintenance staff to maintain the facilities and various systems, support traffic control measures and respond to traffic incidents.

A significant level of planning and coordination is required to operate and maintain a major road tunnel facility. An Operations and Maintenance Plan consists of the various incident and emergency management plans, maintenance management plans, and operational procedures determined to be necessary for safe and efficient operation and maintenance of the tunnel facility.

During the planning and feasibility stage of a major urban road tunnel project it is very important to consider the Operations and Maintenance Plan, so the project design accounts for all of the facilities, infrastructure and other items needed to support the operation of the facility. The development of a Concept of Operations Report serves as the first step to developing the basis of the Operations and Maintenance Plan. The Concept of Operations Report provides a basic understanding of how the facility must function in relation to the overall road network and identifies the individual agencies, entities and other stakeholders dependent on the overall successful operation of the facility. The Concept of Operations Report summarizes the key decisions and operating policies established during the planning and design phases of a road tunnel project and the report also will serve as a basis for the development of the actual operating procedures to be implemented within the Operations Plan portion of the Operations and Maintenance program.

7.2. Transit Requirements

7.2.1. Code Requirements

The primary national standard for fire protection systems in underground trainways is NFPA 130. The City of Seattle Fire Code has adopted NFPA 130 as a code requirement with various amendments. Chapter 33 of the City of Seattle Fire Code identifies requirements for transit systems. The Fire Code requires following NFPA 130 2010 edition (with amendments) for transit. Some of the headline amendments with respect to this initial concept design phase are summarized as follows:

- NFPA 130 (transit):
    - The Authority Having Jurisdiction is nominated as the fire chief or other designated authority charged with the administration of the fire code.
    - Means of egress shall be provided with an unobstructed clear width ranging from: 30 in. at the walking surface, 30 in. at 56 in. above the walking surface, and 30 in. at 80 in. above the walking surface.

  - Ventilation shall address performance with a fan out of service.
  - Emergency ventilation shall be provided with a Class 2, Type 60, Level 1 emergency power supply system.

The transit portions of the tunnel will need to accommodate a variety of operational systems and features within the tunnel in order to support safe traffic and transit operations and to provide the necessary level of fire protection and life safety. The various tunnel systems and features that will be required include:

- Emergency egress.
- Tunnel ventilation.
- Fire protection.
- Tunnel drainage.
- Electrical systems.
- Tunnel lighting.
- Tunnel finishes (fire protection).
- Operations and maintenance.

7.2.2. Egress

Per the Fire Code, egress walkway widths are required as follows:

- Means of egress shall be provided with an unobstructed clear width ranging from: 30 in. at the walking surface, 30 in. at 56 in. above the walking surface, and 30 in. at 80 in. above the walking surface.

Egress is provided by a walkway and a (2-hour fire-rated) dividing wall between the two transit tunnels. NFPA 130 requires exits from the trainways, with a spacing not more than 2,500 feet or cross passages at spacings 800 feet or less. Cross passages are recommended between the two trainways, with a spacing no more than 800 feet. Given the length of 3,200 feet. it would be necessary to have three cross passages along the length. Figure 2 shows the cross section with egress elements identified.

7.2.3. Ventilation

With a length of approximately 3,000 feet, ventilation is required for smoke management per NFPA 130. The design fire for the transit tunnel per Sound Transit is 13.2 MW. Analysis of the ventilation system was conducted based on longitudinal ventilation. Figure 6 shows the fan layout and Table 3 provides results of the analysis. The fans are planned to be installed at each end of the tunnel at cut-and-cover sections in niches. The fans would need to be installed so that they are stationed at approximately 50 feet and 350 feet from each portal. Fans are noted at each end to enable the train direction to be changed. Fan parameters (thrust, size, power) are identical to those analyzed for the road tunnel.

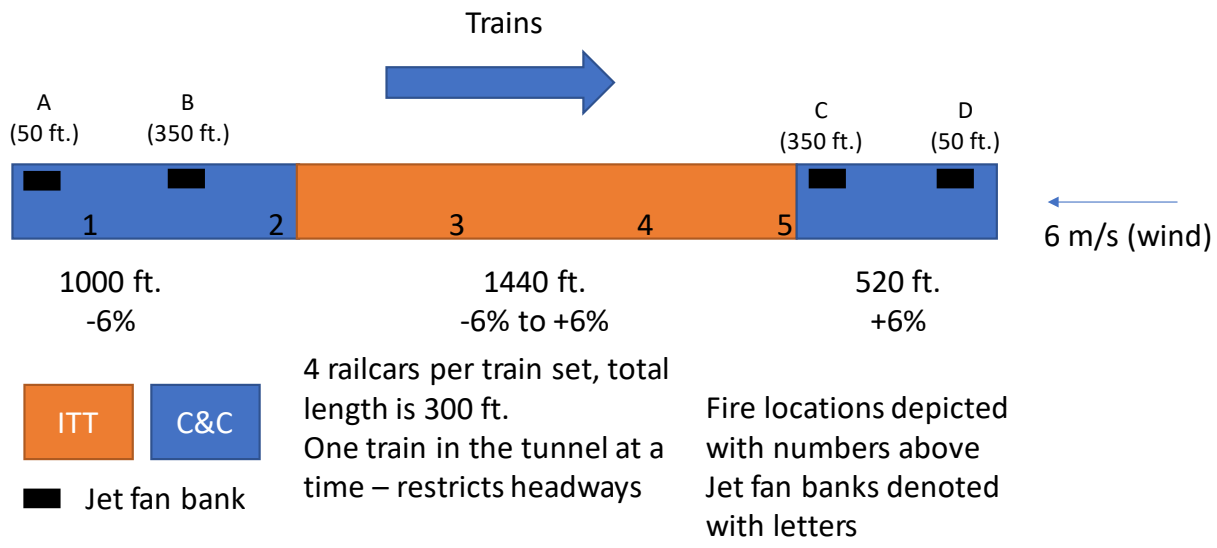

Figure 6 – Fan Layout for the Transit Tunnel

***Table 3** – Analysis for Transit Tunnel Ventilation*

Fire Location	FHRR (MW)	Operational Jet Fan Banks	Q (1,000 cubic feet per minute) (DW-01-12)	Q (1,000 cubic feet per minute) (NFPA 502 217)
1	13.2	A, C	225	118
2	13.2	A, B	212	118
3	13.2	A, B	232	104
4	13.2	A, B	244	99
5	13.2	A, B	235	99

7.2.4. Fire Protection

For fire standpipes, NFPA 14 describes specific system design requirements including material properties, hydraulic performance, functional provisions for fire department interaction, system approval and occupancy classification. The requirements of this standard and other related standards (as referenced in NFPA 130 and the City of Seattle Fire Code) must be incorporated into the tunnel design.

Trainway Standpipe System Configuration

A fire standpipe must be provided to supply fire hose valve outlets located in cabinets throughout both trainways. The standpipe system is supplied with a permanent water supply pressurized from the local utility. Per Seattle Fire Code, system demand is 1000gpm at 130psi residual pressure. Utility pressure is generally not sufficient to provide required residual pressure at hose valve connections. As such, fire department connections (FDC's) are provided at both portals allowing the system to be manually pressurized by the responding fire department. In accordance with NFPA 14 and the Sound Transit Design Criteria Manual,

trainway hose valves are provided at maximum 200-foot intervals. Trainway hose valves are in recessed niches along the center wall of each trainway

8. Geotechnical Design

8.1. Available Data

Available geotechnical data along the proposed tunnel alignment was obtained from the Washington Department of Natural Resources web application, from recent geotechnical reports prepared as part of the Sound Transit alternatives study at the site, from historical geotechnical reports prepared for the WSHB, and from environmental studies for both the WSHB and adjacent infrastructure. A detailed review of laboratory test data and development of static and dynamic soil properties was beyond the scope of this study.

8.2. Geologic Profile

Based upon the available geotechnical data referred to in Section 8.1, from near the west bank of the West Waterway and to the east the proposed tunnel alignment will primarily be founded in alluvial and fluvial-marine deposits, as described in Section 3.3.2. To the west, the tunnel will transition to being founded in recessional lacustrine deposits and glacially deposited or overridden deposits, as described in Sections 3.3.3 and 3.3.4. Excavations along the tunnel corridor are expected to encounter fill materials at the surface, as described in Section 3.3.1.

8.3. Geotechnical Cross Section

The proper placement of foundation, lateral support and vertical protection and restraint materials is integral to the performance of the immersed tunnel. The FHWA Tunnel Design Manual ("FHWA TDM", 2009) provides general recommendations for material types, gradations and placement procedures, which are discussed in their application to this project below, and are illustrated on Figures A-2 and A-3 of Appendix A.

8.3.1. Leveling Materials

Leveling materials will be placed and screeded prior to setting immersed tunnel segments. The FHWA TDM recommends that this material be between about 1.5 feet and 4.5 feet in thickness and "consist of clean, sound, hard durable material with a grading compatible with the job conditions." This preliminary study has assumed that the leveling materials will consist of a uniformly graded gravel placed with a typical thickness of about 3 feet.

8.3.2. Keying / Locking Materials

Keying or locking materials should be placed after the immersed tunnel segments have been installed and connected. Similar to the leveling materials, the FHWA TDM recommends that this material consist of "granular, clean, sound, hard, durable material that will compact naturally and that will remain stable under both non-seismic and seismic conditions." Placement of keying/locking materials should proceed in a balanced fashion, with the level of material on one side of the tunnel segment roughly matching the other, to minimize the risk of lateral displacement of the tunnel segment. This preliminary study has assumed that the keying/locking materials will extend up to 20 feet above the base of the tunnel segment, consistent with the

FHWA TDM recommendation that these materials extend up to at least half the height of the tunnel segment.

8.3.3. Protection Materials

Above the keying/locking materials, a blanket of well graded stones (<12 inches in diameter) is typically placed to provide protection to the tunnel as well as, if needed, act as a barrier layer between riprap armoring and keying/locking materials. When riprap armoring is used it can then be placed above the stone blanket the provide further protection to the tunnel as well as further lock the immersed tunnel segments in place. The sizes and gradations of the stone blanket and the riprap armoring are dependent on potential external impact loads and channel currents/flows. External impact loads and channel flow velocities were not evaluated as part of this preliminary study has assumed 5-foot layer thicknesses for both the stone blanket and riprap armoring.

8.4. Soil Parameters

Soil strength parameters for detailed design were not developed as part of this preliminary study. The development of a detailed design at this site should reference total and effective stress parameters, and particular care should be used in the development of post-seismic strength parameters if the materials at the site are concluded to be susceptible to liquefaction and/or cyclic strength loss.

8.5. Ground Improvement

While immersed tunnels can be designed to accommodate significant ground movements from the loading of soft soils and from seismic events, targeted ground improvements may be necessary should future, more detailed studies conclude that soils at the site are susceptible to liquefaction and/or cyclic strength loss. Other immersed tunnel projects incorporating ground improvement programs to mitigate against the effects of liquefaction and/or cyclic strength loss include compaction grouting beneath the Marmaray Tunnel in Turkey, stone columns for the proposed new George Massey Tunnel in British Columbia, and a combination of preloading and removal-and-replacement of soft soils for the Midtown tunnel in Virginia. Due to the breadth and depth of the alluvial and fluvial-marine deposits beneath Harbor Island (viz. 200+ feet), future studies should evaluate whether or not these materials are susceptible to liquefaction and/or cyclic strength loss, and if so, establish the type(s), depth(s) and lateral extent(s) of ground improvement feasible at this site.

Additionally, ground improvement to reduce seepage, increase the lateral capacity of excavation support and/or decrease the risk of ground displacements toward the excavation may be included for cut-and-cover and open cut segments depending on the excavation support system used.

9. Excavations

9.1. Channel Dredging

9.1.1. Dredging Operations

Dredged materials are anticipated to primarily consist of loose/soft alluvial and fluvial-marine deposits that can be excavated using conventional methods. The lowest dredged elevation is expected to be about -80 to -90 feet (mean-lower low water; MLLW). Dredge operations will abide by jurisdictional permitting requirements, which may include the use of silt curtains and seasonal work-window limitations.

9.1.2. Dredge Prism

For the purposes of this preliminary study a continuous, 3H:1V slope was assumed for the cut slopes of the dredged prism. It is anticipated that this configuration would be refined as geotechnical design parameters are developed based on a thorough, site-specific geotechnical investigation program.

The dredge prism would not span the entire length of the immersed segments; rather, the prism would continue to the end of excavation support near the edges of the West Waterway. Once reaching the excavation support, the excavation configuration would transition to rectangular shape (refer to Section 9.2.1). It is anticipated that the dredged prism may be approximately 750 feet in length.

9.2. Supported Excavations

9.2.1. Immersed Tunnel Segments

Excavation support will be required to facilitate excavations for immersed tunnel segments outside of the main channel as well as cut-and-cover and open cut segments. In general, the length of the immersed tunnel segments should generally be maximized to reduce both the amount of dewatering and the extents of excavations with unbalanced hydrostatic loading. Excavation support for immersed tunnel excavations outside of the main channel is anticipated to consist of a sheet pile wall shoring system that is tied-back. Potential conflicts between tiebacks and adjacent subsurface infrastructure were not evaluated as part of this study; however, given the proximity of bridge foundations and other structures, conflicts are anticipated. It is recommended that any design advancement include a detailed review of existing infrastructure and evaluate the impacts of any excavation support system, particularly one requiring tiebacks or anchors.

9.2.2. Cut-and-Cover / Open Cut Segments

Excavation support for cut-and-cover and open cut segments is required to be essentially watertight to facilitate construction activities in the dry. Excavation support may differ between the west and east ends of the of the south alignment given the large cut anticipated at Pigeon Point, which may require large-diameter secant pile or slurry walls with systematic use of permanent anchors to provide temporary excavation and permanent slope support. The east cut-and-cover and open cut excavations of both alignments, and the west cut-and-cover and

open cut excavations of the north alignment, will likely use a sheet pile wall shoring system that is tied-back, similar to the immersed segment excavations. Depending on the results of future, more detailed geo-structural and seepage analysis, the sheet pile wall system may include a ground improvement program to provide additional wall support and reduce seepage potential, or these cut-and-cover and open cut excavations may also employ watertight secant pile or slurry walls.

9.3. Material Reuse / Disposal

It is anticipated that roughly 2 million cubic yards of materials will be excavated during construction of the project, not including any materials excavated for the development of a new casting facility. The cut volume was estimated in simplified form using 2D profiles and typical excavation widths; however, future, more detailed studies should estimate anticipated cut volumes using 3D models. A rough distribution of the estimated excavation volumes is presented for the south alignment in Table 4 and for the north alignment in Table 5.

Table 4 – *Distribution of Estimated Tunnel Excavation Volumes (South Alignment)*

	West Open Cut / Cut-and-Cover	**Immersed Segments**	**East Open Cut / Cut-and-Cover**	**TOTAL**
Volume (cubic yards)	0.8 million (±)	1.2 million (±)	0.2 million (±)	**2 million (±)**

Table 5 – *Distribution of Estimated Tunnel Excavation Volumes (North Alignment)*

	West Open Cut / Cut-and-Cover	**Immersed Segments**	**East Open Cut / Cut-and-Cover**	**TOTAL**
Volume (cubic yards)	0.5 million (±)	1.1 million (±)	0.3 million (±)	**2 million (±)**

10. Structural Design

10.1. Summary

At this preliminary stage, the immersed tunnels have been designed for strength and serviceability considerations, while confirming the elements can be buoyant for transport and able to be immersed with a certain height of ballast while maintaining the space-proofing requirements indicated in the previous report sections. The typical cross section with the structural dimensions noted is presented in Figure 7.

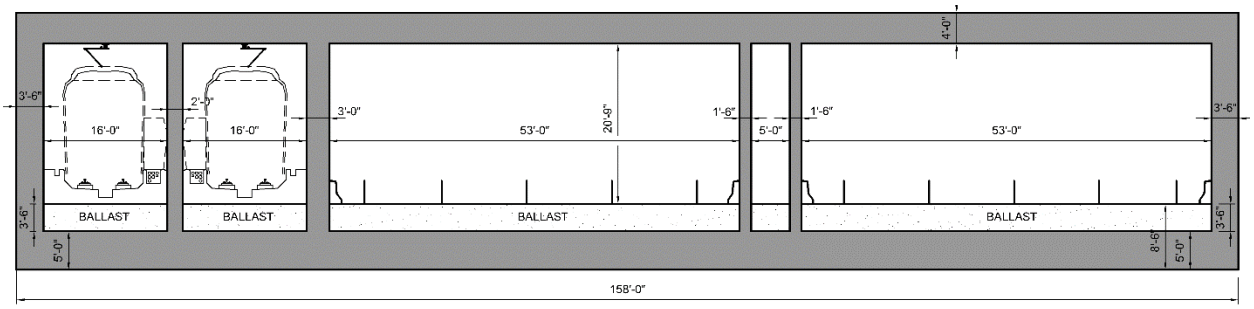

Figure 7 – Typical Cross Section

This cross section allows for a buoyant condition during transport and can achieve a factor of safety against buoyancy of 1.02 during immersion.

Based on the unit weights of water, at the casting site and at the immersion site, and concrete, the approximate freeboard and draft are as follows[2]:

Freeboard (distance between water surface and top of box structure) = 5 to 7 feet
Draft (distance between water surface and bottom of box structure) = 25 to 30 feet

10.2. Codes and Standards

- AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, 2017, American Association of State Highway and Transportation Officials
- AASHTO LRFD Bridge Design Specifications, 2020, American Association of State Highway and Transportation Officials
- Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-18), American Concrete Institute

10.3. Methodology

Based on our experience, the required structural dimensions are most often controlled by the transverse cross-sectional analysis. Because of this, a longitudinal analysis was not performed at this preliminary stage.

To perform the transverse analysis, a 2-dimensional finite element frame model was developed by using a commercial structural analysis program LARSA. The depth of the model was taken as one foot, which corresponds to a single unit of length in the longitudinal direction of the tunnel. The following assumptions were made to develop the preliminary transverse model:

[2] It is noted that many other factors can affect buoyancy, but those explained above are the primary drivers and are considered satisfactory for this study.

- The roof, invert, exterior and interior walls were modeled as 2-node beam elements. Nodes of the beam elements were developed using the centroidal axis of the structural elements.
- Soil-structural interaction was incorporated using simplified, non-linear springs under the invert in the vertical direction and adjacent to the exterior walls in the horizontal direction.
- Stiffness of the soil springs was computed by multiplying the relevant modulus of subgrade reaction and the tributary area at the corresponding node.
- Soil springs possess no tensile capacity.

The material properties assumed for the analysis are as follows:

- Unit weight of concrete = 150 pcf
- Unit weight of water = 63 pcf
- Unit weight of soil = 115 pcf
- Unit weight of rip-rap = 140 pcf
- Unit weight of ballast = 150 pcf
- Compressive strength of concrete f'_c = 6,000 psi
- Yield strength of steel reinforcement f_y = 60,000 psi
- Coefficient of thermal expansion of concrete $\alpha_c = 6 \times 10^{-6}\ °F^{-1}$
- Modulus of subgrade reaction (vertical direction) of soil = 15 kcf
- Coefficient of at-rest earth pressure = 0.5

For strength design, moment, axial force and shear force demands were checked against the ultimate capacities of the sections.

For serviceability design, crack width was computed based upon moment and axial force demands, and checked against the maximum flexural crack width limit for reinforced concrete, which was assumed to be 0.008" – a value commonly used for water-resistant reinforced concrete structures.

10.4. Loads (Normal Loads) Considered for Preliminary Concept

The assumptions and geometries indicated above have yielded the following preliminary loading conditions:

10.4.1. Dead Load (DC)

Self-Weight of Box Structure

- Based upon unit weight of concrete (150 pcf)

Ballast

- Based upon unit weight of concrete (150 pcf)
- Thickness of Ballast = 3'-6"

Non-Structural Component on Roof

- Uniform load intensity = 50 psf

Concrete Protective Cover

- Based upon submerged weight of concrete
- Based upon unit weight of concrete (150 pcf) and water (63 pcf)
- Thickness of concrete protective cover = 3”

10.4.2. Vertical Soil and Rip-Rap Loads on Roof (EV)

- Based upon submerged weight of rip-rap, soil and water
- Based upon unit weight of rip-rap (140 pcf), soil (115 pcf) and water (63 pcf)
- Thickness of rip-rap above the top of box structure = 3’-0”
- Thickness of soil above the top of box structure = 2’-0”

10.4.3. Horizontal Soil Load on Exterior Walls (EH)

- Based upon submerged weight of rip-rap, soil and water
- Based upon unit weight of rip-rap (140 pcf), soil (115 pcf) and water (63 pcf)
- Based upon coefficient of earth pressure at rest at K_0 = 0.5
- Thickness of rip-rap above the top of box structure = 3’-0”
- Thickness of soil above the top of box structure = 2’-0”

10.4.4. Water Load (WAp)

- Based upon unit weight of water (63 pcf)
- Distance between water surface and the top of box structure = 45’-0”

10.4.5. Temperature Load – Hot Temperature (T(hot))

- Based upon thermal expansion of concrete $\alpha_c = 6 \times 10^{-6}\ °F^{-1}$
- For roof, invert and exterior walls:
  Inside face - Increase in temperature = + 18 °F
  Outside face - increase in temperature = 0 °F
- For interior walls:
  Uniform increase in temperature = +18 °F

10.4.6. Temperature Load – Cold Temperature (T(cold))

- Based upon thermal expansion of concrete $\alpha_c = 6 \times 10^{-6}\ °F^{-1}$
- For roof, invert and exterior walls:
  Inside face - decrease in temperature = - 18 °F
  Outside face - decrease in temperature = 0 °F
- For interior walls:
  Uniform decrease in temperature = -18 °F

10.5. Load Combinations and Strength Reduction Factors

10.5.1. Strength Design

The following load combinations and strength reduction factors were considered for strength design:

- LC1 = 1.25 DC + 1.35 EV + 1.35 EH + 1.0 WA_p
- LC2 = 1.25 DC + 1.35 EV + 1.35 EH + 1.0 WA_p + 1.0 $T_{(hot)}$
- LC3 = 1.25 DC + 1.35 EV + 1.35 EH + 1.0 WA_p + 1.0 $T_{(cold)}$
- LC4 = 0.90 DC + 0.75 EV + 0.75 EH + 1.0 WA_p
- LC5 = 0.90 DC + 0.75 EV + 0.75 EH + 1.0 WA_p + 1.0 $T_{(hot)}$
- LC6 = 0.90 DC + 0.75 EV + 0.75 EH + 1.0 WA_p + 1.0 $T_{(cold)}$

Strength Reduction Factors

- For tension-controlled failure, ϕ_b = 0.9
- For compression-controlled failure, ϕ_c = 0.75
- For pure axial compression, ϕ_a = 0.8
- For shear, ϕ_v = 0.85

10.5.2. Serviceability Design

The following load combinations were used to check against the maximum flexural crack width limit for reinforced concrete (0.2 mm = 0.008”):

- LC7 = 1.0 DC + 1.0 EV + 1.0 EH + 1.0 WA_p
- LC8 = 1.0 DC + 1.0 EV + 1.0 EH + 1.0 WA_p + 1.0 $T_{(hot)}$
- LC9 = 1.0 DC + 1.0 EV + 1.0 EH + 1.0 WA_p + 1.0 $T_{(cold)}$

10.6. Loads (Extreme Loads) and Design Issues Not Considered for Preliminary Concept

Immersed tunnels can be subjected to a number of extreme load cases due to their unique nature. These extreme load cases are typically checked during later stages of design, although they were not checked as part of this study, these extreme load cases are identified and explained in the following sections.

10.6.1. Ship Anchors

Immersed tunnel structures are designed to account for the effect of an anchor directly impacting or being dragged across the tunnel structure. Depending on the project requirements, either the tunnel structure itself is designed to resist the full loading imposed by the design anchor system, or the protection materials over the tunnel (i.e. riprap) are designed to attenuate the loading, in which case the tunnel structure is designed for a reduced load. The penetration depth of a falling anchor through protection materials is typically limited to a maximum of 90% of the total thickness of the protection materials. Development of the design load depends on the mass of the anchor, the impact velocity of anchor, the cross-sectional area and equivalent diameter of the striking area of anchor, and the modulus of elasticity in the

longitudinal direction of the protection materials. A static equivalent load is typically used, which is related to the dynamic impact load by way of a dynamic load factor (DLF), which is simply the ratio of the static equivalent load to the dynamic impact load.

10.6.2. Ship Sinking and Grounding

Immersed tunnels are also designed for sunken or grounding ship consistent with the ship type and size expected in the waterway. The imposed loading of a ship on the tunnel is typically taken as an equivalent uniform loading over an area no more than the full width of the tunnel multiplied by a length of 100 feet measured on the longitudinal axis of the tunnel. The intensity of the uniformly distributed loading can be determined by methods such as those outlined in Chapter 6 of the State-of-the-Art Report, 2nd Edition, ITA Immersed and Floating Tunnels Working Group, Pergamon Press, 1997. Collison impact loading shall not be considered in this case.

A second sunken ship design case is sometimes considered and assumed to consist of a smaller vessel, such as a ferry or barge, sinking and impacting the tunnel structure. This load case consists of a static equivalent concentrated load of 225 kips acting on an area of approximately 3 feet x 6 feet on the tunnel.

10.6.3. Support Loss

Immersed tunnels are designed to accommodate a loss of support over the full width of the tunnel up to a distance of 30 feet in the longitudinal direction at any point along the length of the element. Uneven support is considered as follows: zero on one side of the element transitioning to full support on the opposite side of the element for a length of 30 feet along the longitudinal axis of the tunnel.

10.6.4. Seismic Load

Performance during seismic shaking is considered with regards to the three primary modes of deformation during seismic shaking: racking, axial and curvature deformations.

Racking deformation is the distortion of the cross section due to horizontal forces and is the differential sideways movement between the top and bottom elevations of rectangular tunnel structures. It is caused primarily by seismic waves propagating perpendicular to the longitudinal axis of the tunnel, causing deformations in the plane of cross section. Vertically propagating shear waves are generally considered the most critical type of waves for this mode of deformation. The resulting structural internal forces (axial force, moment and shear) in the tunnel structure associated with the seismic racking deformation may be derived by imposing the differential deformation on a 2-D finite element frame model.

The tunnel structure is checked for its ability to accommodate seismic strains caused by axial and curvature deformations of the ground. They are dependent upon the design peak ground velocity, design peak ground acceleration, horizontally travelling shear wave velocity, geometry of the cross section, and the angle of wave propagation with respect to the tunnel axis.

10.6.5. Fire Load

In addition to the fire and life safety precautions discussed in Section 7, the impact of a design fire within the tunnel is considered with respect to the tunnel structure. Due to the degradation in material properties associated with exposure to high temperatures considered for the design fire, the integrity of the tunnel structure, following exposure, is evaluated against collapse. Explosive spalling of concrete is not permitted.

10.6.6. Accidental Explosion

It is expected that potentially explosive material transport will not be allowed in the tunnel, thereby reducing both the likelihood and intensity of an accidental explosion. In any case, an appropriate design loading for a lesser intensity, accidental explosion would still need to be determined.

10.6.7. Flooded Tunnel

Although any design advancement should mitigate the risk of any inundation hazards, it is typical for immersed tunnels to be designed to withstand the effects of a flooded tunnel. Loads associated with flooding of the tunnel are not be combined with other extreme loads.

10.6.8. Settlement and Heave

Immersed tunnels are typically designed to accommodate anticipated settlements, or heave, (total and differential) for each stage of construction, taking the expected duration of time related effects into account, as well as for long-term consolidation settlements and/or post-seismic settlements.

10.7. Loads (Construction Loads) Not Considered for Preliminary Concept

Construction loads include loads imposed when the tunnel section is launched, transported and outfitted/placed. Loads in transport include those imposed when towing the sections, wind and wave action on the floating section, and current loads. Outfitting and placement loads include the loads imposed when the section is floating and concrete is placed in or on the section.

10.8. Joints

As the tunnel is made up of individual elements, joining the elements is critical to ensure a watertight seal. The initial seal between the elements can be provided by the compression of rubber gasket (Gina gasket) attached to the face of one tunnel element bearing onto a smooth surface on the adjoining element. Hydraulic coupling jacks extending from one of the elements are attached to the element to be positioned. The jacks pull the elements together and compress the gaskets. This initial compression of the gasket provides a good enough seal for the joint area to be drained from the inside. This brings the full hydrostatic pressure onto the far end of the element and further compresses the gasket. After dewatering of the tunnel element, the joint area can be entered via doors in the bulkhead and a permanent connection can be made by a gasket (Omega seal) mounted inside.

Some design considerations are as follows:

- Joints should be watertight throughout the design life.
- Joints should be capable of accommodating movements caused by temperature fluctuations, creep, settlement, earthquake motions and method of construction.
- The influence of normal forces and bending moments on the shear capacity of the section should be considered when determining shear capacity of the joint.
- The axial compression of tunnel elements and bulkheads due to depth of immersion and temperature effects should be considered in determining joint dimensions at installation.
- The design of primary flexible seals should take into account the maximum deviations of the supporting frames relative to their theoretical location, the maximum deviation of the planes of the frames, and any relaxation of the seal.
- The secondary seal (Omega seal) should be capable of absorbing the long-term movements of the joint.

11. Casting Facilities

11.1. General Overview

Immersed tunnel segments are typically constructed in dry docks or casting basins that are adjacent to a navigable waterway. Many immersed tunnel projects have required the development of dedicated casting facilities that are designed specifically for the project, such as the Fehmarnbelt Tunnel project, though some have been able to utilize existing dry dock facilities, such as the Elizabeth River Tunnels project. The cost to develop a project-specific casting facility is typically quite high; however, project constraints can often make this the only practical solution, whether this is due to lack of capable, existing facilities or due to navigation challenges or conflicts.

Recent WSDOT experience with the SR-520 Pontoon Construction Project ("SR-520 PCP") included a combination of both approaches, with lesser portion of the pontoons constructed at the Concrete Technology Corporation casting basin ("CTC Facility") in Tacoma and the majority of the pontoons constructed at a newly developed Aberdeen casting basin ("Aberdeen Facility") in Grays Harbor. According to the project management plan prepared for the SR-520 PCP, the anticipated cost for design, development, construction and delivery of the pontoons was estimated to be $591 million, with "*the approximate allocation of funds…as follows:*

- *Preliminary Engineering - $43 million*
- *Right of Way - $7 million*
- *Construction - $541 million*"[3]

SR-520 Pontoon Construction Project

[3] WSDOT, 2009. Pontoon Construction Project. Project Management Plan, SR 520 Bridge Replacement and HOV Program. Submitted to Julie Meredith, SR 520 Program Director. July 2009.

As part of the concept development, three alternatives for casting facilities have been considered: the CTC Facility, the Aberdeen Facility and the development of a new, project-specific facility. Requirements and considerations for the selection of one or more facilities are described in the following subsections.

11.2. Existing Facility Review

The dimensional requirements of the casting basin are, intuitively, governed by the size of the planned immersed tunnel segments. The preliminary, immersed tunnel segment width and height dimensions are based on required clearances for vehicular lanes, transit allowances and tunnel spaceproofing. The maximum immersed tunnel segment length is limited by navigation channel clearances and the vertical curve along the alignment, as discussed in Section 4.2. Based on these assumptions and limitations, the maximum immersed tunnel segment dimensions were assumed to be: 300 feet (length) x 159 feet (width) x 30 feet (±, draft). A comparison the preliminary design dimensions to the dimensions of the CTC and Aberdeen Facilities is provided in Table 6.

Table 6 – *Available Casting Facility Dimensional Comparison*

Design / Facility	Length		Width		Draft	
Design	*300 feet*		*159 feet*		*30 feet (±)*	
CTC Facility*	475 feet	☑	150 feet	☒	20 feet	☒
Aberdeen Facility	930 feet	☑	185 feet (110 feet at gate)	☒	20 feet (at high tide)	☒

* *CTC Facility dimensions are approximate and derived from Google Earth and scales shown in site photos*

As indicated in Table 6, the CTC and Aberdeen Facilities do not provide adequate widths or heights to construct and convey the immersed tunnel segments out of the respective casting basins in their current configurations. In order accommodate the preliminary, maximum immersed tunnel segment dimensions, the CTC and Aberdeen Facilities would require major modifications to their widths and base depth. To conform to the width limitations of the existing facilities, it is possible that the vehicular tunnel could be separated from the transit tunnel, resulting in two roughly parallel tunnels rather than the combined, single tunnel reviewed as part of this study; however, vertical modifications to the facilities would still be required to accommodate the necessary draft.

11.3. New Facility Considerations

Potential sites for a new casting facility were not identified as part of this preliminary study, but it is reasonable to assume that a site selection, procurement, development and construction program similar in scope to the program for the SR-520 PCP (though with a much smaller number of units to be fabricated) would be required. It is recommended that any new casting facility developed for this project be sized to accommodate the simultaneous fabrication of multiple immersed tunnel segments, not just one at a time, as temporary float storage of immersed segments and/or significant down time between placement of immersed segments can increase project costs and risks.

12. Risk

12.1. Engineering

12.1.1. Civil Tie-Ins

As noted in Section 1.3, the design of civil tie-ins with existing facilities was not completed as part of this study. The proposed tunnel's vertical and horizontal alignments have been developed to maintain general compatibility with existing infrastructure. Details of roadway connections were not developed, and although potential, alternative configurations for Sound Transit's light rail expansion are being studied by others at this time, the preferred alternative has not yet been identified.

12.1.2. Geologic Hazards

As noted in Section 3.4, a detailed geologic hazard review was not completed as part of this study. Detailed design advancement will need to account for seismic effects at the site, including differential site response along the tunnel alignment; ovaling and racking deformations during ground shaking; post-seismic settlements and material strength loss; and post-seismic slope deformations. In addition, the site is located in a mapped tsunami inundation zone (Walsh et al., 2003), and detailed design advancement will need to account for potential inundation heights at all portal locations.

In addition to the seismic hazards at the site, the south alignment intersects Pigeon Point at a mapped landslide location. Any detailed design advancement of the south alignment will need to study this feature, and the potential for nearby, undocumented features, to assess the feasibility of different excavation and slope support types and configurations, as well as the risk for damage to upslope properties should any slides reinitiate.

12.1.3. Ground Improvement

As noted in Section 8.5, ground improvement to mitigate against seismic hazards was not considered as part of this study. Future, more detailed studies should characterize the seismic hazards and ground response at the site to evaluate the need for a ground improvement program. A number of other immersed tunnel projects have included ground improvement programs as part of their design and construction, and it is anticipated that ground improvement may be required for this project.

12.2. Environmental

12.2.1. Dredging Constraints

Required permits and site-specific requirements for dredging operations were not reviewed as part of this study, but it is anticipated that best management practices (BMPs) and performance constraints may include: silt curtains, turbidity monitoring, biological monitoring, seasonal work limitations, and navigation channel use provisions (regular and/or emergency).

12.2.2. Contaminated Soil

An environmental review to assess the likelihood of encountering contaminants was not conducted as part of this preliminary study, and for the purposes of cost estimating it has been assumed that materials at the site are clean. If further environmental review and/or investigation reveals that contaminants may be expected at the site, then the cost associated with transport and disposal can be assumed to increase substantially, and material import requirements can be assumed to increase as well (any contaminated materials that are dredged or excavated will not be used for backfilling).

12.3. Land Use / Acquisition

12.3.1. Navigation Channel Impacts

The West Waterway is currently maintained with a required channel bottom elevation of -30 feet mean lower-low water ("MLLW"; -32.34 feet NAVD88), though the existing dredge depths are as low as -42 feet MLLW (-44.34 feet NAVD88) in the vicinity of the tunnel alignments. The top of the tunnel structure has been set with a maximum elevation of -45 feet MLLW (-47.34 feet NAVD88) to allow for protection materials, to sit deeper than the existing dredged depth and allow for the potential that dredge requirements may change in the future. In light of this, permanent facilities developed as part of the project are not anticipated to have long-term impacts on use of the West Waterway unless required dredging depths are appreciably lowered.

Temporary impacts to navigation in the West Waterway are expected during construction. Impacts to navigation can be expected during dredging excavation for immersed segments, floating and placing immersed segments, and backfilling around and above immersed segments. Costs and coordination requirements associated with these impacts have not been evaluated as part of this study, but it is possible that delays and/or construction inefficiencies could be realized due to restrictions related to temporary closures of the navigation channel.

12.3.2. Right-of-Way Impacts

Significant temporary and permanent right-of-way impacts are expected for both concepts considered in this study. Any further development of these concepts should define the range of impacts and necessary coordination requirements, including costs, operational impacts (i.e. Port), and temporary and permanent easement agreements.

12.3.3. Utility Conflicts

Aside from the overhead power lines north of the WSHB, particular utility conflicts were not evaluated as part of this study. Utility impacts associated with the project are not limited to those within the footprint of the tunnel alignment, but also include those utilities that may be within the range of tie-backs and anchors used for temporary excavation and permanent slope support as well as those within close proximity that may be sensitive to ground movements associated with soil relaxation and dewatering, as needed, in the vicinity of temporary excavations.

12.3.4. Casting Facility

As noted in Section 11.2, the CTC and Aberdeen Facilities do meet depth or width requirements for the tunnel segments considered as part of this study. The development of a new facility

appears to be the best solution for the project; however, the process to identify, acquire, design and develop a new facility is expected to be both lengthy and costly.

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Walsh, Timothy, et al. 2003. Tsunami Hazard Map of the Elliott Bay Area, Seattle,Washington: Modeled Tsunami Inundation from a Seattle Fault Earthquake. Prepared for the Washington State Department of Natural Resources and the National Oceanic and Atmospheric Administration. Washington Division of Geology and Earth Resources, Open File Report 2003-14.

ATTACHMENT A

Concept Plans

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-122.3750° 47.7000mE 48 49 50 51 52 53 54 55 56 -122.2500° 47.6

T25N R3E T25N R4E

SEATTLE

Capitol Hill

Lincoln Reservoir

Seattle Central College

Cornish College of the Arts

Antioch University Seattle

Argosy University - Seattle

Seattle School of Theology and Psychology

The Art Institute of Seattle

Seattle University

Madrona

Yesler Terrace

King County Courthouse

Elliott Bay

Lake Washington

Faben Point

Mercer Island

Lacey V Murrow Bridge

SEATTLE - BAINBRIDGE ISLAND FERRY

SEATTLE - BREMERTON FERRY

SEE INSET AT RIGHT

T24N R3E T24N R4E

Harbor Island

NORTH ALIGNMENT

SOUTH ALIGNMENT

Duwamish West Waterway

Duwamish East Waterway

Pigeon Point

Harbor Island Reach

Beacon Hill

Beacon Hill Reservoir

Mount Baker

Mount Baker Beach

Mount Baker Ridge Tunnel

South Seattle

Columbia

Ohlers Island

Rainier Valley

Brighton

Brighton Beach

Holly Park

Andrews Bay

Bailey Peninsula

Delridge

Georgetown

Kellogg Island

Georgetown Reach

First Avenue South Bridge

Forest Lawn Cem

Seattle Community College South Campus

South Seattle College - Georgetown Campus

South Seattle College - Newholly Learning Center

Boeing Field/King County International Airport

First Avenue Eighth Avenue Reach

W SEATTLE F.WY

SW ANDOVER ST

SW GENESEE ST

SW MYRTLE ST

SW OTHELLO ST

S SPOKANE ST

S MICHIGAN ST

S ANDOVER ST

S GENESEE ST

S OREGON ST

S ALASKA ST

S ADAMS ST

S HUDSON ST

S BRANDON ST

S BENNETT ST

S ORCAS ST

S MEAD ST

S JUNEAU ST

S FINDLAY ST

S FERDINAND ST

S ANGELINE ST

S EDMUNDS ST

S SHELTON ST

S DAWSON ST

S LUCILE ST

S FIDALGO ST

S GRAHAM ST

S EDDY ST

S WARSAW ST

S MORGAN ST

S HOLLY ST

S WILLOW ST

S FRONTENAC ST

S MYRTLE ST

S GARDEN ST

S OTHELLO ST

S WEBSTER ST

S STATE ST

S GRAND ST

S PLUM ST

S WALKER ST

S BAYVIEW ST

S MCCLELLAN ST

S HANFORD ST

S HORTON ST

S HINDS ST

S STEVENS ST

S LANDER ST

E MARGINAL WAY S

ALASKAN WAY S

MARTIN LUTHER KING JR WAY S

RAINIER AVE S

LAKE WASHINGTON BLVD

Mount Baker Ridge Tunnel

Lacey V Murrow Bridge

NOT TO SCALE

T24N R3E

T24N R4E

A ST

SW FLORIDA ST

SW ANDOVER ST

SW GENESEE ST

SW MYRTLE ST

SW OTHELLO ST

16TH AVE SW

13TH AVE SW

22ND AVE SW

DELRIDGE WAY SW

HIGHLAND PARK WAY SW

W MARGINAL WAY SW

E MARGINAL WAY S

ALASKAN WAY S

UTAH AVE S

COLORADO AVE S

KLICKITAT AVE BRG

Duwamish West Waterway

Duwamish East Waterway

Harbor Island

Pigeon Point

Harbor Island Reach

NORTH ALIGNMENT

SOUTH ALIGNMENT

NOT TO SCALE

Produced by the United States Geological Survey
North American Datum of 1983 (NAD83)
World Geodetic System of 1984 (WGS84). Projection and
1 000-meter grid:Universal Transverse Mercator, Zone 10T

VAULT	SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE CH'D	REV'D

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Sep-03-20 8:07pm

WSP USA Inc.
1301 Fifth Avenue
Suite 1200
Seattle, WA 98101-2677
TEL: 206-357-5600
FAX: 206-357-5601

PRELIMINARY CONCEPT FIGURE

INITIALS AND DATE	INITIALS AND DATE
DESIGNED	REVIEWED:
CHECKED	DES. CONST.
	SDOT PROJ. MGR.
DRAWN	RECEIVED
CHECKED	REVISED AS BUILT

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Seattle Department of Transportation

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SCALE:

WEST SEATTLE HIGH RISE BRIDGE
COST BENEFIT ANALYSIS

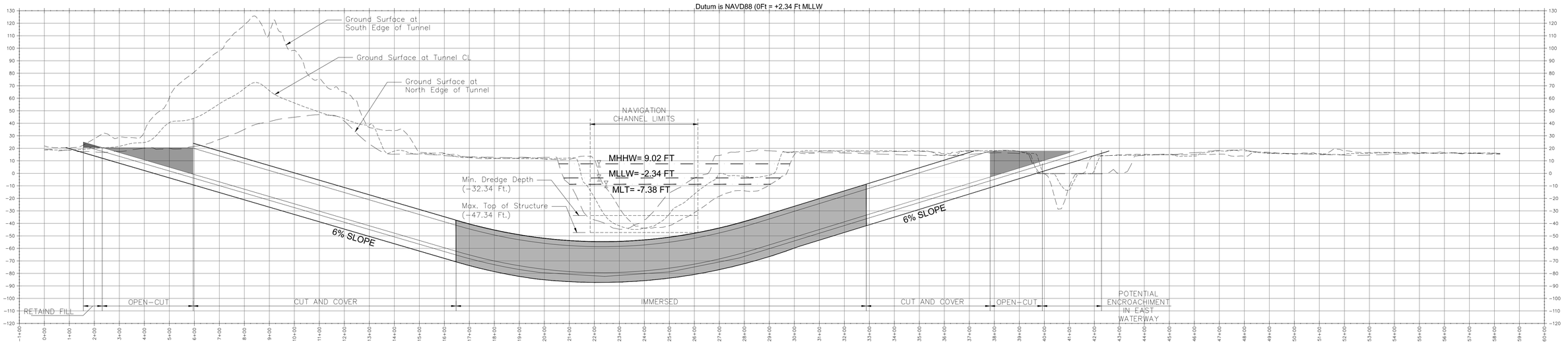
IMMERSED TUNNEL STUDY
SITE VICINITY MAP

JOB PC CO
VPI #

FIGURE A-1

PLAN

SCALE: 1"=200'



NOTES:

1. MHHW = MEAN HIGHER-HIGH WATER
2. MLLW = MEAN LOWER-LOW WATER
3. MLT = MINIMUM OBSERVED LOW TIDE

PROFILE

HORIZ. SCALE: 1"=200'
VERT. SCALE: 1"=40'

TUNNEL PROFILE LINEWORK KEY

SECTION VIEW

PROFILE VIEW

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WEST SEATTLE HIGH RISE BRIDGE
COST BENEFIT ANALYSIS

IMMERSED TUNNEL STUDY
TUNNEL PLAN AND PROFILE
(SOUTH ALIGNMENT)

JOB PC CO
VPI #

FIGURE A-2

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE CH'K'D	REV'D

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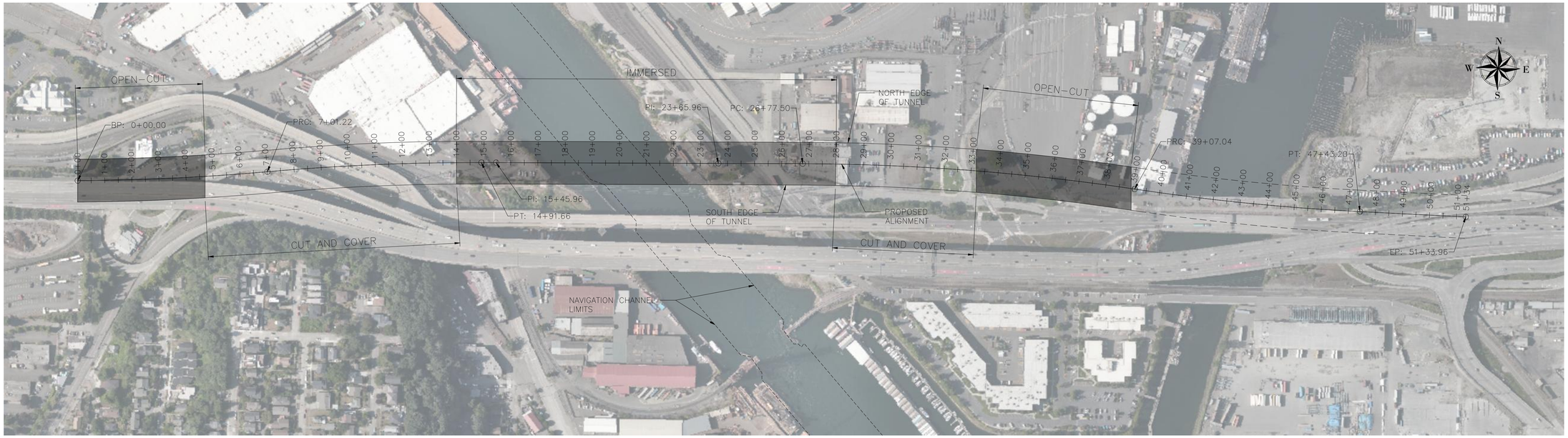
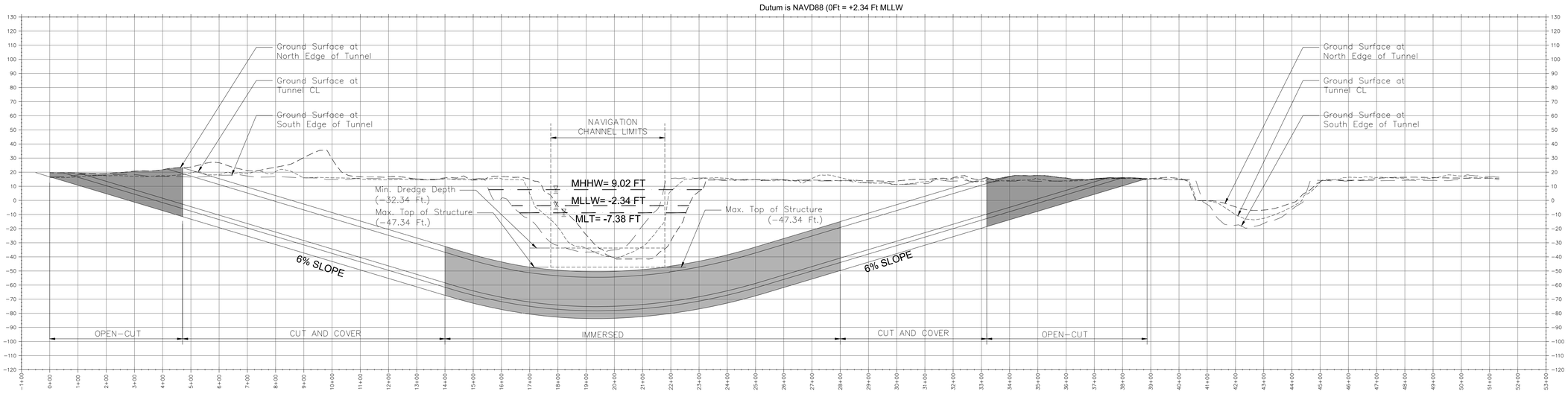


PLAN

SCALE: 1"=200'



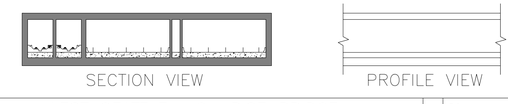
NOTES:

1. MHHW = MEAN HIGHER–HIGH WATER
2. MLLW = MEAN LOWER–LOW WATER
3. MLT = MINIMUM OBSERVED LOW TIDE

PROFILE

HORIZ. SCALE: 1"=200'
VERT. SCALE: 1"=40'

TUNNEL PROFILE LINEWORK KEY



SECTION VIEW PROFILE VIEW

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COST BENEFIT ANALYSIS

IMMERSED TUNNEL STUDY
TUNNEL PLAN AND PROFILE
(NORTH ALIGNMENT)

JOB PC CO
VPI #

FIGURE A–3

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE CHK'D	REV'D

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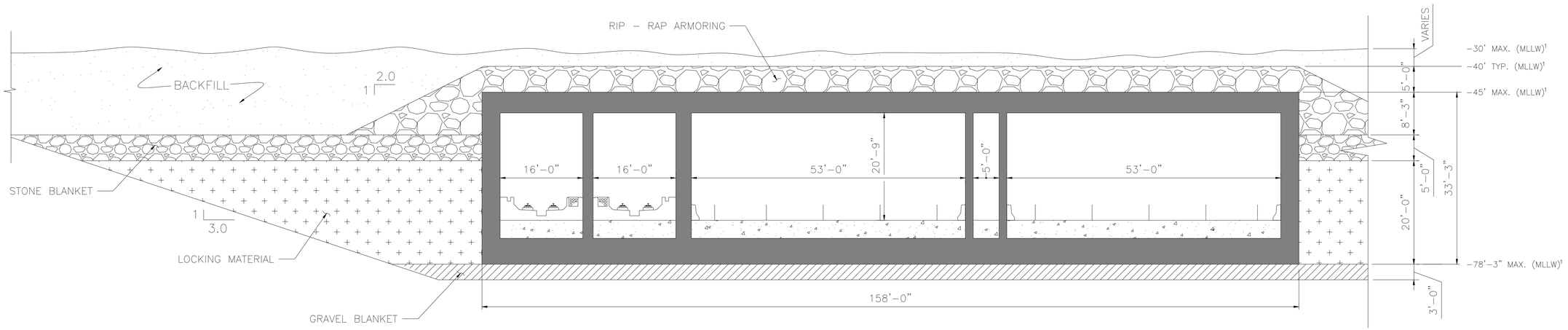
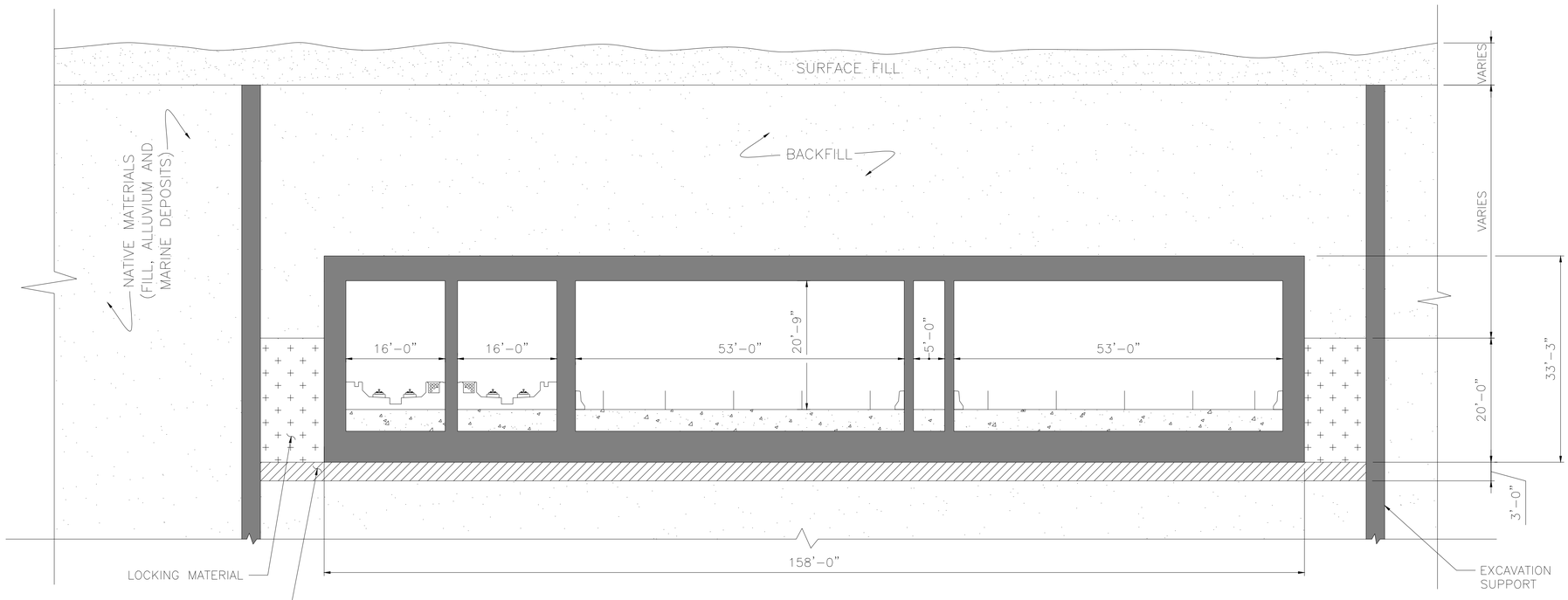


TYPICAL IMMERSED SECTION IN WATERWAY

0 8 16 24 32

SCALE IN FEET



NOTES:

1. ELEVATIONS ARE PROVIDED IN REFERENCE TO MEAN LOWER–LOW WATER. USE THE BELOW FORMULA TO CONVERT TO NAVD88.

   0.0' MLLW=–2.34' NAVD88

TYPICAL IMMERSED SECTION OUT OF WATERWAY

0 8 16 24 32

SCALE IN FEET

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE	CH'D	REV'D

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		SDOT	PROJ. MGR.
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CHECKED		REVISED AS BUILT	

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FUND:

SCALE:

WEST SEATTLE HIGH RISE BRIDGE
COST BENEFIT ANALYSIS

IMMERSED TUNNEL STUDY
TYPICAL GEOTECHNICAL SECTIONS (1 OF 2)

JOB PC CO

VPI #

FIGURE A–4

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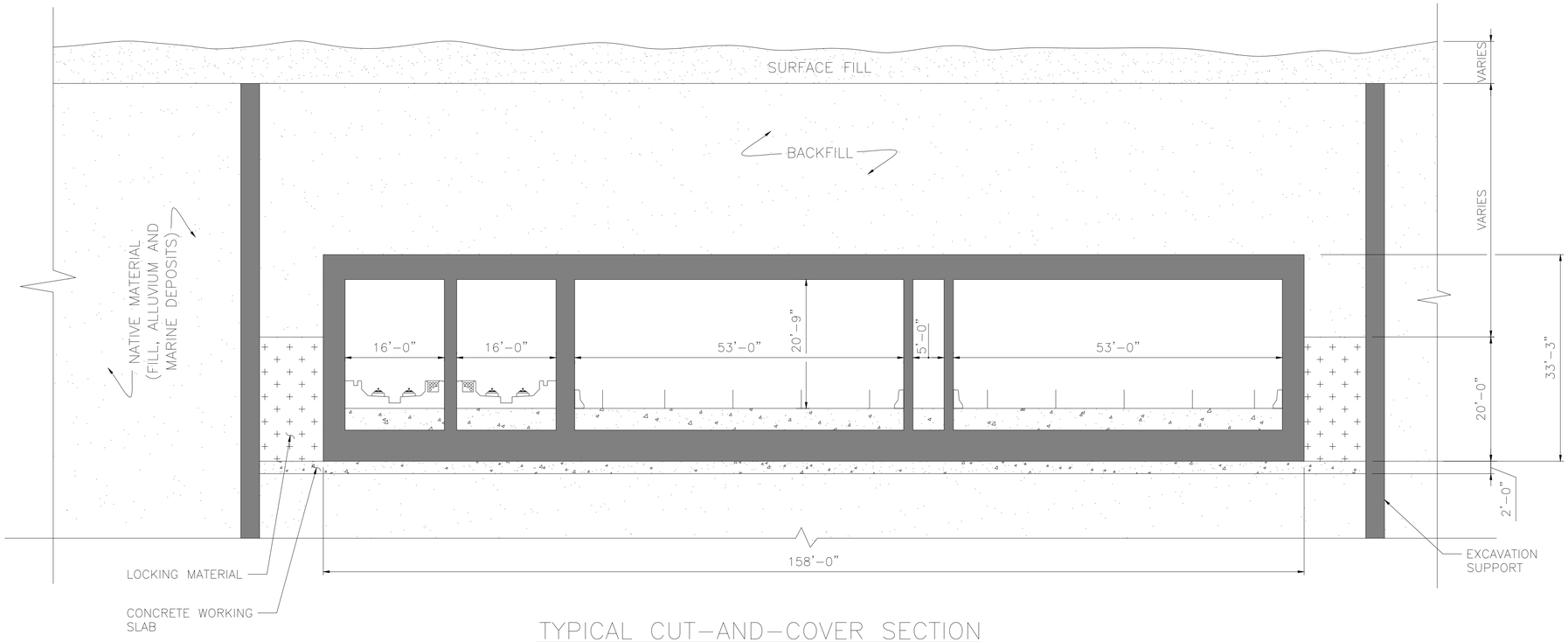

TYPICAL CUT-AND-COVER SECTION

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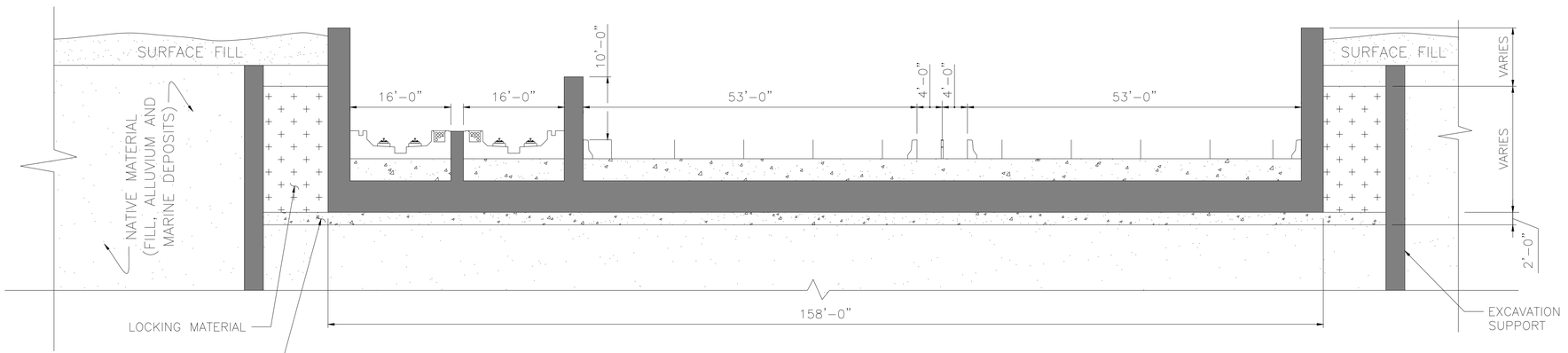
SCALE IN FEET



TYPICAL OPEN CUT SECTION

0 8 16 24 32

SCALE IN FEET

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Seattle Department of Transportation

ORDINANCE NO. PW NO.
FUND:
SCALE:

WEST SEATTLE HIGH RISE BRIDGE
COST BENEFIT ANALYSIS

IMMERSED TUNNEL STUDY
TYPICAL GEOTECHNICAL SECTIONS (2 of 2)

JOB PC CO
VPI #

FIGURE A-5

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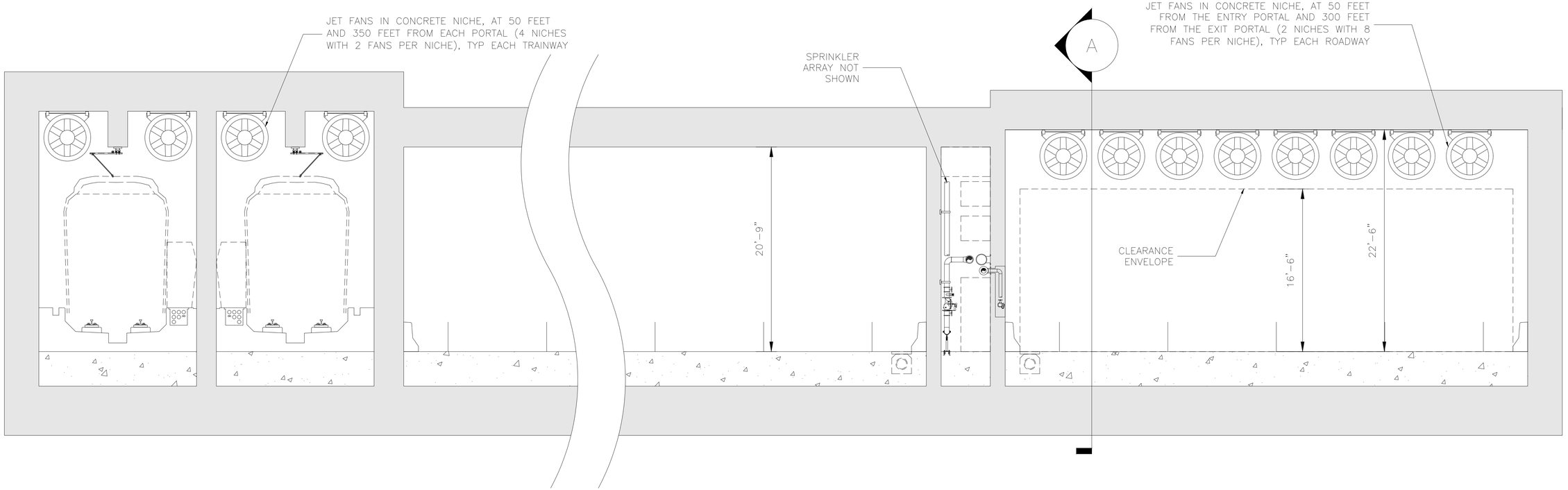
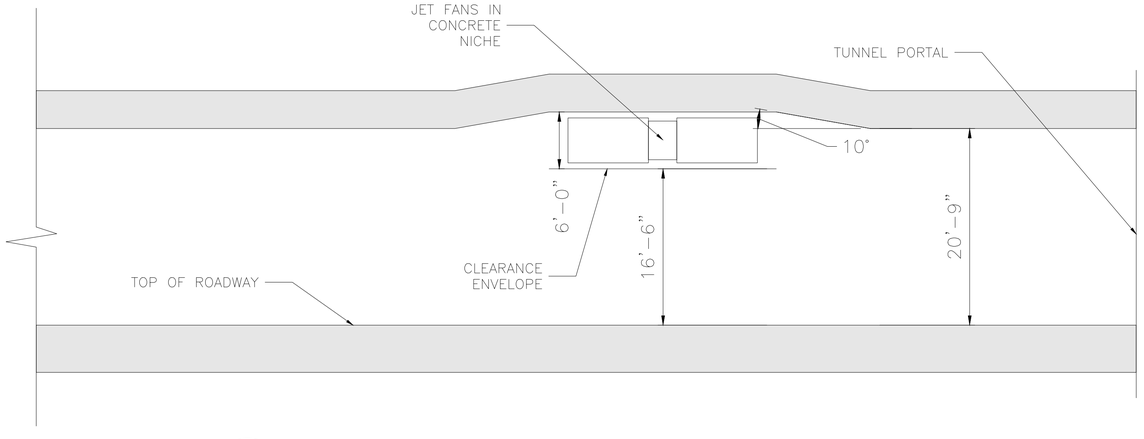


CUT-AND-COVER SECTION AT FAN NICHES



A ROADWAY JET FAN NICHE LONGITUDINAL SECTION (TRAINWAY FAN NICHES SIMILAR)

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE CH'K'D	REV'D

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PRELIMINARY CONCEPT FIGURE

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	SDOT PROJ. MGR.
DRAWN	RECEIVED
CHECKED	REVISED AS BUILT

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FUND:
SCALE:

WEST SEATTLE HIGH RISE BRIDGE
COST BENEFIT ANALYSIS

IMMERSED TUNNEL STUDY
TYPICAL MECHANICAL SECTIONS (1 OF 2)

JOB PC CO
VPI #

FIGURE A-6

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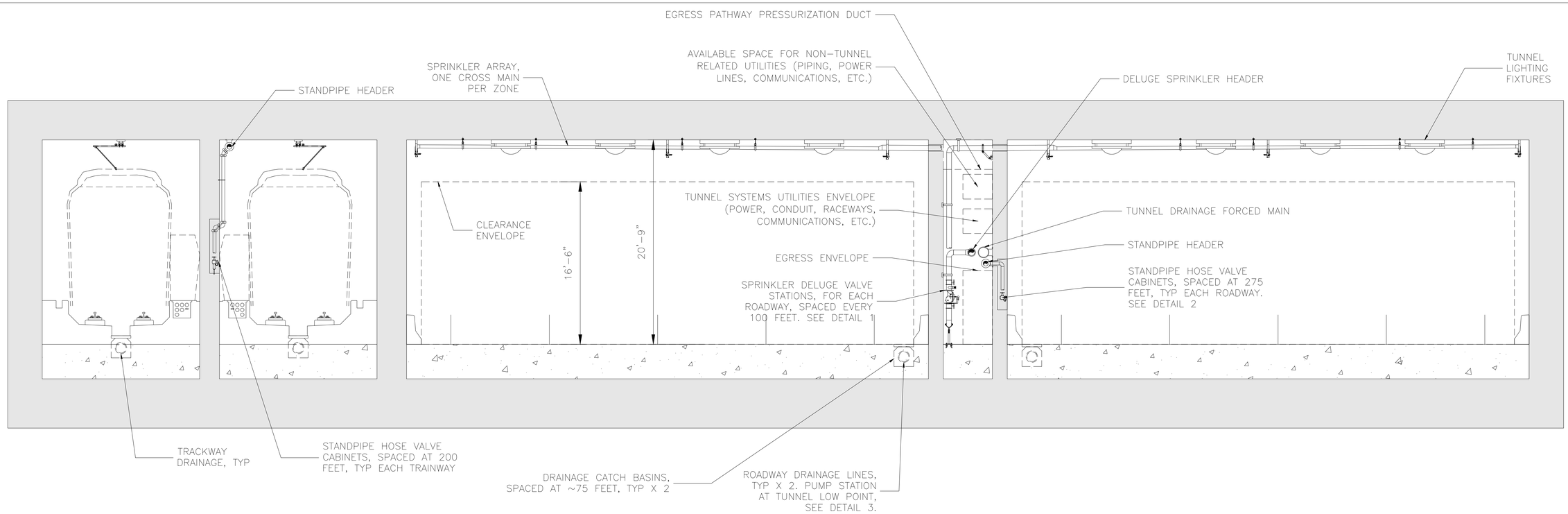
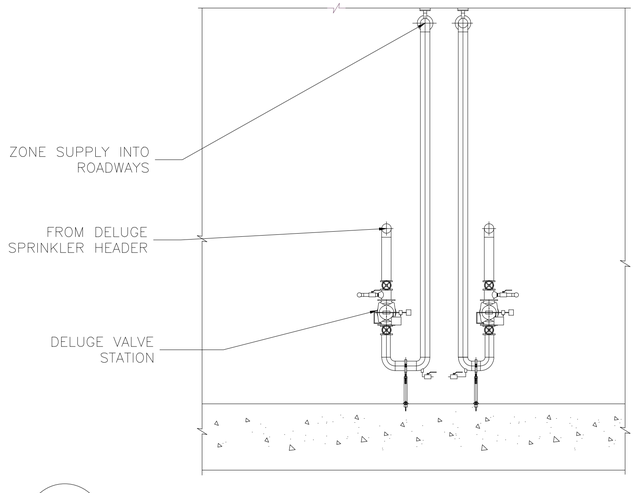
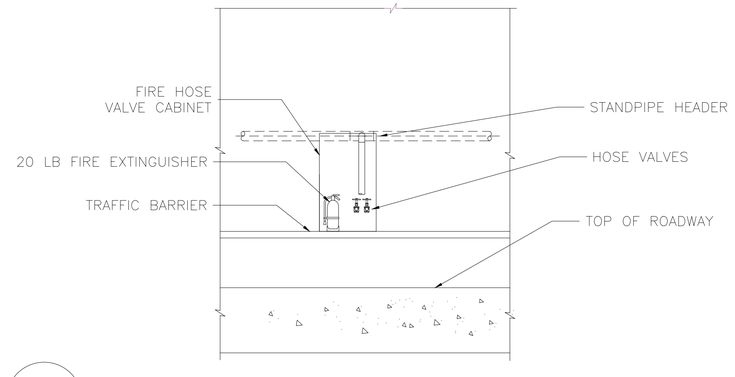
Sep-15-20 10:24am



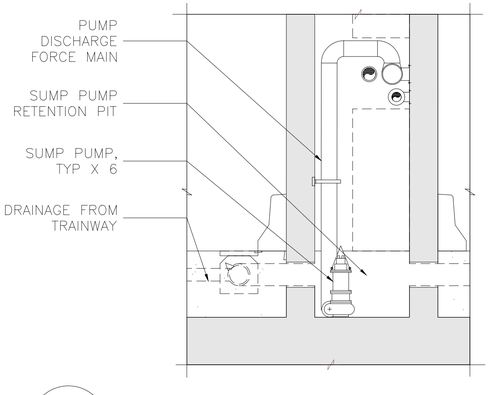
TYPICAL IMMERSED TUNNEL SECTION



1 TYPICAL SPRINKLER DELUGE VALVE STATION



2 TYPICAL ROADWAY HOSE VALVE STATION



3 LOW POINT SUMP STATION

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE CHK'D	REV'D

WSP USA Inc.
1301 Fifth Avenue
Suite 1200
Seattle, WA 98101-2677
TEL: 206-357-5600
FAX: 206-357-5601

PRELIMINARY CONCEPT FIGURE

INITIALS AND DATE	INITIALS AND DATE
DESIGNED	REVIEWED:
CHECKED	DES. CONST.
	SDOT PROJ. MGR.
DRAWN	RECEIVED
CHECKED	REVISED AS BUILT

ALL WORK SHALL BE DONE IN ACCORDANCE WITH THE CITY OF SEATTLE STANDARD PLANS AND SPECIFICATIONS AND OTHER DOCUMENTS CALLED FOR IN SECTION 0-02.3 OF THE PROJECT MANUAL.

Seattle Department of Transportation

ORDINANCE NO. PW NO.
FUND:
SCALE:

WEST SEATTLE HIGH RISE BRIDGE
COST BENEFIT ANALYSIS

IMMERSED TUNNEL STUDY
TYPICAL MECHANICAL SECTIONS

JOB PC CO
VPI #

FIGURE A-7

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Sep-15-20 10:24am

ATTACHMENT B

ROW Impacts and Parcel Map

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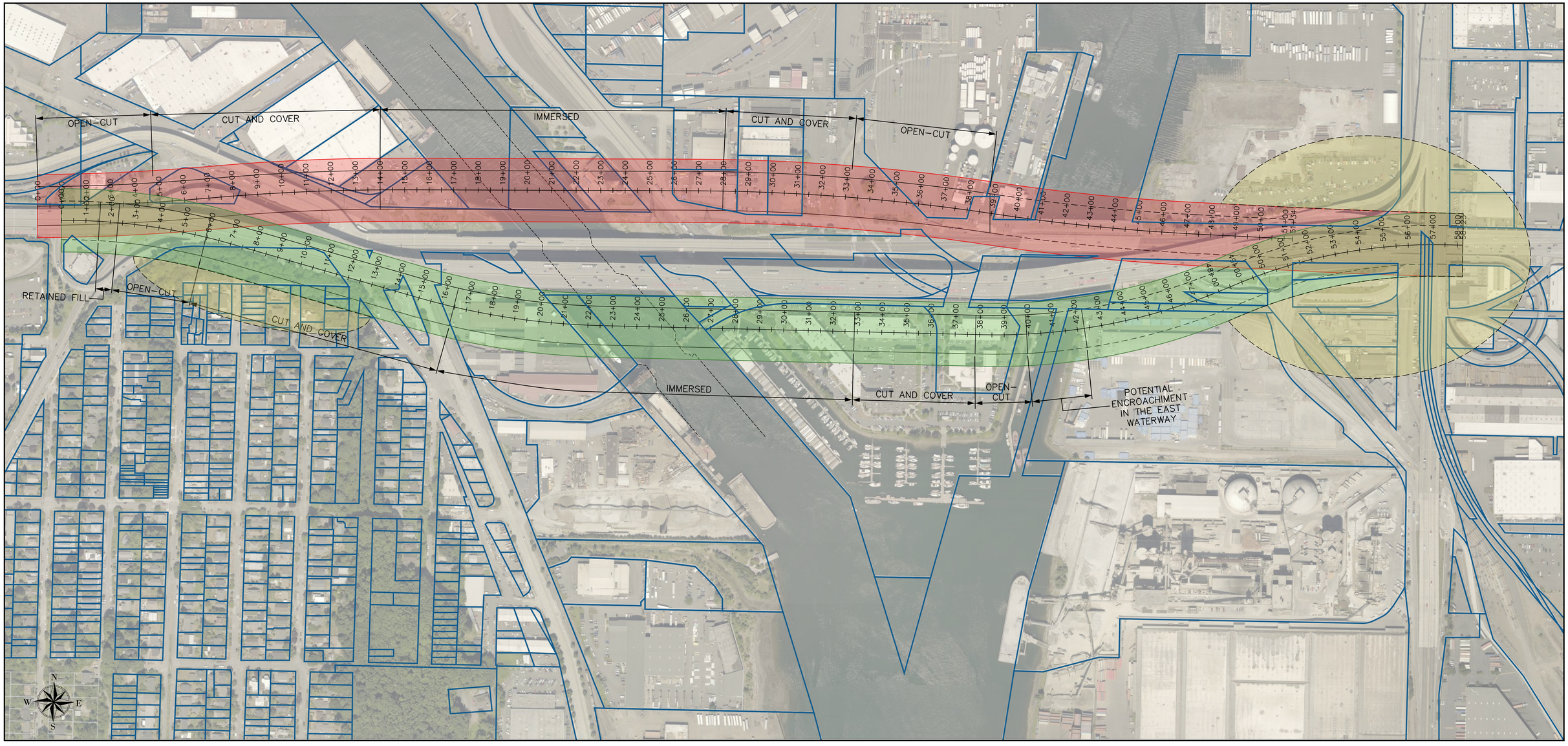

KEY

- = Parcel Boundary
- = Tunnel Alignment
- = South Alignment Impacts
- = North Alignment Impacts
- = Other Impacts
  - *1) Slope at Pigeon Point for South Alignment*
  - *2) SR 99 Interchange for both alignments*

PLAN

SCALE: 1"=200'

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE CHK'D	REV'D

WSP USA Inc.
1301 Fifth Avenue
Suite 1200
Seattle, WA 98101-2677
TEL: 206-357-5600
FAX: 206-357-5601

PRELIMINARY CONCEPT FIGURE

INITIALS AND DATE	INITIALS AND DATE
DESIGNED	REVIEWED:
CHECKED	DES. CONST.
	SDOT PROJ. MGR.
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Seattle Department of Transportation

ORDINANCE NO. PW NO.
FUND:
SCALE:

WEST SEATTLE HIGH RISE BRIDGE
COST BENEFIT ANALYSIS

IMMERSED TUNNEL STUDY
ROW IMPACTS & PARCEL MAP

JOB PC CO
VPI #

FIGURE B–1

APPENDIX M

FUNDING OPTIONS MATRIX

SUMMARY

This appendix contains the evaluation of funding and financing options at the federal, state, and local level to compare the likelihood of funding for the rehabilitate or replace alternatives, based on the nature of each design.

This attribute seeks to answer the questions:

- What funding will be available?
- What will the potential funding burdens be on local resources and communities?
- Is this rehabilitate/replace concept eligible for federal, state, local, or emergency funding?

The review focused on near-term (through 2030) funding or financing options. When a definitive answer can be given, such as with eligibility, that will be used. However, many qualitative criteria were reviewed as high (5), medium (3), or low (1).

WSP USA
Suite 300
33301 Ninth Avenue South
Federal Way, WA 98003-2600
wsp.com

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

9/17/2020

Evaluation Criteria / *Potential Revenue Source*	Type of Funding	Local, Regional, Federal	Funding Source	Description	Eligibility							Funding Availability	Revenue	Stability		Likelihood of Funding							Timing		Administrative		Legal Authority		Equity	
					Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Eligibility Details (if the project is deemed ineligble for funds, the source will not be evaluated)			Score	Detail	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Detail	Score	Detail	Score	Detail	Score	Detail	Score	Detail
Infrastructure for Rebuilding America (INFRA) Formerly FASTLANE	Discretionary Grant	Federal	FHWA	The Infrastructure for Rebuilding America (INFRA) grant program (formerly known as FASTLANE) is a USDOT grant program focusing on nationally significant freight and highway transportation projects that meet four key objectives: (1) support economic vitality at the national and regional level; (2) leverage Federal funding to attract other, non-Federal sources ; (3) use innovative approaches to improve safety and expedite project delivery; and (4) performance accountability. As authorized by the FAST Act, these funds will support primarily freight and highway projects of national or regional significance, but also intermodal projects. In FY 2019, 20 projects were awarded $855 million.	Y	Y	Y	Y	Y	Y	SDOT is an eligible applicant and the WSHB is an eligible project. Given its proximity to the Port of Seattle and its importance to the region, this would likely prove to be a competitive project.	$1 billion in FY20 funds Awards in FY20 ranged from $6.1 million to $135 million	5	3	The FAST Act will be reauthorized for an additional year at FY20 funding levels. It remains to be seen if this program will be included in the future Transportation Bill, and if included, at what amount.	3	1	1	3	3	3	The significant need for this project and it's regional priority will make the WSHB a competitive project, however the program is extremely competitive so it cannot be guaranteed that this project will secure funding. The repair only options will not fare as well, as the BCA will not show long-term user benefits.	5	If awarded, funding would be obligated within 12 months of award.	5	SDOT has the capability to oversee a grant application and handle the reporting requirements, however it does require effort and coordination at the federal, state, and local level for environmental approvals and match requirements.	5	SDOT is an eligible applicant and has experience overseeing similar grants.	5	Discretionary funds appropriated to a federal government agency by Congress does not discernibly result in disproportionate taxation effects on the local populations.
Better Utilizing Investments to Leverage Development (BUILD)	Discretionary Grant	Federal	USDOT	The Better Utilizing Investments to Leverage Development (BUILD) grant program (formerly known as TIGER) is a highly competitive USDOT grant program which supports the capital costs of road, rail, transit, and port projects that have a significant impact on the nation, a region, or a metropolitan area. The program is extremely competitive, in FY19 USDOT selected 91 out of 851 applications, leaving $9.5 billion in requests unfunded.	Y	Y	Y	Y	Y	Y	SDOT is an eligible applicant and the WSHB is an eligible project	$1 billion in FY20 funds Max award in FY19 was $25 million	1	3	BUILD is not authorized under the FAST Act and is not guaranteed to be reauthorized each year.	3	1	1	3	3	3	The significant need for this project and it's regional priority will make the WSHB a competitive project, however the program is extremely competitive so it cannot be guaranteed that this project will secure funding. The repair only options will not fare as well, as the BCA will not show long-term user benefits.	5	If awarded, funding would be obligated within 12 months of award.	5	SDOT has the capability to oversee a grant application and handle the reporting requirements, however it does require effort and coordination at the federal, state, and local level for environmental approvals and match requirements.	5	SDOT is an eligible applicant and has experience overseeing similar grants.	5	Discretionary funds appropriated to a federal government agency by Congress does not discernibly result in disproportionate taxation effects on the local populations.
Advanced Transportation and Congestion Management Technology (ATCMTD)	Discretionary Grant	Federal	FHWA	Annual discretionary grant program for the development of model deployment sites for large scale installation and operation of advanced transportation technologies to improve safety, efficiency, system performance, and infrastructure return on investment. The program is particularly focused on the areas of multimodal integrated corridor management; installation of connected vehicle technologies at intersections and pedestrian crossing locations; unified fare collection and payment systems; freight community systems; technologies to support connected communities; infrastructure maintenance, monitoring, and condition assessments; and rural technology deployments.	Y	Y	Y	Y	Y	Y	Only certain technological elements of the bridge will be eligible for these funds that can show they are using advanced technology, such as dynamic tolling, dynamic signage, or connected and automated vehicle technology	$60 million for FY20 Max award of 20% of total award pool ($12 million)	1	3	The FAST Act will be reauthorized for an additional year at FY20 funding levels. It remains to be seen if this program will be included in the future Transportation Bill, and if included, at what amount.	1	1	1	1	1	1	The WSHB could receive funding if it includes certain technology elements funded through this program.	5	If awarded, funding would be obligated within 12 months of award.	5	SDOT is an eligible applicant and has experience overseeing similar grants.	5	SDOT is an eligible applicant and has experience overseeing similar grants.	5	Discretionary funds appropriated to a federal government agency by Congress does not discernibly result in disproportionate taxation effects on the local populations.
Surface Transportation System Funding Alternative (STSFA)	Discretionary Grant	Federal	FHWA/FTA	The FAST Act established the Surface Transportation System Funding Alternatives Program to provide grants to States or groups of States to demonstrate user-based alternative revenue mechanisms that utilize a user fee structure to maintain the long-term solvency of the Highway Trust Fund. There is only $20 million authrorrized for this program in FY20.	N	N	N	N	N	N	This funding is not intended to fund capital projects like the WSHB	N/A	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

9/17/2020

Evaluation Criteria / *Potential Revenue Source*	Type of Funding	Local, Regional, Federal	Funding Source	Description	Eligibility							Funding Availability	Revenue	Stability		Likelihood of Funding							Timing		Administrative		Legal Authority		Equity	
					Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Eligibility Details (if the project is deemed ineligble for funds, the source will not be evaluated)			Score	Detail	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Detail	Score	Detail	Score	Detail	Score	Detail	Score	Detail
Marine Highway Program (MHP)	Discretionary Grant	Federal	MARAD	MARAD's Marine Highway Program (MHP) has one major goal: expand the use of America's navigable waters. MARAD works closely with public and private organizations to develop and expand marine highway service options and facilitate their further integration into the current U.S. surface transportation system, especially where water-based transport is the most efficient, effective and sustainable option. MARAD's Marine Highway Program also highlights the benefits, increase public awareness and promote waterways as a viable (in some cases a superior) alternative to "landside" shipping and transportation options. There is $9.5 million authrorized for FY21.	N	N	N	N	N	N	The WSHB project does not cross over a designated Marine Highway Project, making it ineligible for these funds.	N/A	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
State Homeland Security Program (SHSP)	Formula	Federal	FEMA	The objective of the Homeland Security Grant Program (HSGP) is to provide funds to eligible entities to support state, local, tribal, and territorial efforts to prevent terrorism and other catastrophic events and to prepare the Nation for the threats and hazards that pose the greatest risk to the security of the United States. The State Homeland Security Program (SHSP) supports preparedness activities that support terrorism preparedness.	Y	Y	Y	Y	Y	Y	SDOT is an eligible applicant and can make a case that the WSHB is an eligible project by highlighting its proximity to the Port of Seattle and including the WSHB in its Threat and Hazard and Risk Assesment.	$6.7 million to Washington State in FY20	1	3	It is unknown what future year funding amounts will be, but it is likely this program, or a similar one will continue to be offered.	1	1	1	1	1	1	The Project does not align with the competitive criteria. It may receive some funding for elements of the bridge, such as closed circuit television (CCTV) cameras, lighting, and monitoring threats. To receive funding SDOT will have to show how this bridge is critical to the Seattle's Threat and Hazard Identification and Risk Assessment. Projects must have a	5	This funding will be available quickly after award.	3	This application submission and reporting process will require close coordination with FEMA	5	SDOT is an eligible applicant and has the authority to apply.	5	Discretionary funds appropriated to a federal government agency by Congress does not discernibly result in disproportionate taxation effects on the local populations.
Urban Area Security Initiative (UASI)	Formula	Federal	DHS/FEMA	The Urban Area Security Initiative (UASI) Program assists high-threat, high-density Urban Areas in efforts to build, sustain, and deliver the capabilities necessary to prevent, protect against, mitigate, respond to, and recover from acts of terrorism.	Y	Y	Y	Y	Y	Y	SDOT is an eligible applicant and can make a case that the WSHB is an eligible project by highlighting its proximity to the Port of Seattle and including the WSHB in its Threat and Hazard and Risk Assesment.	$5- 6.2 million to the Seattle region in FY20	1	3	It is unknown what future year funding amounts will be, but it is likely this program, or a similar one will continue to be offered.	1	1	1	1	1	1	The Project does not align with the competitive criteria. It may receive some funding for elements of the bridge, such as closed circuit television (CCTV) cameras, lighting, and monitoring threats. To receive funding SDOT will have to show how this bridge is critic to the city's Threat and Hazard Identification and Risk Assessment. Projects must have a demonstrated nexus to	5	This funding will be available quickly after award.	3	This application submission and reporting process will require close coordination with DHS/FEMA	5	SDOT is an eligible applicant and has the authority to apply.	5	Discretionary funds appropriated to a federal government agency by Congress does not discernibly result in disproportionate taxation effects on the local populations.
Operation Stonegarden (OPSG)	Formula	Federal	DHS/FEMA	Operation Stonegarden (OPSG) supports enhanced cooperation and coordination among Customs and Border Protection (CBP), United States Border Patrol (USBP), and federal, state, local, tribal, and territorial law enforcement agencies. The program is authorized at $90 million in FY20.	N	N	N	N	N	N	This program is not intended to fund capital projects like the WSHB	N/A	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

9/17/2020

Evaluation Criteria / *Potential Revenue Source*	Type of Funding	Local, Regional, Federal	Funding Source	Description	Eligibility							Funding Availability	Revenue	Stability		Likelihood of Funding							Timing		Administrative		Legal Authority		Equity	
					Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Eligibility Details (if the project is deemed ineligble for funds, the source will not be evaluated)			Score	Detail	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Detail	Score	Detail	Score	Detail	Score	Detail	Score	Detail
Transportation Infrastructure Finance and Innovation Act (TIFIA) Loan	Financing	Federal	USDOT/BAB	Transportation Infrastructure Finance and Innovation Act (TIFIA) Loan program provides important financing debt options (direct loans, loan guarantees, and standby lines of credit) for large projects and public-private partnerships. Broadly speaking, TIFIA provides credit assistance for qualified projects of regional and national significance. Any highway and transit capital project eligible for federal aid is eligible for the TIFIA program. Advantages of the program include low cost of financing and flexibility of repayment terms. Challenges of the program include competitiveness of the program, lengthy negotiations and structuring process, and the cap on the percentage of TIFIA financing. The project must have a dedicated revenue source pledged to secure both the TIFIA and senior debt financing.	Y	Y	Y	Y	Y	Y	SDOT is eligible to apply for these funds if they can identify a revenue source to repay the loan.	$300 million in FY19 and FY20	5	5	The FAST Act will be reauthorized for an additional year at FY20 funding levels. It remains to be seen if this program will be included in the future Transportation Bill, and if included, at what amount.	5	5	5	5	5	5	If SDOT can show a dedicated revenue source to repay the TIFIA loan, SDOT will likely receive this financing	3	A TIFIA loan can be issued in the near term once SDOT is ready to apply, but it may take longer than one year.	3	Overseeing a TIFIA loan will require close coordination with the Build America Bureau at USDOT. This application process will be stringent to ensure SDOT is a worthy borrower.	5	SDOT is eligible to receive this financing	3	The development of a dedicated revenue source to service debt financing may disproportionately affect low-income users, depending on mode of revenue collection and how the income level of the revenue base is considered.
Private Activity Bonds (PABs)	Financing	Federal	USDOT/BAB	Federal Private Loan Program provides favorable financing terms (tax-exempt loans) to private investors such as Private Activity Bonds (PABs)	Y	Y	Y	Y	Y	Y	SDOT is eligible to apply for these funds if they can identify a revenue source to repay the loan.	There is $1.95 billion in bonding capacity available	5	5		5	5	5	5	5	5	If SDOT can show a dedicated revenue source to repay the PAB loan, SDOT will likely receive this financing	3	95% of net proceeds from bonds must be expended within 5 years of issuance.	3	Overseeing a PAB loan will require close coordination with the Build America Bureau at USDOT. This application process will be stringent to ensure SDOT is a worthy borrower.	5	SDOT is eligible to receive this financing	3	The development of a dedicated revenue source to service debt financing may disproportionately affect low-income users, depending on mode of revenue collection and how the income level of the revenue base is considered.
Grant Anticipation Revenue Vehicles (GARVEE) Bonds	Financing	Federal	USDOT/BAB	A Grant Anticipation Revenue Vehicle (GARVEE) bond can be issued by USDOT to cover the costs of immediate construction needs, which will be repaid with known future federal-aid grants for highways from FHWA. Using a Direct GARVEE Bond, SDOT can be reimbursed for interest and issuance costs, but funds must be used for a specific project, such as the WSHB, with FHWA's approval. In CY12 and CY13 USDOT issued $785 million in GARVEE bonds to finance the SR 530 Floating Bridge and Eastide Project connecting Seattle and King County's Eastside communities.	Y	Y	Y	Y	Y	Y	The WSHB is classified as a City Street T-1 Class Strategic Freight Corridor m and as an Intermodal connector on the NHS. These bonds are intended projects that do not have access to another revenue stream (such as taxes or tolls)	Dependent on State FHWA Federal-Aid.	5	5	Federal-aid highway funding is likely to continue, although the amounts are unknown.	3	3	3	3	3	3	SDOT would need to show USDT that it could not find any other reasonable revenue stream to pay for this project, and the State and City of Seattle would need to agree to divert Federal Aid highway funds away from other projects to repay the bond	3	-	3	The City of Seattle will need to work closely with FHWA and WashDOT .	5	-	3	The diversion of funds from other Federal Aid highway projects may result in inequitable conseqeuences, if the existing funds were meant for projects in low-income communities.
COVID Relief Funds	Discretionary Funding	Federal	US Treasury	The Coronavirus Aid, Relief, and Economic Security (CARES) Act provided over $131 million to the City of Seattle, with an additional $166 million to Sound Transit. The City of Seattle may use those funds for any expense it chooses to prioritize. The potential future COVID Relief bill - the Health and Economic Recovery Omnibus Emergency Solutions (HEROES) Act - would give $500 billion to states and $375 billion to local governments. These funds could be used by the City of Seattle to address the need on the WSHB Project, but it is unknown what will be available until the bill is actually passed by the House and Senate.	Y	Y	Y	Y	Y	Y	States and municipalities will receive funding from the CARES act, and will receive additional funding from the future COVID relief bill that has not yet been passed. These future funds - similar to the CARES Act funds, will not have strict eligibility/use requirements attached to them, rather the City of Seattle can choose how to spend these funds. It is possible that the City of Seattle may	$131 million to the City of Seattle from the CARE Act	3	1	Future funding from an additional Coronavirus Relief bill may be released soon, but these payments will not reoccur	3	3	3	3	3	3	This depends on the City of Seattle's priorities.	5	This funding has already been awarded to the City of Seattle and can be made available immediately.	3	This is a new pool of funding, which may cause some internal administrative work, but since it will all be completed internally it is not a heavy burden	5	The City of Seattle can choose how to spend its COVID Relief funds	3	The diversion of funds from other COVID relief fund projects may result in inequitable conseqeuences, if the existing funds were meant for projects or services for low-income communities.

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

9/17/2020

Evaluation Criteria / *Potential Revenue Source*	Type of Funding	Local, Regional, Federal	Funding Source	Description	Eligibility							Funding Availability	Revenue	Stability		Likelihood of Funding							Timing		Administrative		Legal Authority		Equity	
					Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Eligibility Details (if the project is deemed ineligble for funds, the source will not be evaluated)			Score	Detail	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Detail	Score	Detail	Score	Detail	Score	Detail	Score	Detail
Special Federal Funding	Discretionary Funding	Federal	NA	In the past, certain capital projects for transportation could receive special funding through the use of Earmarks. The federal government no longer allows for earmarked funds for special transportation projects, not even ones designated as a State of Emergencies Project.	N	N	N	N	N	N	Special funding for a singular project is no longer set-aside for transportation projects, regardless of a State of Emergency Status	N/A	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	
Congestion Mitigation and Air Quality (CMAQ)	Formula	Federal	FHWA/PSRC	CMAQ provides a flexible funding source to State and local governments for transportation projects and programs to help meet the requirements of the Clean Air Act. Funding is available to reduce congestion and improve air quality for areas that do not meet the National Ambient Air Quality Standards for ozone, carbon monoxide, or particulate matter (nonattainment areas) and for former nonattainment areas that are now in compliance (maintenance areas).	Y	Y	Y	Y	Y	Y	SDOT is an eligible applicant and the WSHB is an eligible project	FY20 - $37 million to WA, $30 million to PSRC	1	3	The FAST Act will be reauthorized for an additional year at FY20 funding levels. It remains to be seen if this program will be included in the future Transportation Bill, and if included, at what amount.	3	3	3	3	3	3	This project will drastically reduce travel times.	5	This funding will be available quickly after award.	1	Known funding source	5	Existing funding source	5	Discretionary funds appropriated to a federal government agency by Congress does not discernibly result in disproportionate taxation effects on the local populations.
Surface Transportation Block Grant Program (STBG)	Formula	Federal	FHWA/PSRC	STBG funds, also known as the Surface Transportation Program (STP), are distributed by FHWA to states and metropolitan planning organizations (MPOs) using a highway-based funding formula. It is a flexible funding source for a range of transportation projects including transit safety infrastructure improvements for existing services and transit capital funding for new projects.	Y	Y	Y	Y	Y	Y	SDOT is an eligible applicant and the WSHB is an eligible project	$127 million allocated to WSDOT, $52.3 million to PSRC in FY20. Individual projects generally receive <$10 million	1	3	The FAST Act will be reauthorized for an additional year at FY20 funding levels. It remains to be seen if this program will be included in the future Transportation Bill, and if included, at what amount.	5	5	5	5	5	5	The flexibility of these funds make all alternatives likely candidates for funding, subject to PSRC determination	5	One year application and grant distribution process.	5	Would require SDOT to submit a grant funding request application.	5	SDOT is an eligible applicant and has received grants in the past.	5	Formula funds appropriated to a federal government agency by Congress does not discernibly result in disproportionate taxation effects on the local populations.
Transportation Alternatives Program (TAP)	Formula	Federal	FHWA/PSRC	The former TAP was replaced by a series of STBG's encompassing a variety of smaller-scale transportation projects such as pedestrian and bicycle facilities, recreational trails, safe routes to school projects, community improvements such as historic preservation and vegetation management, and environmental mitigation related to stormwater and habitat connectivity.	Y	Y	Y	Y	Y	Y	WSHB project would require an active transportation component to compete for funding (such as bike lanes or a pedestrain crossing). However this project is often used for small-scale projects.	$8.4 million allocated to WSDOT, $4.6 million to PSRC in FY20. Individual projects generally receive <$2.5 million	1	3	The FAST Act will be reauthorized for an additional year at FY20 funding levels. It remains to be seen if this program will be included in the future Transportation Bill, and if included, at what amount.	1	1	1	1	1	1	If the chosen alternative includes an active transportation component, PSRC can consider awarding funding.	3	Biennium grant application cycle. FY21 beginning of next cycle.	5	Would require SDOT to submit a grant funding request application to PSRC.	5	SDOT is an eligible applicant and has received grants in the past.	5	Formula funds appropriated to a federal government agency by Congress does not discernibly result in disproportionate taxation effects on the local populations.
WA Freight Mobility Strategic Investment Board (FMSIB)	Discretionary Funding	State	FMSIB	The FMSIB prioritizes and funds improvements on strategic freight corridors, using state allocations as part of the 2-year budget cycle. Program depends on biannual allocation of funds by WA State.	Y	Y	Y	Y	Y	Y	The WSHB is classified as a City Street T-1 Class Strategic Freight Corridor by the FMSIB.	FY19-21: $18 - $23 million available. Individual projects generally receive <$7 million.	1	3	Program funding subject to WA State budget process. Covid-related revenue shortfalls could threaten funding.	5	5	5	5	5	5	Program focused on improvements to strategic freight corridors. The WSHB shutdown has negatively impacted the lower bridge and surrounding arteries, which are heavily used by trucks serving the port and local industrial areas.	3	Biennium grant application cycle. FY21 beginning of next cycle.	5	Would require SDOT to submit a grant funding request application.	5	SDOT is an eligible applicant and has received grants in the past.	5	Funds appropriated to a state government agency by the State Legislature does not discernibly result in disproportionate taxation effects on the local populations.
Transportation Improvement Board (TIB) Urban Arterial Program (UAP)	Formula	State	WSDOT	The Urban Arterial Program (UAP) funds projects on a competitive basis annually based on Safety, Growth & Development, Physical condition, and Mobility criteria.	Y	Y	Y	Y	Y	Y	SDOT is eligible to receive funds and the WSHB Project would be an eligible Project	FY20: $60 million available. FY19: 31 out of 76 applications selected. Individual projects generally receive <$5 million.	1	3	Program funding subject to WA State budget process. Covid-related revenue shortfalls could threaten funding.	3	3	3	3	3	3	Program focused on arterials and sidewalks. Funded project scopes are generally limited to street improvements, road widenings, and safety improvements.	5	One year application and grant distribution process.	5	Would require SDOT to submit a grant funding request application.	5	SDOT is an eligible applicant and has received grants in the past.	5	Funds appropriated to a state government agency by the State Legislature does not discernibly result in disproportionate taxation effects on the local populations.

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

9/17/2020

Evaluation Criteria / *Potential Revenue Source*	Type of Funding	Local, Regional, Federal	Funding Source	Description	Eligibility							Funding Availability	Revenue	Stability		Likelihood of Funding							Timing		Administrative		Legal Authority		Equity	
					Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Eligibility Details (if the project is deemed ineligble for funds, the source will not be evaluated)			Score	Detail	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	Detail	Score	Detail	Score	Detail	Score	Detail	Score	Detail
Future State General Transportation Funding	Discretionary Funding	State	WSDOT	During the November 2019 election, voters passed Initiative 976 - Limits on Motor Vehicle Taxes and Fees Measure (I-976). I-976 has limited the annual license fees for vehicles at $30, basing vehicle taxes on Kelly Blue Book values rather than manufactures' base, and repealing the authorization of transit agencies, such as Sound Transit, to impose motor vehicle excise taxes. Because of its passing, the Governor directed WSDOT to postpone work on capital expansion projects not yet underway to understand the impacts to estimated transportation revenue sources. This means that there it is unlikely at this time that additional revenue from the State will be available.	N	N	N	N	N	N	Based on the pass of I-976 it is unlikely WSDOT has any additional revenue for this Project.	N/A	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Connecting Washington (CW)	Discretionary Funding	State	WSDOT	The Connecting Washington funding package was enacted in 2015 by Washington State Legislature. The revenue package is a 16-year program, funded primarily by an 11.9-cent gas tax increase that was fully phased-in on July 1, 2016. Connecting Washington funds various types of projects including state highways and local roads, state highway maintenance, operations & preservation, non-highway projects (bike paths, walkways, rail and transit), ferries & terminals, and fish barriers. The funding package is a $16 billion investment and roughly ten percent of the funding package ($1.3 billion) is applied towards non-highway projects.	N	N	N	N	N	N	These funds are already programmed out through FY31.	N/A	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
2005 Gas Tax - Transportation Partnership Program	Discretionary Funding	State	WSDOT	The 2005 Transportation Partnership Program (TPA) was enacted in 2005 by the Washington State Legislature. The revenue transportation package funds 274 projects, including at-risk structure projects, safety investment projects, choke points and congestion projects, multimodal improvement projects, environmental projects, and freight mobility & economic projects, over a 16-year period. The funding package includes a 9.5 cents gas tax increase phased in over four years, vehicle weight fee on passenger cars, the light truck weight fee increase, and annual motor home fee. In total, the projects are funding by a $7.1 billion investment, with $94.8 million in funding for multimodal projects.	N	N	N	N	N	N	These funds are already programmed out through FY31.	N/A	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
WSDOT Local Bridge Program	Formula	Federal / State	WSDOT	In Washington, the Advisory Group created a set-aside for a local bridge program that includes funding from the National Highway Partnership Program(NHPP) and Surface Transportation Blok Grants (STBC) for both on and off system bridges totaling $45 million per year, every other year (FY17, F719, FY21). The program will preserve and improve the condition of bridges owned by cities and counties that are physically deteriorated.	Y	Y	Y	Y	Y	N	SDOT is an legible recipient and the WSHB is an eligible project.	$45 million in the FY19 program. Individual projects generally receive <$9 million.	1	3	FY21 funding subject to WA State budget process. Covid-related revenue shortfalls could threaten funding.	3	3	3	3	3	1	Program generally funds bridge projects much smaller in scope than WSHB.	3	Biennium grant application cycle. FY21 beginning of next cycle.	5	Would require SDOT to submit a grant funding request application.	5	SDOT is an eligible applicant and has received state grants in the past.	5	Funds appropriated to a state government agency by the State Legislature does not discernibly result in disproportionate taxation effects on the local populations.

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

9/17/2020

Evaluation Criteria / *Potential Revenue Source*	Type of Funding	Local, Regional, Federal	Funding Source	Description	Eligibility							Funding Availability	Revenue	Stability		Likelihood of Funding							Timing		Administrative		Legal Authority		Equity	
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Regional Mobility Grants	Discretionary Grant	State	WSDOT	Program supports local efforts to improve connectivity between counties and regional population centers, and reduce transportation delay. It is classified as a "Public Transportation" grant program. Funded projects include new transit services, park and ride lots, transportation demand management programs, transit speed and reliability improvements, new buses, and transit service expansion.	Y	Y	Y	Y	Y	Y	Project would likely require a transit component, e.g. a dedicated bus lane. SDOT would need to coordinate with Sound Transit or King County Metro.	$104 million in FY19-21 Biennium. Individual projects generally receive <$9 million.	1	3	Any new funding subject to WA State budget process. Covid-related revenue shortfalls could threaten funding.	1	1	1	1	1	1	Program generally funds smaller-scale transit projects, e.g. increased bus service frequencies, park and rides, and BRT routes.	3	Biennium grant application cycle. FY21 beginning of next cycle.	3	Would require SDOT to submit a grant funding request application and coordinate with Sound Transit/King County Metro.	5	SDOT is an eligible applicant and has received state grants in the past.	5	Funds appropriated to a state government agency by the State Legislature does not discernibly result in disproportionate taxation effects on the local populations.
Move Seattle Levy	Discretionary Funding - Tax Revenues	Local	City of Seattle	Approved by voters in November 2015, the 9-year, $930 million Levy to Move Seattle provides funding to improve safety, maintain streets and bridges, and invest in reliable, affordable travel options. Levy to Move Seattle funds may be re-allocated on an annual basis to improve safety for all travelers, maintain streets and bridges, and invest in reliable, affordable travel options.	Y	Y	Y	Y	Y	Y	This project would be eligible for funding.	9-year, $930 million program approved in FY15. $250-303 million in spending planned for FY20.	5	3	Move Seattle is funded through property tax, which is a stable funding source. However, recent budget cuts to SDOT reduce the agency's ability to fund a major, new project through Levy funds.	5	5	5	5	5	5	Bridge maintenance and replacement is a key goal of the plan, but the WSHB would be competing for funding with many smaller projects.	5	Program priorities and funding are re-evaluated on an annual basis.	5	Existing local funding source for transportation.	5	Existing local funding source for transportation.	3	The Move Seattle Levy is funded by revenues collected as taxes on properties within the levy district; the additional levy costs may represent a disproprtionately higher percentage of household income for low-income households than median-income households, depending if income level is not considered.
External Contribution	Discretionary Funding	Private	N/A	Direct contribution from Port of Seattle/NW Seaport Alliance/King County to fund portions of the bridge. Contributions would come with underlying terms and conditions that may change. These contributions could come from the Port of Seattle or other major employers in the area, such as Microsoft, Amazon, or the cruise lines. The Port of Seattle contributed $281 million to the Alaskan Way Viaduct replacement project in Seattle in 2013, among contributions to other off-port projects to improve traffic and safety on freight and transit corridors throughout the city.	Y	Y	Y	Y	Y	Y	The City of Seattle is able to enter into a funding agreement for the WSHB from an outside party	N/A	3	3	Depends on budget process at external agency or organizaton.	3	3	3	3	3	3	It is not guaranteed that SDOT will collect funds from outside private entities, but this Project is extremely important to the economic connectivity for the city and region, so it is possible.	3	Depends on the agreements	5	This would require little oversight for SDOT	5	SDOT is able to receive funds from the Port of Seattle or other outside contributors.	5	An external contribution would be sourced from existing revenue sources and therefore would be expected to not result in additional charges, fees or taxes to local populations.
Tax-Exempt Municipal Bonds	Financing	Local	City of Seattle	Bonds issued by the City of Seattle; the interest paid to bondholders is exempt from federal taxes. There are two types of municipal bonds: general obligation bonds - backed by the credit and full faith of the issuer - and revenue bonds - backed by a specific revenue source like tolls or taxes. Seattle was assigned Aaa ratings by Moody's for its latest general obligation improvement bond offering in 2019.	Y	Y	Y	Y	Y	Y	The WSHB Project is eligible to receive these funds	The City of Seattle has $2.2 billion of unlimited tax general obligation debt capacity and $2.8 billion of limited tax general obligation debt capacity remaining. Past bonds include: FY19 series A limited tax general obligation improvement bonds offering: $35.87 million FY19 series B bond offering: $11.1 million	5	5	The City of Seattle has significant debt capacity remaining.	3	3	3	3	3	3	SDOT will be able to access this financing if they can either idenifiy a tax revenue source to back a general obligation bond, or a revenue stream for a revenue bond.	3	Subject to City Council vote.	5	The City of Seattle is familiar with the bond issuance process, but it does require administrative oversight.	5	Seattle has authority to issue municipal bonds.	3	The development of a dedicated revenue source to service debt financing may disproportionately affect low-income users or households, depending on mode of revenue collection and how the income level of the revenue base is considered.

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

9/17/2020

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Revenue Generating Mechanisms																														
Public-Private Partnerships (P3)	Partnership	Project Specific	N/A	Public-Private Partnership (P3) is a strategy for procurement which involves a long-term contractual agreement between the government and a private firm targeted towards financing, designing, constructing, maintaining and operating infrastructure facilities and services that were traditionally provided by the public sector. P3 addresses limited funding resources for infrastructure or development projects of the public sector, thereby allowing the allocation of public funds for other local priorities. Two general forms of P3 structures are common: availability payment- and concession-based P3s. WA State's 2005 Transportation Innovative Partnership's Act governs P3s. It requires state-issued debt for P3 toll projects, effectively requiring legislative approval for any tolled P3 project. No tolled P3 projects have occurred in	Y	Y	Y	Y	Y	Y	The City of Seattle is able to enter into a P3 structure.	A P3 would require legislative approval. No P3 toll projects have occurred in WA State to date, making this option uncertain.	5	3	If SDOT enters into a P3 agreement, the City of Seattle will transfer it's financial and operational risk over to a private party. However, even with strict Key Performance Indicators and agreement measures in place, there is always a possibility the WSHB project is not completed as desired.	3	3	3	3	3	3	This would be the first P3 structure of its kind in WA state, which means it may take extra effort to attract the right developers. Additionally, to toll the WSHB (if these is how the deal is structured), it will require special legislative approval.	3	P3 deals can take time to form and approve	5	This new partnership structure may take additional administrative work on the City of Seattle's part to solicit, select, and secure a P3 deal.	5	P3s are authorized in the state.	3	The development of a dedicated revenue stream to support capital and operational costs of P3 projects may disproportionately affect low-income users or households, depending on mode of revenue collection and how the income level of the revenue base is considered.
Tolls	Fee	Project Specific	N/A	Local road tolling permitted under a Regional Transportation Investment District (RTID) or Transportation Benefit District (TBD) structure, but all tolls must be approved by the state Transportation Commission and administered by WSDOT. The Transportation Commission sets toll rates within a set of funding requirements, but any change in tolling rates or authority requires legislative approval. WSDOT is the operator of authorized toll facilities.	Y	Y	Y	Y	Y	Y	The City of Seattle can toll the bridge, with legislative approval.	Amounts unknown, but likely a significant amount each year.	5	3	Subject to traffic levels.	3	3	3	3	3	3	Tolls require legislative approval and SDOT will need to contend with public input, possible making if more difficult to toll the bridge.	1	Actual toll funding will not be available until the WSHB is completed and open, however several the financing sources discussed can leverage future toll revenues to build the bridge.	5	Significant administrative burden	3	Requires approvals	3	The development of a revenue stream from tolling operations to support capital and operational costs may disproportionately affect low-income users or households. A flat fee for all users of the toll facility may represent a higher percentage of household income for low-income communities compared to median income households, thereby restricting access on public road
Impact Fee	Tax	Local	City of Seattle	Impact fees are one-time charges on new development to fund new public capital facilities. Cities can charge impact fees for transportation, parks and recreation, schools, and/or fire facilities. Currently, Seattle does not charge any impact fees. WA State allows local governments to impose impact fees to fund public improvements. Ongoing discussion in Seattle about implementing impact fees.	Y	Y	Y	Y	Y	Y	The City of Seattle does not currently charge these.	Would depend on the fee structure approved by the City Council. Uncertain whether new development is feasible/ongoing in the areas impacted by the WSHB.	5	1	Impact fees are one-time fees on new development -- highly dependent on the real estate cycle.	1	1	1	1	1	1	It is not likley the City of Seatltle will chose to use this mechanism, as much of the areas surrounding the WSHB is already developed.	3	It would take some time to implement this fee and see the revenues from future development.	1	The City of Seattle has the ability to implement impact fees, but would need to design a collection mechanism.	5	Seattle has the ability to implement impact fees.	3	Impact fees result in higher costs for constructing residential and commercial developments, which may result in housing affordability issues for low-income communities.
Property Tax	Tax	Local	King C ounty	Property taxes are collected by the county. There is a 1% annual levy increase limit without voter approval. Voter approval is necessary for increases beyond the levy limit. These funds are already a possibility to fund the WSHB through the City of Seattle's general revenues.	Y	Y	Y	Y	Y	Y	The City of Seattle is able to use these funds for the WSHB, and can also increase the levy limit.	A $0.15 per $1,000 countywide levy would net $7 - $12 million.	3	5	Property tax revenues are subject to the real estate cycle.	1	1	1	1	1	1	These funds are already being collected and may be used via general funds to help this project. Any increase in an effort to raise funds specifically for this project is unlikely.	3	A substantial increase would require voter approval.	5	Property tax is already collected at the county level.	3	Legal authority exists at the county-level.	1	Property taxes are calculated from the market value of properties and represent a disproprtionately higher percentage of household income for low-income households than median-income households, if income level is not considered.

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

9/17/2020

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Transportation Benefit District (TBD)	Financing Mechanism	Local	City of Seattle /King County	A city or county may form a TBD to generate revenue for transportation projects; revenue typically comes from a sales tax or drivers license fee. Seattle has a $80 car license fee and 0.10% sales tax as part of a city TBD, which expires in 2021. A referendum to extend the 0.10% sales tax through 2026 is on the November 2020 ballot. The existing TBD has primarily funded bus service expansions in and around the city, with more limited funding for transit corridor improvement projects, street maintenance, and bicycle/pedestrian improvements.	Y	Y	Y	Y	Y	Y	Seattle can create a new TBD	6-year, $45 million per year program approved in CY14; program expires in FY2021. 6-year, $27 million per year extension of existing sales tax levy on the November 2020 ballot	1	3	Renewal of the sales tax portion of the TBD is subject to voter approval in November 2020. Sales tax revenues subject to economic cycle.	1	1	1	1	1	1	A major bridge repair or replacement projects appears to be outside of the scope of the TBD, which generally funds improved and expanded bus service and street maintenance along transit corridors.	5	Program budget adopted on an annual basis.	5	The City of Seattle has an existing TBD.	3	Seattle has an existing TBD, subject to voter re-approval in November, 2020.	1	A Transportation Benefit District (TBD) is supported by revenues from license fees and sales tax revenues; the additional fees and taxes represent a higher percentage of household income for low-income communities, as compared to higher-income communities.
Vehicle Registration/Licensing Fees	Fee	Regional / Local	City of Seattle	The City of Seattle has an existing $80 car license fee as part of the city's Transportation Benefit Distirct (TBD), which will expire in 2021 and not be renewed due to uncertainty over the legality of local car license fees (due to I-976). If I-976 is not upheld, the city could enact a fee of up to $50 per vehicle without voter approval, and a fee of up to $100 with voter approval, but that is not feasible at this time.	Y	Y	Y	Y	Y	Y	Additional revenues from these fees will	TheCity of Seattle's existing TBD, funded by a $80 vehicle license fee and 0.10% sales tax, provides ~$45 million per year towards transportation.	1	1	I-976 initiative would eliminate vehicle registration/license fees as source of local revenue, pending court order.	1	1	1	1	1	1	Any funding certainty depends on outcome of I-976 legal challenge. If I-976 is not upheld, the City of Seattle would need to re-implement a car license fee, either through city council vote (if <$50) or through a referendum. After those steps, the WSHB would be a likely candidate for funding.	1	Any funding certainty would depend on outcome of I-976 legal challenge. The City of Seattle would then need to re-implement a car license fee, either through city council vote (if <$50) or through a referendum.	1	To create a new vehicle fee would require overturning I-976	1	Existing funding source at local level, but future viability in question due to legal challenges.	3	The calculation and collection of vehicle registration or licensing fees determine whether they disproprtionately affect low-income communities; flat fees may represent a higher percentage of their household income, while fees based on the value of the vehicle or the income level of the user provide a more balanced distribution of costs across all users.
Local Improvement District (LID)	Financing Mechanism	Local	City of Seattle /King County	A Local Improvement District (LID) is sponsored by a city or county and approved by both the local government and benefiting property owners for the purposes of financing local improvements. Improvements are funded by a one-time levy on benefiting property owners; property owners can elect to pay the levy over 20 years, with an interest rate based on the interest rate on the LID bonds issued by the City of Seattle. A local example is the Seattle Waterfront Improvement District, established in 2019 by the city to finance $200 million in improvements along the waterfront through a special assessment on downtown properties.	Y	Y	Y	Y	Y	Y	A new LID would need to be created and a bridge repair or replacement project would fall outside of the usual scope of LIDs, which are generally limited to new parks or streetscape improvements.	N/A	5	5	LIDs, when established, are a stable financing option.	1	1	1	1	1	1	Dependent on political process (a new LID would need to be established, requiring majority of affected property owners' approval).	5	When established, LIDs provide immediate bond financing.	1	Would require the establishment of a new LID.	5	Legal authority for the City of Seattle to establish a new LID exists.	3	A Local Improvement District (LID) is supported by bond financing paid by levy revenues on property values within the district area; the additional levy fees may represent a higher percentage of household income for low-income households within the district, as compared to higher-income households.

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

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Road Improvement District (RID)	Financing Mechanism	Local	King County	A variant of an Local Improvement District (LID) RIDs are special tax assessment districts sponsored by a county for the purposes of road maintenance and improvement. A special tax assessment on local properties directly benefiting from an improvement must be approved by the benefiting property owners (means of approval can vary).	Y	Y	Y	Y	Y	Y	A new RID would need to be created and a bridge repair or replacement project would fall outside of the usual scope of RIDs, which are generally limited to roadway maintenance and minor road improvements (e.g. paving).	N/A	5	5	RIDs, when established, are a stable financing option.	1	1	1	1	1	1	Dependent on political process (a new RID would need to be established, requiring majority of affected property owners' approval).	1	These funds do not exist yet, but if established, RIDs provide immediate bond financing.	1	Would require the establishment of a new RID.	3	An RID would require sponsorship by King County.	3	A Road Improvement District (RID) is supported by tax revenues on property values within the district area; the additional proerty fees may represent a higher percentage of household income for low-income households within the district, as compared to higher-income households.
Community Facility District (CFD)	Financing Mechanism	Local	City of Seattle/King County	CFDs are special districts designed to finance infrastructure improvements. A district may include one or more noncontiguous tracts, lots, parcels, or other properties meeting the requirements of this chapter. Formation requires a petition from 100% of affected landowners. The city or county legislature must approve creation of the CFD, which is then governed by a board consisting of legislators and property owners. One private party could pay for improvements, e.g. a large company, in exchange for a special property assessment.	Y	Y	Y	Y	Y	Y	A new CFD would need to be created.	N/A	5	5	CFDs, when established, are a stable financing option.	1	1	1	1	1	1	Dependent on political process/outreach to private parties (a new CFD would need to be established, requiring *100%* of affected property owners' approval).	1	These funds do not exist yet, but if established, CFDs provide immediate bond financing.	1	Would require the establishment of a new CFD.	3	Legal authority for property owners in the City of Seattle to establish a new CFD exists, but would require their buy-in.	3	A Community Facility District (CFD) is supported by bond financing paid by levy revenues on property values within the district area; the additional levy fees may represent a higher percentage of household income for low-income households within the district, as compared to higher-income households.
Regional Transportation Investment District (RTID)	Financing Mechanism	Local	King County	King, Pierce, and Snohomish Counties, or each county individually, can form a special district to plan and finance certain highway improvements, subject to voter approval. Up to 0.1% in sales tax or $100 in vehicle licensing fees, special fuel tax, parking tax, etc. could be implemented. No RTID currently exists in the Puget Sound region.	Y	Y	Y	Y	Y	Y	A new RTID would need to be created. Eligible projects are generally limited to capital improvements along highways of statewide significance (HSS); the West Seattle High Bridge is not a HSS.	N/A	3	3	RTIDs, if approved by voters, are a stable financing option, but revenues come from sources that are subject to market cycles (sales tax, fuel tax).	1	1	1	1	1	1	A new RTID would need to be approved by voters. The West Seattle :High Bridge would need to be designated as an HSS. In 2007 a proposal for a regional RTIF was defeated at the ballot box.	1	These funds do not exist yet, but if established, RTIDs provide immediate bond financing.	1	Would require a new tax and creation of a King County or regional RTID.	3	Legal authority exists at the county-level.	3	A Regional Transportation Investment District (TBD) is supported by revenues from license fees and sales tax revenues; the additional fees and taxes represent a higher percentage of household income for low-income communities, as compared to higher-income communities.
Sales Tax	Tax	Local	City of Seattle/ King County	A TBD permits up to a 0.2% local sales tax; a RTID a tax of up to 0.1%. Both would require ballot initiatives and approval.	Y	Y	Y	Y	Y	Y	These proceeds could be used for the WSHB Project	County-wide: a 0.1% increase could net $5 - $15 million.	3	3	Sales tax revenues are subject to economic cycle.	3	3	3	3	3	3	3	1	Subject to voter approval. This would take time to get approved and enact.	5	Sales tax is already collected at the county level.	1	Legal authority exists at the county-level.	1	Sales taxes represent a flat fee applied to the sale of goods and services; sales taxes represent a higher percentage of household income for low-income communities, as compared to higher-income households.

West Seattle High-Rise Bridge (WSHB) - Evaluation of Funding Options

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Fuel Tax	Tax	Local	King County	May impose an increase at a rate of up to 10% of the current state tax ($0.0494 per gallon), but would require a ballot measure for voter approval. Can be implemented under aRegional Transporation Investment District (RTID) structure or under a Special Fuel Tax for Counties structure.	Y	Y	Y	Y	Y	Y	These proceeds could be used for the WSHB Project	A county-wide $0.01 fuel tax per gallon could net approx. $2 - $5 million.	1	3	Fuel tax revenues are subject to economic cycle and vehicle efficiency changes.	3	3	3	3	3	3	Dependent on political process (voter approval would be necessary).	1	Subject to voter approval. This would take time to get approved and enact.	1	A new local tax would be necessary.	1	Legal authority exists at the county-level.	1	Fuel taxes represent a flat fee applied to the sale of gasoline fuel; fuel taxes represent a higher percentage of household income for low-income communities, as compared to higher-income households.
"Jump Start" or Head Tax	Tax	Local	City of Seattle	Seattle's City Council passed a Jumpstart Tax in July, 2020 that levies taxes based on a company's highly-paid employees' payroll expenses. Short-term spending priorities include housing security, food security, and small business assistance. Long-term revenues will primarily go towards affordable housing, small business assistance, and preserving city services. The program recieves $214 annually.	N	N	N	N	N	N	Transportation and infrastructure is not within the scope of the JumpStart Tax's funding priorities.	N/A	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

APPENDIX N

WORKSHOP PRESENTATIONS

SUMMARY

This appendix contains the presentations presented to the CITY and select stakeholders and committees.

Workshop No. 01 – July 16, 2020
The objective of this workshop was to solicit input from the CITY and select stakeholders and committees on the alternatives developed, the evaluation process, and the defined attributes and attribute weighting, in order to have an agreed upon structure for conducting the Cost Benefit Analysis.

Workshop No. 02 – August 20, 2020
The objective of this workshop was to solicit input from the CITY, Technical Advisory Panel (TAP), and Community Task Force (CTF) on the performance evaluation and to-date cost information.

Workshop No. 03 – September 8, 2020
Presented the results from the input made by the CITY, TAP and WSP in regards to the Attribute Weighting and Performance Scores. Reviewed the capital cost parameters for the rough order of magnitude cost ranges.

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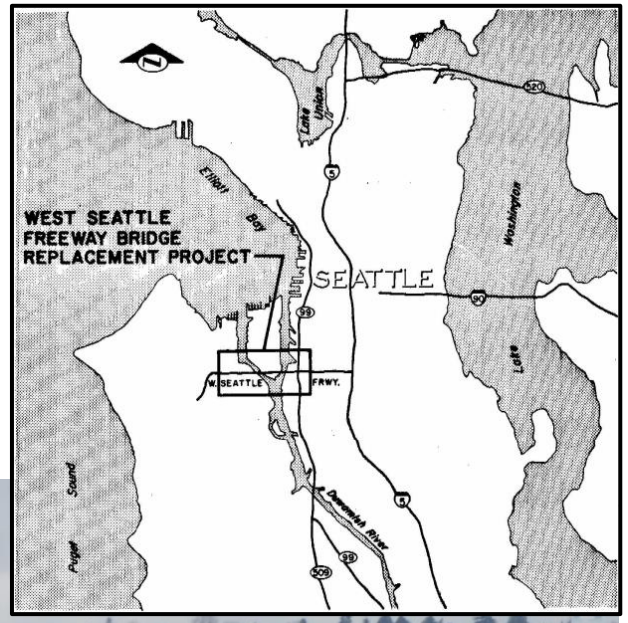
WORKSHOP PRESENTATIONS

WORKSHOP NO. 1

West Seattle High-Rise Bridge

Cost Benefit Analysis – Workshop No. 01



Client: City of Seattle | Location: Seattle, WA | Date: 16 July 2020

Agenda

West Seattle High-Rise Bridge

- Workshop Objective
- Cost-Benefit-Analysis Framework
- Alternative Definition
- Evaluation Process
- Attributes (Value Definition)
- Action Items

Pre-workshop Provided Materials

West Seattle High-Rise Bridge

- Alternative Definition Figures
- Evaluation Matrix
- Capital Cost and BLCCA Spreadsheets
- Risk Spreadsheet
- Basis of Design
- Preliminary Geotechnical Information

Agenda

West Seattle High-Rise Bridge

- **Workshop Objective**
- Cost-Benefit-Analysis Framework
- Alternative Definition
- Evaluation Process
- Attributes (Value Definition)
- Action Items

Objective

West Seattle High-Rise Bridge

- **Cost Benefit Analysis -** be prepared to make informed decisions related to further rehabilitation of the existing bridge or pivoting towards replacing the bridge.

- **Workshop No. 01 –** solicit input from the CITY and select stakeholders and committees on the alternatives developed, the evaluation process, and the defined attributes and attribute weighting, in order to have an agreed upon structure for conducting the CBA.

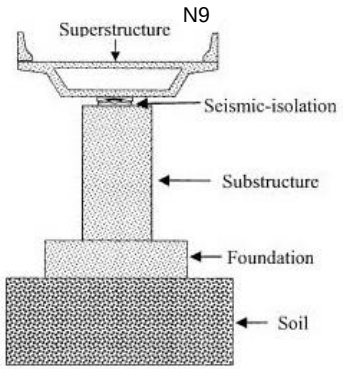
Agenda

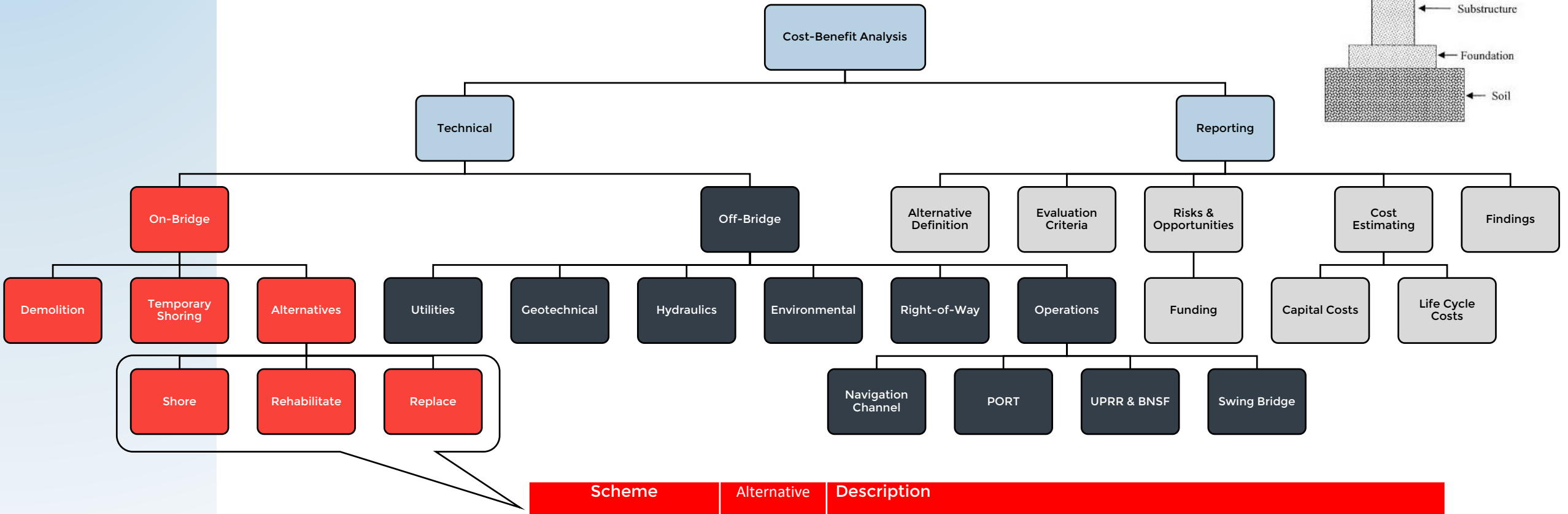
West Seattle High-Rise Bridge

- Workshop Objective
- **Cost-Benefit-Analysis Framework**
- Alternative Definition
- Evaluation Process
- Attributes (Value Definition)
- Action Items

Cost Benefit Analysis - Framework

West Seattle High-Rise Bridge





Scheme	Alternative	Description
Shore	1	Temporary Shoring to Restore Live Load (3 to 5 years)
Rehabilitate	2	Direct Rehabilitation to Restore Live Load (15 to 40 years)
	3	Partial Superstructure Replacement to Restore Live Load (15 to 50 years)
Replace "In-Kind"	4	Accelerated Superstructure Replacement (50 to 75 years)
	5	Accelerated Bridge Replacement (75+ years)

Note:
Arterial/corridor planning study assuming bridge is rehabilitated not considered as part of RP-CBA.

Cost Benefit Analysis – Framework

West Seattle High-Rise Bridge

- Demolition Methods
  - Conventional vs controlled detonation
- Temporary Shoring Methods
  - Restore LL, For Demolition, For Expansion Piers, Construction Activity, Etc.
- Rehabilitation Schemes
  - Partial Replacement (remove and replace a portion of the superstructure)
  - Direct Rehabilitation (PT + CFRP)
  - Preliminary seismic assessment to identify vulnerabilities and potential means to retrofit
- Replacement Schemes
  - Superstructure Only - compare CIP box to original steel tub girder alternate
  - In-Kind complete replacement - feasibility w/ no modifications to approaches
  - Preliminary seismic assessment of a critical structure (applied equally to existing and new elements)

Cost Benefit Analysis – Framework

West Seattle High-Rise Bridge

- Assumptions:
  - Replacement structure will be on alignment, grade/profile, and not require replacement of the approach structures.
  - CBA will not quantitatively assess off-alignment bridge replacement alternatives of the high-rise and approaches.
  - Preliminary geotechnical input is limited to pile axial capacities, preliminary lateral soil design parameters (including considerations of liquefaction), and providing preliminary seismic hazard information.
  - Preliminary corrosion assessment will define assumptions for the condition of the existing foundations.
  - Seismically, a replacement bridge will be considered essential.

Cost Benefit Analysis – Framework

West Seattle High-Rise Bridge

- Assumptions:
  - Direct potential rehabilitation concepts to be evaluated include:
    - External Post-Tensioning
    - Carbon Fiber Reinforced Polymers (CFRP)
  - Partial replacement concepts include steel tubs and concrete box girders
  - City will provide historic repair and rehabilitation, and operating and maintenance costs.
  - Assumed “year one” from BLCCA is the year 2022

Cost Benefit Analysis – Framework

West Seattle High-Rise Bridge

- City led activities…
  - Public outreach and engagement
  - Community and technical stakeholder meetings
  - Traffic Assessment
  - Right-of-Way Services
- Exclusions…
  - Hydraulic Assessment
  - Site explorations (geotechnical, survey, etc)
  - OHWM determination
  - Civil Design Services

Agenda

West Seattle High-Rise Bridge

- Workshop Objective
- Cost-Benefit-Analysis Framework
- **Alternatives Definition**
- Evaluation Process
- Attributes (Value Definition)
- Action Items

Alternative Definition

West Seattle High-Rise Bridge

- Alternatives 1 – 5 will each be investigated as part of the CBA
- Each alternative may consist of multiple items of work as part of BLCCA (year range 2022 to 2100)

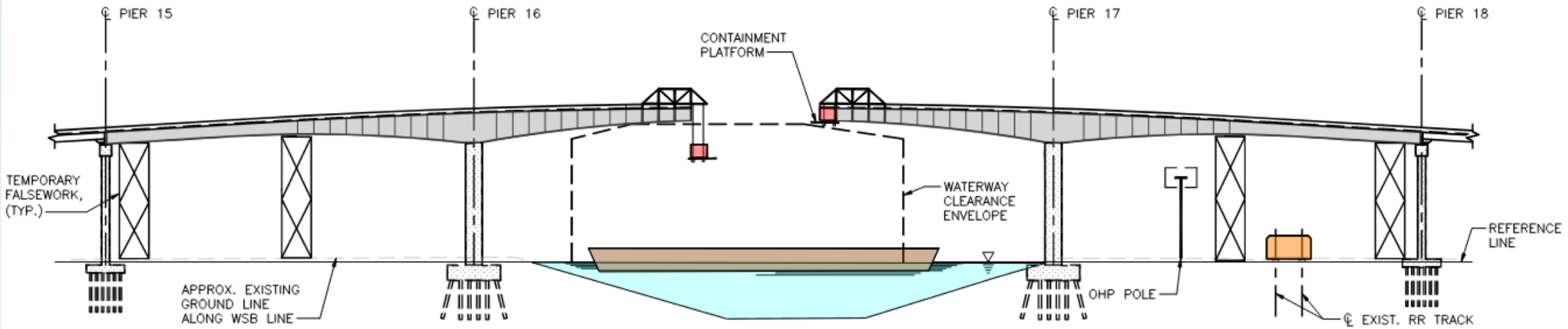
Description of Work	Alt #1		Alt #2			Alt #3			Alt #4		Alt #5	
Year	2022	2027	2022	2032	2062	2022	2032	2062	2025	2075	2025	2075
Demolition Concept		X			X	X		X	X		X	
Shoring Concept (for restoring LL)	X											
Direct Strengthening			X							X		X
Partial Superstr. Replacement						X						
Full Superstructure Replacement									X			
Foundation Strengthening				X			X		X			
“In-Kind” Bridge Replacement		X			X			X			X	

Alternative Definition – Demolition Concept

N16

West Seattle High-Rise Bridge

Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
Demolition Concept	X	X	X	X	X

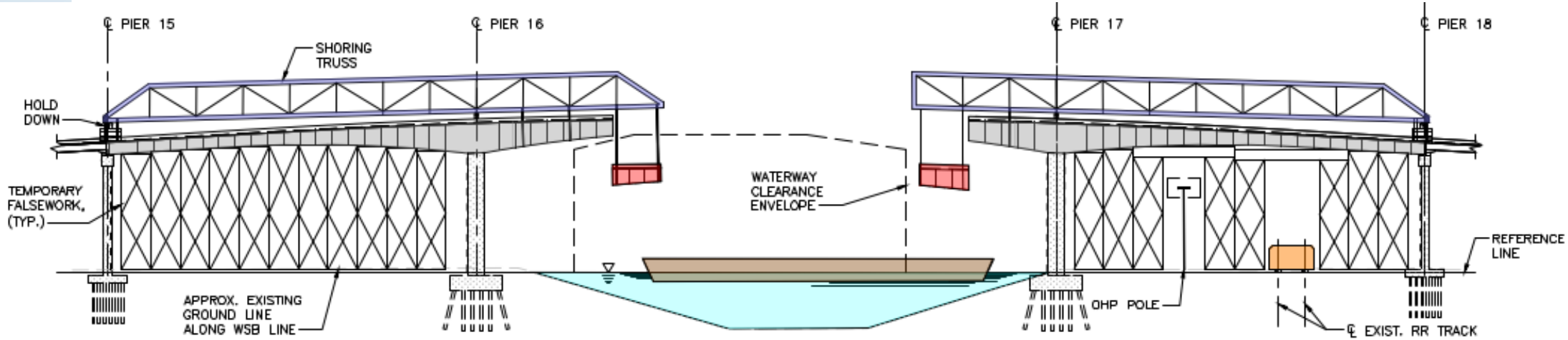


Standard Demolition – Concept 1

Alternative Definition – Demolition Concept

West Seattle High-Rise Bridge

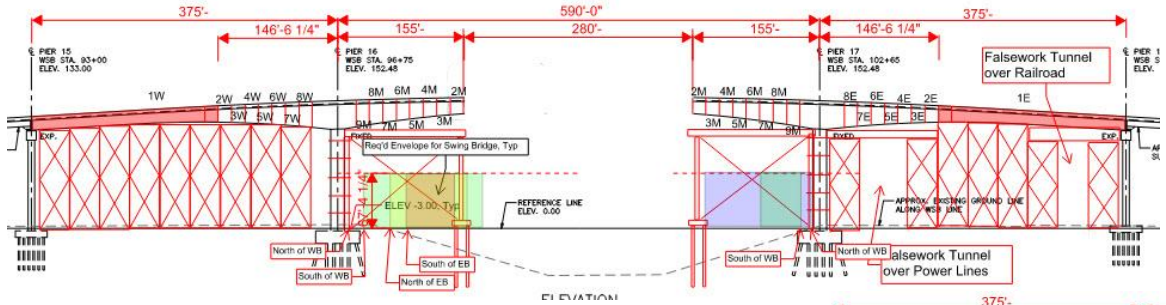
Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
Demolition Concept	X	X	X	X	X



Demolition – Concept 2

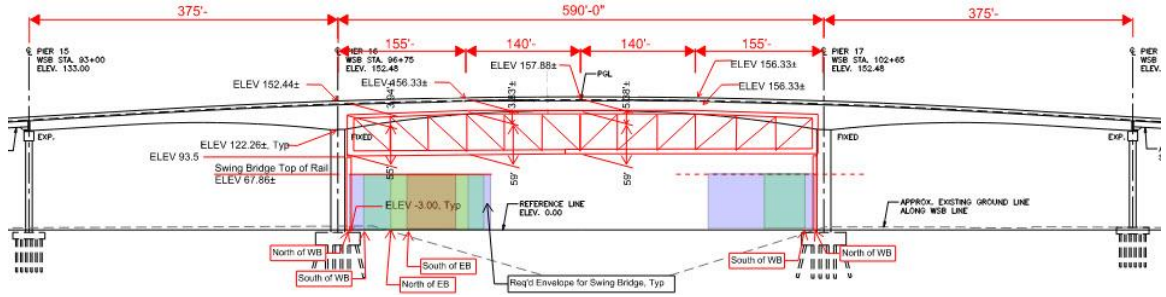
Demolition Concepts Considered
West Seattle High-Rise Bridge

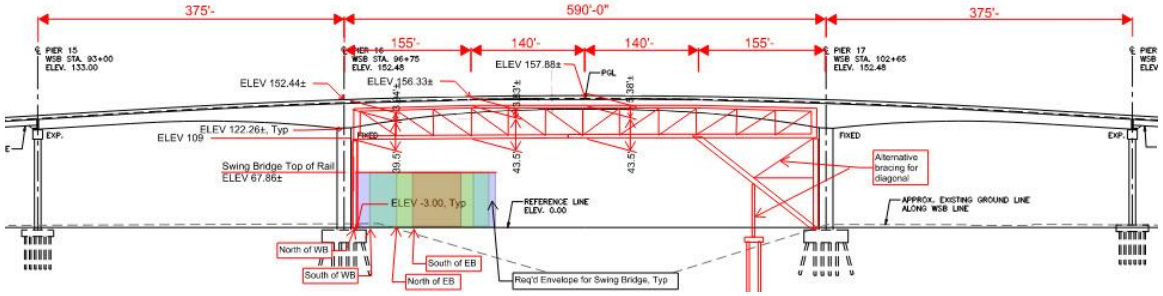
Concepts Considered:



In-water shoring

59’ deep truss



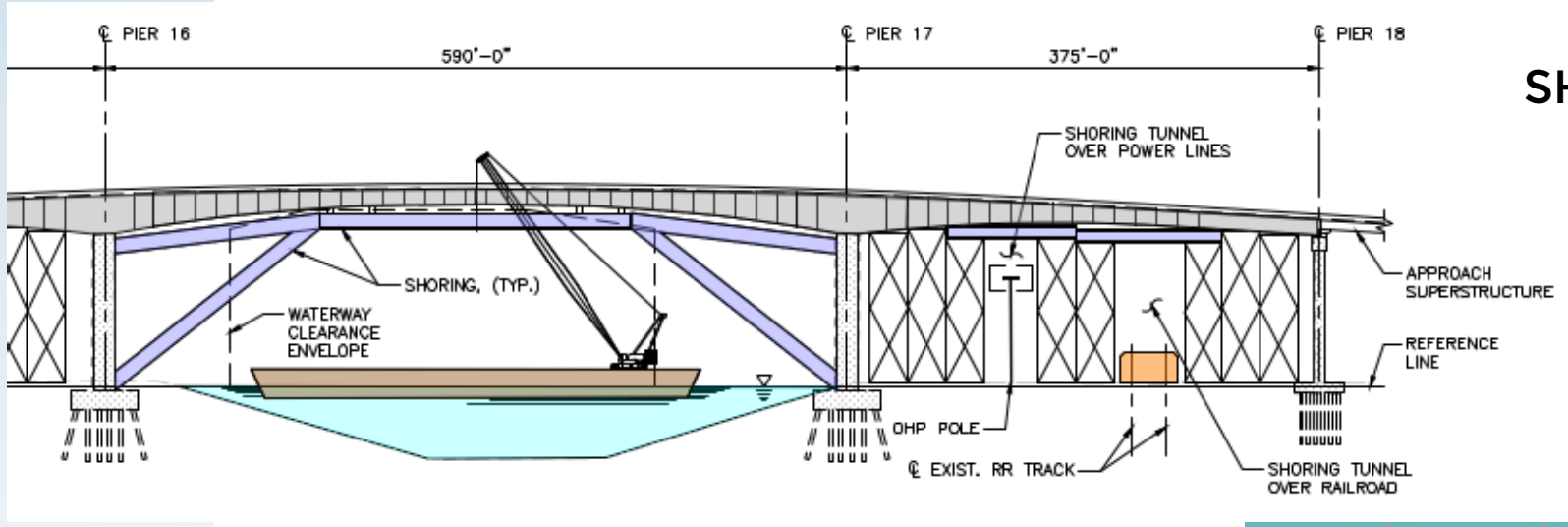


44’ deep truss

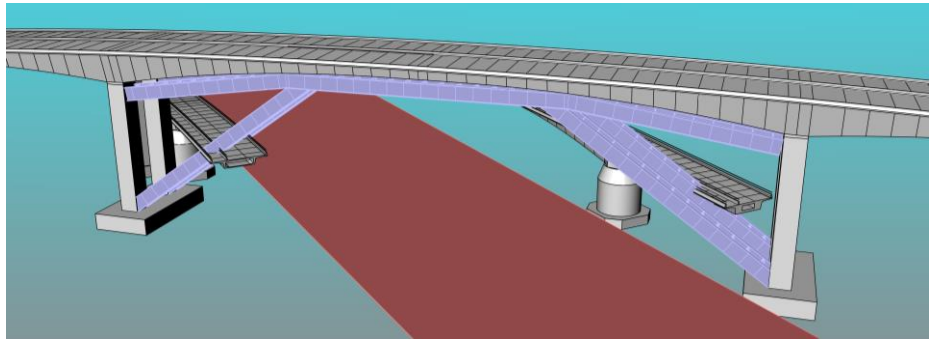
Alternative Definition – Shoring to Restore Live Load

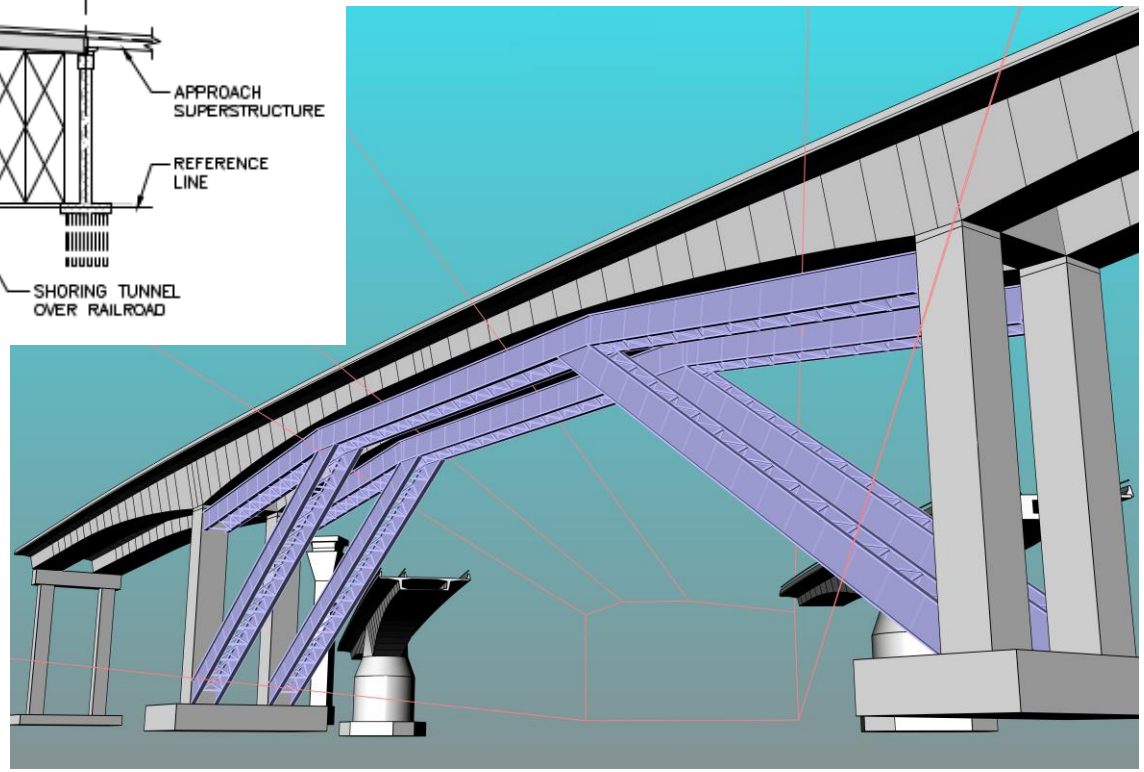
West Seattle High-Rise Bridge

Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
Shoring Concept (for restoring LL)	X				



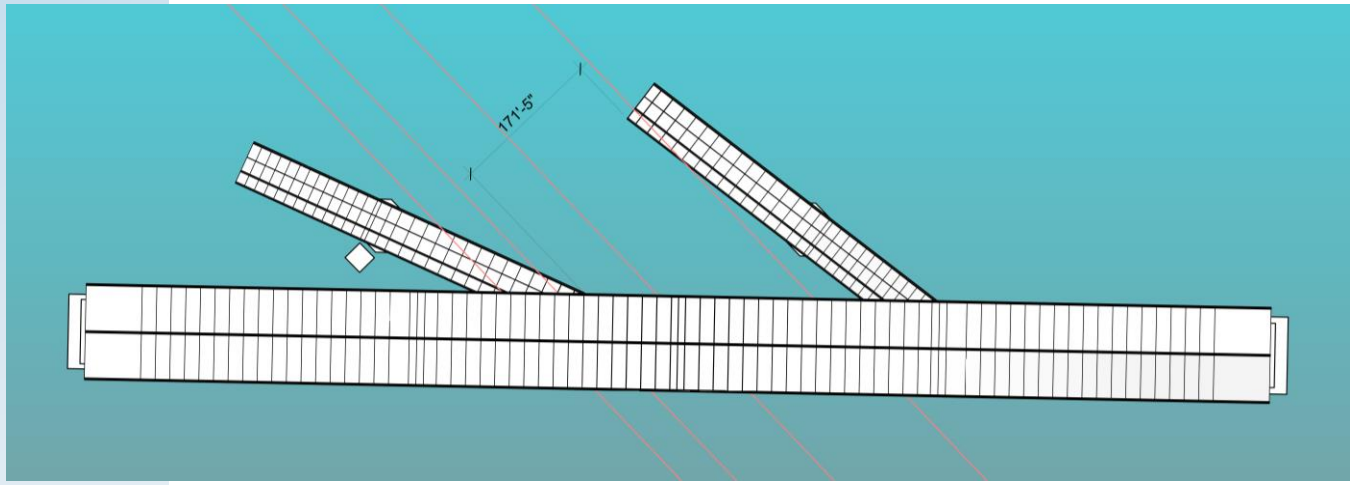
SHORING – CONCEPT 1: K-FRAME

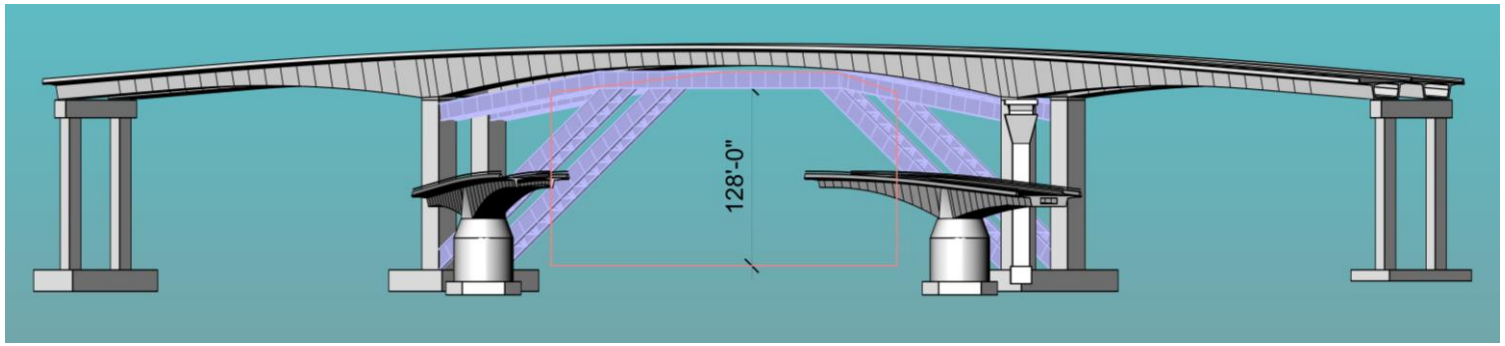




Alternative Definition – Shoring to Restore Live Load

West Seattle High-Rise Bridge



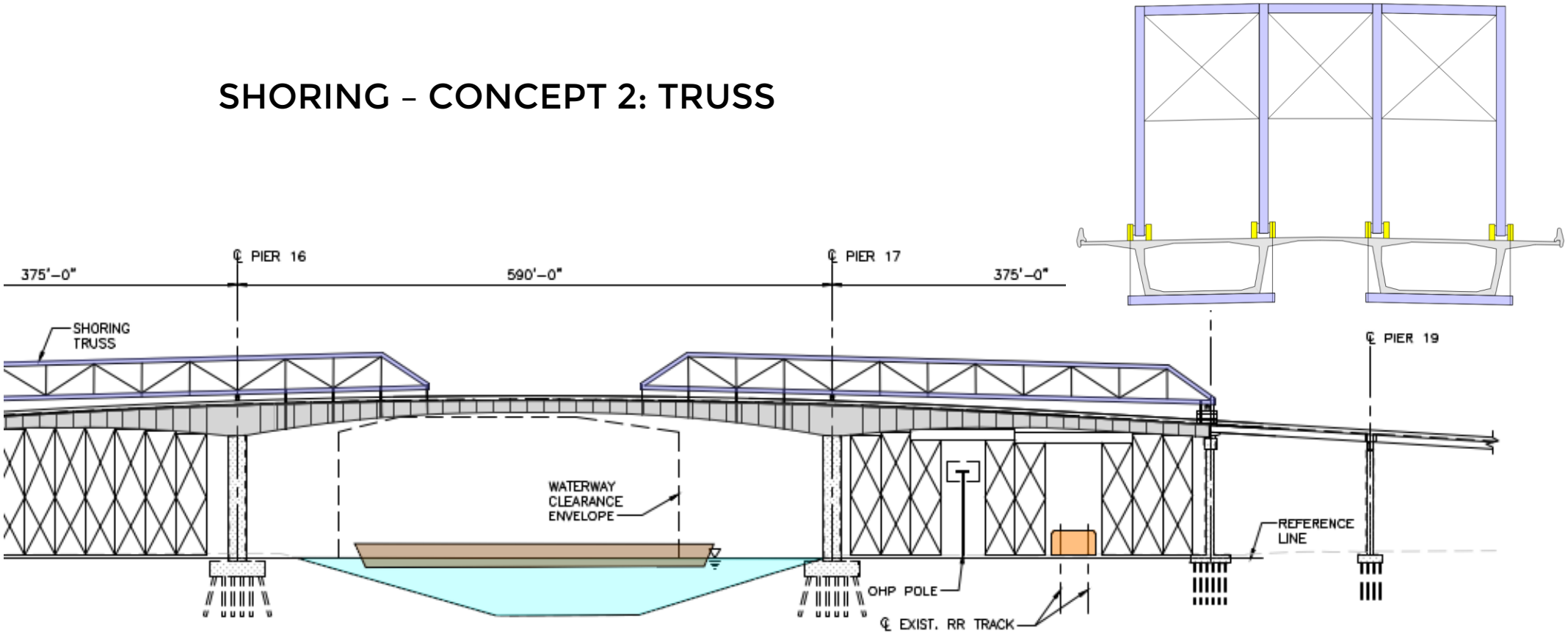


Alternative Definition – Shoring to Restore Live Load

N21

West Seattle High-Rise Bridge

Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
Shoring Concept (for restoring LL)	X				

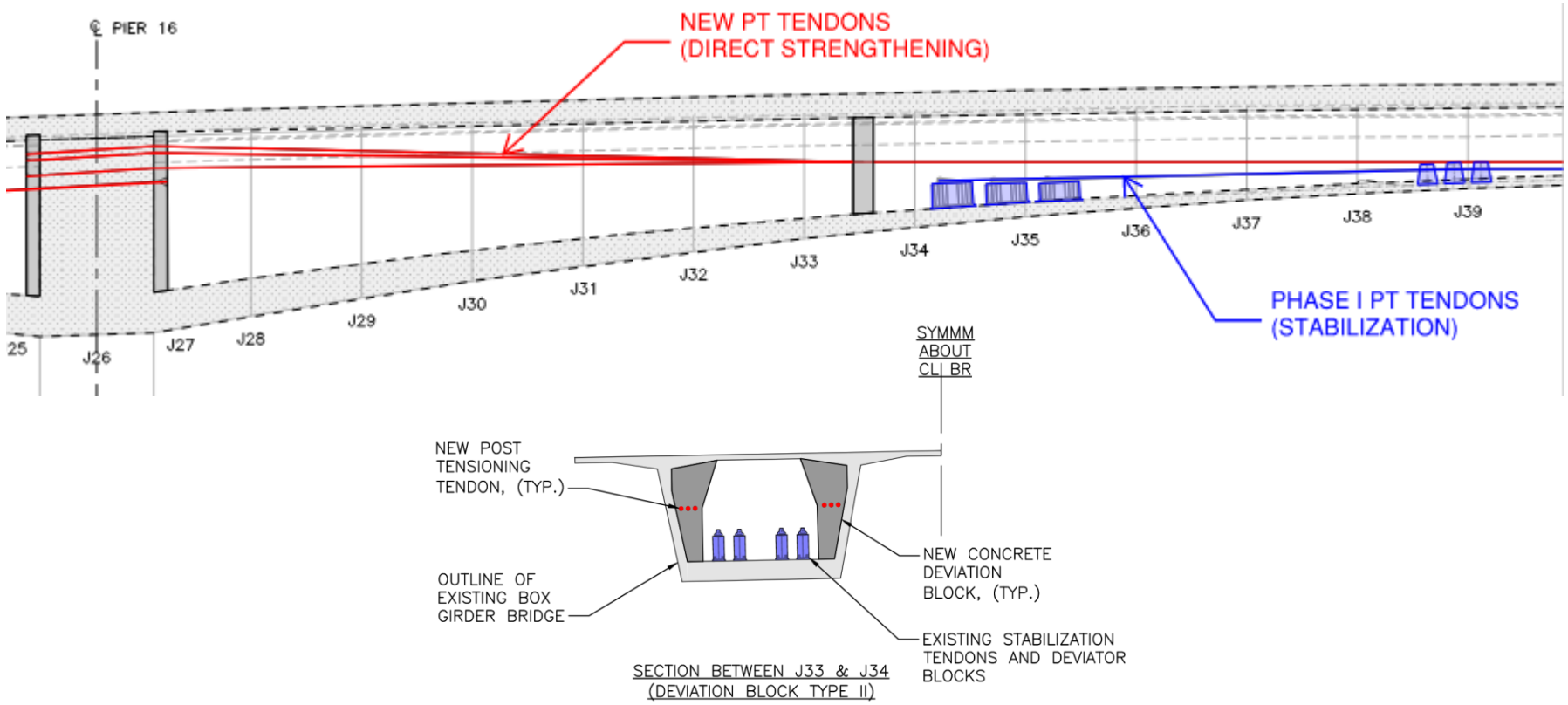


Alternative Definition – Direct Strengthening

N22

West Seattle High-Rise Bridge

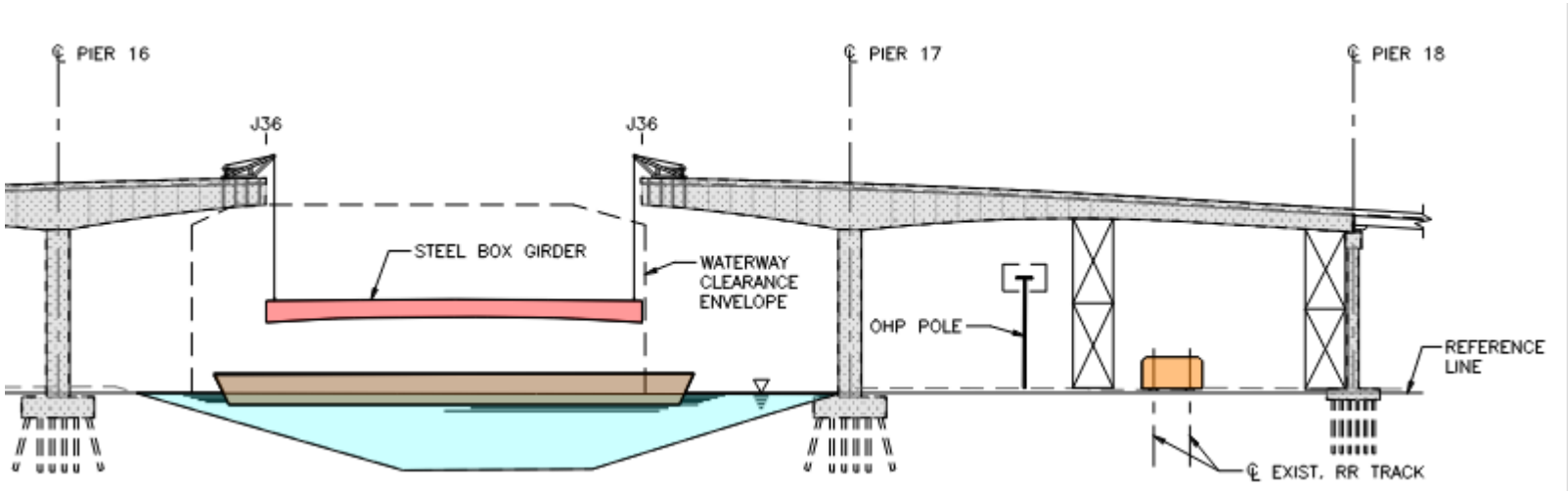
Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
Direct Strengthening		X			



Alternative Definition – Partial Superstr. Replacement

West Seattle High-Rise Bridge

Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
Partial Superstructure Replacement			X		

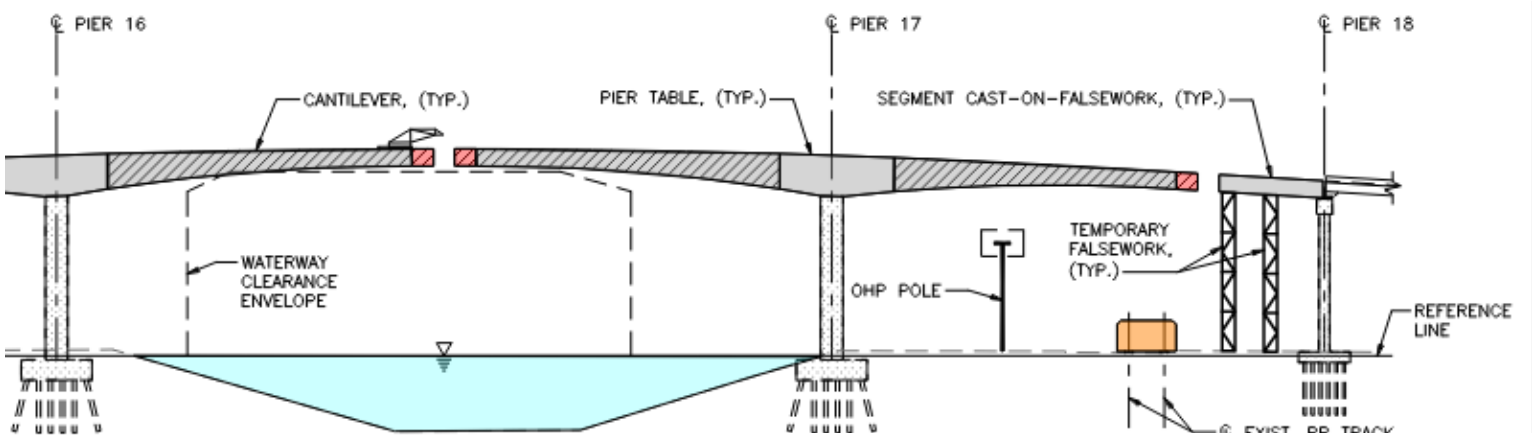


STEEL BOX GIRDER
CENTER SECTION

Alternative Definition – Full Superstr. Replacement N24

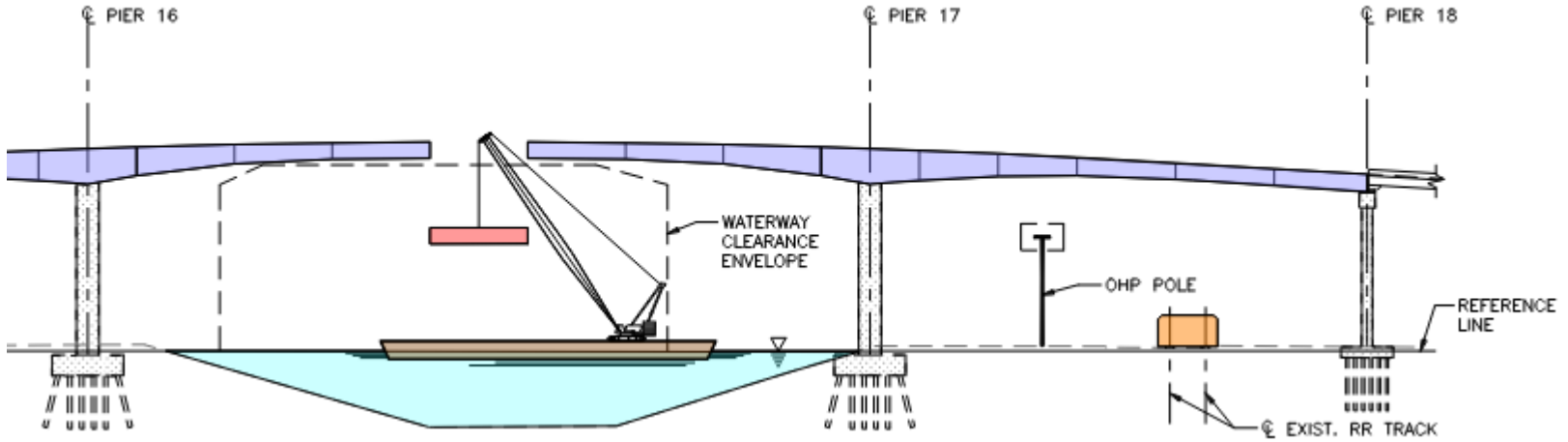
West Seattle High-Rise Bridge

Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
Full Superstructure Replacement				X	



CONCRETE BALANCED CANTILEVER OPTION

STEEL BOX GIRDER OPTION

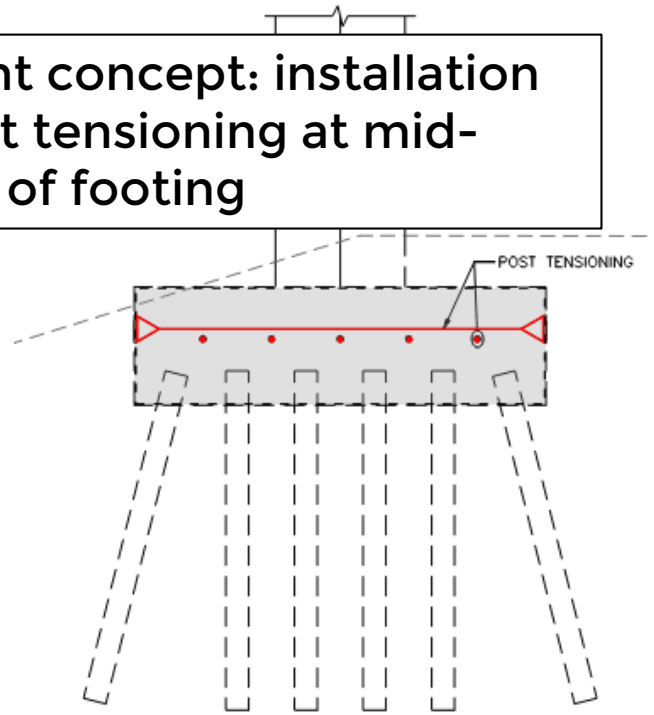


Alternative Definition – Foundation Retrofit

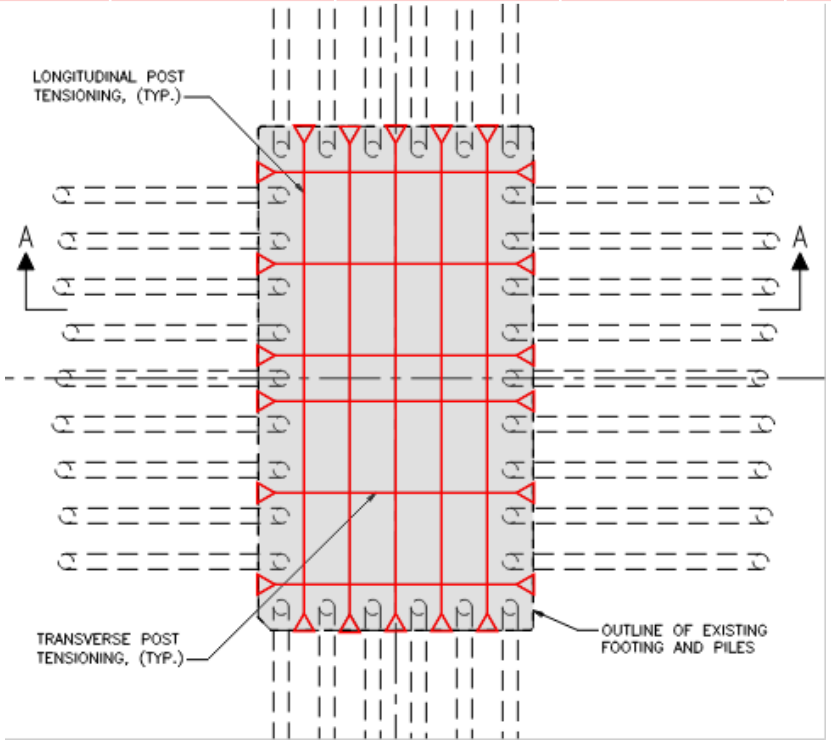
West Seattle High-Rise Bridge

Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
Foundation Retrofit		X	X	X	

Current concept: installation of post tensioning at mid-depth of footing



SECTION A–A
SCALE: 3/32" = 1'-0"



PLAN – RETROFIT EXISTING FOUNDATION
SCALE: 1/16" = 1'-0"

Alternative Definition – Base Isolation

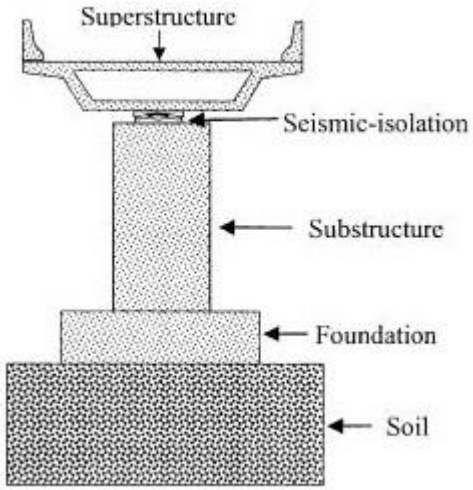
N26

West Seattle High-Rise Bridge

Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
Base Isolation				X	

- Isolation Bearings
  - Potential Isolation Benefits
    - Reduced Structural Demands
    - Improved Performance
    - Savings on Construction Costs
  - Vigorous testing requirements
  - Bearing assemblies are costly
  - Avoids Pier 16 and 17 modifications, however may require strengthening of expansion piers.

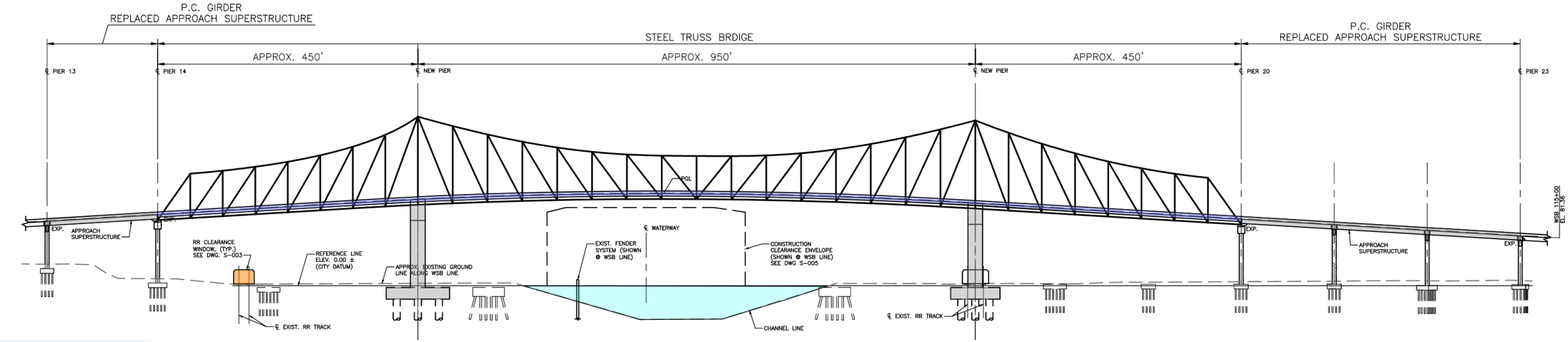
Isolation Bearings



Alternative Definition – In-Kind Bridge Replacement

West Seattle High-Rise Bridge

Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
In-Kind Replacement	X	X	X		X

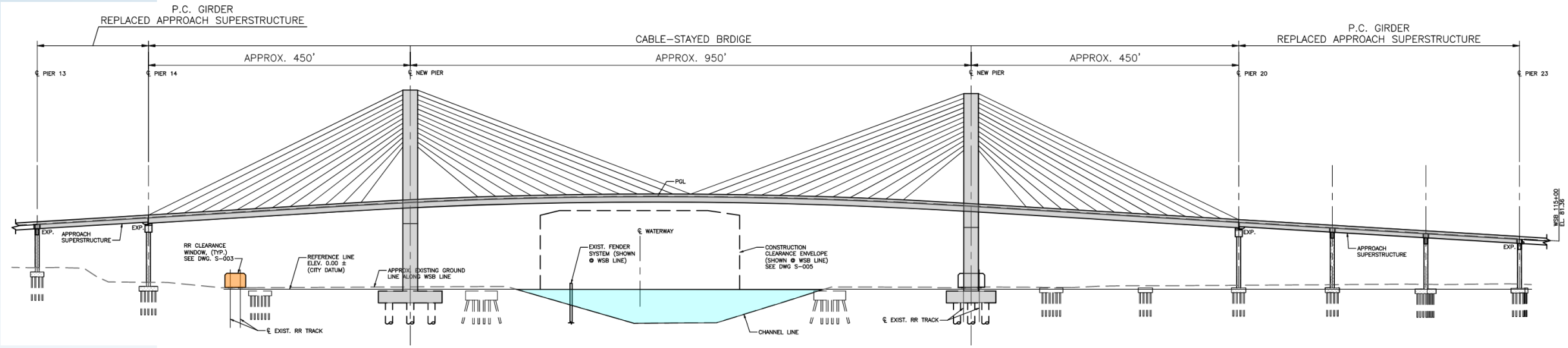


STEEL TRUSS

Alternative Definition – In-Kind Bridge Replacement

West Seattle High-Rise Bridge

Description of Work	Alt #1	Alt #2	Alt #3	Alt #4	Alt #5
In-Kind Replacement	X	X	X		X



CABLE-STAYED BRIDGE

In-Kind Bridge Replacement

West Seattle High-Rise Bridge

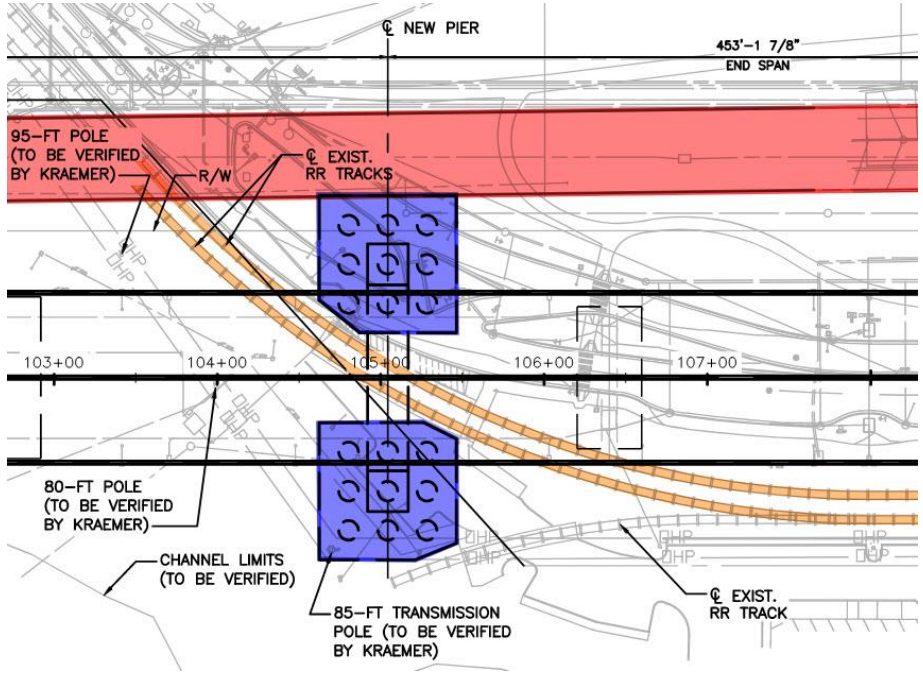
- Multiple replacement schemes are being considered
- Concepts include:
  - *Balanced Cantilever*
  - *Extradosed*
  - *Through Truss*
  - *Cable-Stayed*

- An initial screening process will be used to select the best option to be carried forward through the CBA

In-Kind Bridge Replacement

West Seattle High-Rise Bridge

- Initial screening considerations:
  - *Viable pier placement options and span lengths*
  - *Interference with adjacent structures (i.e. the swing bridge)*
  - *Clearances (Navigation, FAA, BNSF)*
  - *Demolition extents/tie in with approach spans*



Agenda

West Seattle High-Rise Bridge

- Workshop Objective
- Cost-Benefit-Analysis Framework
- Alternatives Definition
- **Evaluation Process**
- Attributes (Value Definition)
- Action Items

Cost Benefit Analysis – Evaluation

West Seattle High-Rise Bridge

- Potential list of evaluation criteria “attributes” includes…
  - Funding Opportunity
  - Forward Compatibility
  - Constructability
  - Environmental
  - Local Business Impacts
  - Mobility Impacts
  - Utility Impacts
  - Multi-Modal Impacts
  - Maintenance and Inspection

Actual List is TBD in conjunction with the City and desired committees, panels, stakeholders, etc.

Cost Benefit Analysis – Evaluation - Matrix

West Seattle High-Rise Bridge

- Attribute Weighting

Which attribute contributes more to the overall success of the project?

										TOTAL	%
Bridge Maintenance and Inspection	A	B	C	D	A	F	G	H	I	2.0	4.4%
Constructability	B	B	D	B	F	B	H	B		6.0	13.3%
Environmental	C	D	C	F	G	H	I			3.0	6.7%
Equity	D	E	D	G	D	I				6.0	13.3%
Mobility patibility	E	F	E	H	E					4.0	8.9%
Funding Opportunities	F	G	F	I						6.0	13.3%
Local Business Impacts	G	G	I							6.0	13.3%
Mobility Impacts	H	I								5.0	11.1%
Multi-modal Impacts	I									7.0	15.6%
										45.0	100%

Weight Factor

The number of times the letter "D" appears in the table

*NOTE: Attribute Ratings shown have been provided for illustration purposes only. Actual ratings to be deteremined with input from the City.

$$\textbf{Weight Factor} = \frac{\text{Count of Individual Attribute Letter Designation}}{\text{Count of Total Attribute Letter Designations}}$$

Cost Benefit Analysis – Evaluation - Matrix

West Seattle High-Rise Bridge

- Alternative Scoring

Attributes

Alternatives

Numerical Rating

VALUE MATRIX

West Seattle Bridge Rehabilitation/Replacement Study

	Attribute	Attribute Weight	Alternative	Performance Rating	Total Performance
A	**Funding Opportunities**	4.4	Temporary Shoring to Restore Live Load (3 to 5 years)	5	22.2
			Direct Strengthening to Restore Live Load (5 to 15 years)	5	22.2
			Partial Superstructure Replacement to Restore Live Load (15 to 50 years)	3	13.3
			Accelerated Superstructure Replacement (50 to 75 years)	3	13.3
			Accelerated Bridge Replacement (75+ years)	5	22.2
B	**Forward Compatibility**	13.3	Temporary Shoring to Restore Live Load (3 to 5 years)	3	40.0
			Direct Strengthening to Restore Live Load (5 to 15 years)	5	66.7
			Partial Superstructure Replacement to Restore Live Load (15 to 50 years)	7	93.3
			Accelerated Superstructure Replacement (50 to 75 years)	7	93.3
			Accelerated Bridge Replacement (75+ years)	9	120.0
			Temporary Shoring to Restore Live Load (3 to 5 years)	3	20.0

Baseline

Option / Baseline Rating Scale	
Rating	Performance Attribute Scale
9	Option is strongly preferred
7	Option is moderately preferred
5	Baseline
3	Baseline is moderately preferred
1	Baseline is strongly preferred

Performance

Weight Factor

- **Performance** = Numerical Rating x Weight Factor [per Attribute]
- **Total Performance** = sum of performance for each Attribute

Cost Benefit Analysis – Evaluation - Matrix

West Seattle High-Rise Bridge

- Summary of Costs

	Alternative	Construction**	Right of Way	Other Costs*	Program Capital Cost (less Life Cycle Cost)	Life Cycle Cost	Program Life Cycle Capital Cost
1	Temporary Shoring to Restore Live Load (3 to 5 years)	$ 1.00	$ 0.25	$ 0.25	**$ 1.50**	$ 1.00	**$ 2.50**
2	Direct Strengthening to Restore Live Load (5 to 15 years)	$ 1.00	$ 0.25	$ 0.25	**$ 1.50**	$ 1.00	**$ 2.50**
3	Partial Superstructure Replacement to Restore Live Load (15 to 50 years)	$ 1.00	$ 0.25	$ 0.25	**$ 1.50**	$ 1.00	**$ 2.50**
4	Accelerated Superstructure Replacement (50 to 75 years)	$ 1.00	$ 0.25	$ 0.25	**$ 1.50**	$ 1.00	**$ 2.50**
5	Accelerated Bridge Replacement (75+ years)	$ 1.00	$ 0.25	$ 0.25	**$ 1.50**	$ 1.00	**$ 2.50**
				Min	**$ 1.50**	Min	**$ 2.50**

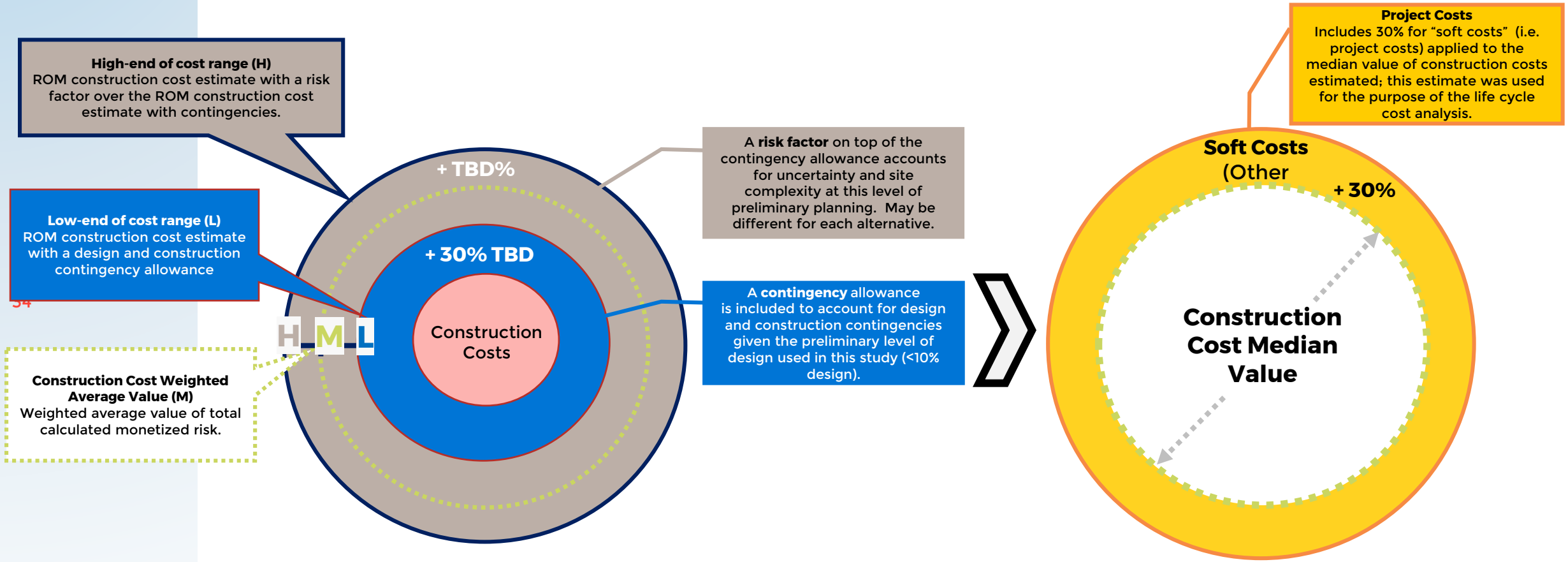
- Calculate for each alternative.
- Include two types:
  - Capital costs (ie. construction, ROW, and other)
  - Life cycle costs (ie. O&M and R&R)

Cost is not an attribute.

*Other Costs Include:
Engineering, CA&I, CSS, utility relocation costs, third party review costs, City costs, etc.

Cost Benefit Analysis – Evaluation - Matrix

West Seattle High-Rise Bridge



Step 1. Estimate the range of **construction costs.**

Step 2. Estimate **project costs** for the life cycle analysis.

Cost Benefit Analysis – Evaluation - Matrix

West Seattle High-Rise Bridge

Risk Identification				Qualitative Analysis			Risk Response		Monitoring & Controlling	
		(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)
Risk Number	Category	Date	Specific Risks or Opportunity	Probability Table IV (VH, H, M, L, VL)	Impact Table V (a&b) (VH, H, M, L, VL)	Overall Rating Table VI (H, M, L)	Strategy (Avoid, Transfer, Mitigate, Accept)	Planned Response	Assigned Responsible Person	Current Status, Date
1	Schedule		Securing funding to avoid delays in construction	M	H	H	Mitigate	Identify the funding sources available for the different alternatives.	CITY	
2	Schedule		If project stakeholders are not well informed as the project proceeds, then design decisions will be in jeopardy and project delays could occur.	VL	M	L	Avoid	Meet with the key stakeholders as part of the CBA process to keep the stakeholders informed and updated when we need key decisions or input from them to keep the project moving forward.	CITY	
3	Schedule		Delays in getting agency approval could lead to overall project delays	VL	H	M	Avoid	Meet and discuss the project with the agencies as part of the CBA process.	CITY	
35 4	Cost		Geotechnical standards for seismic acceleration will likely change in next couple years, which may change design and sizing. This is relation to Subduction Zones, Near Fault Effects, and Basin Effects.	VH				Replacement alternatives being considered	WSP	

Table IV
Risk/Opportunity Probability Ranking

Ranking	Probability of Risk Materializing
VH	*80 – 99%*
H	*60 – 79%*
M	*40 – 59%*
L	*20 – 39%*
VL	*1 – 19%*

TABLE VI
Overall Rating Matrix

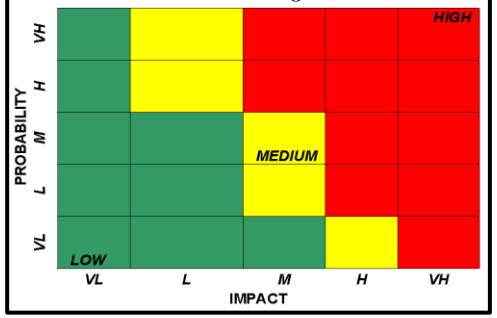


TABLE V(a)
Evaluating Impact of a Risk on Major Project Objectives

Objective	IMPACT	VL	L	M	H	VH
	Scope	Scope change Barely noticeable	Changes in project features with small effect on schedule, budget or customer satisfaction	Changes in project features with moderate effect on schedule, budget or customer satisfaction	Scope does not meet stakeholders needs	Scope does not meet project purpose and need
	Schedule	Insignificant schedule slippage	Low Schedule slippage	Delay of deliverable or milestone	Delay of critical path deliverable or milestone	project delivery outside fiscal year, funding cycle or construction season or does not meet political commitments
	Budget	Insignificant cost increase	<10% cost increase	10 – 20% cost increase	20 – 40% cost increase	>40% cost increase
	Customer Satisfaction	Minor customer dissatisfaction				Project does not meet customer's needs

Cost Benefit Analysis – Evaluation - Matrix

West Seattle High-Rise Bridge

- Life Cycle Assumptions

Description of Work	Alt #1		Alt #2			Alt #3			Alt #4		Alt #5	
Year	2022	2027	2022	2032	2062	2022	2032	2062	2025	2075	2025	2075
Demolition Concept		X			X	X		X	X		X	
Shoring Concept (for restoring LL)	X											
Direct Strengthening			X							X		X
Partial Superstr. Replacement						X						
Full Superstructure Replacement									X			
Foundation Strengthening				X			X		X			
“In-Kind” Bridge Replacement		X			X			X			X	

Cost Benefit Analysis – Evaluation - Matrix

West Seattle High-Rise Bridge

SDOT Input - Life Cycle Costs:

- What annual costs should be assumed for Operations and Maintenance? How does this vary for the different structure types/alternatives?

- Both superstructure and full bridge replacement will assume one superstructure rehabilitation will be needed during the life of the structure. What cost should be assumed for this rehabilitation?

Cost Benefit Analysis – Evaluation - Matrix

West Seattle High-Rise Bridge

- Summary of Findings

OVERALL PERFORMANCE		Performance (P)	Program Capital Cost (less long-term maint)	Capital Value Index (P/C)	Program Life Cycle Capital Cost	Life Cycle Value Index (P/C)
1	Temporary Shoring to Restore Live Load (3 to 5 years)	900.0	$ 1.50	600.00	$ 2.50	360.00
2	Direct Strengthening to Restore Live Load (5 to 15 years)	900.0	$ 1.50	600.00	$ 2.50	360.00
3	Partial Superstructure Replacement to Restore Live Load (15 to 50 years)	900.0	$ 1.50	600.00	$ 2.50	360.00
4	Accelerated Superstructure Replacement (50 to 75 years)	900.0	$ 1.50	600.00	$ 2.50	360.00
5	Accelerated Bridge Replacement (75+ years)	900.0	$ 1.50	600.00	$ 2.50	360.00
		900.0 Max		600.00 Max		360.00 Max

- **Performance-to-Cost Ratio** = Total Performance / Costs
  - Calculated for each alternative
  - Alternative with the highest value index is the preferred alternative.

Cost Benefit Analysis - Schedule

West Seattle High-Rise Bridge

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	May 2020				June 2020					July 2020				August 2020					September 2020			
	5/4/20	5/11/20	5/18/20	5/25/20	6/1/20	6/8/20	6/15/20	6/22/20	6/29/20	7/6/20	7/13/20	7/20/20	7/27/20	8/3/20	8/10/20	8/17/20	8/24/20	8/31/20	9/7/20	9/14/20	9/21/20	9/28/20
Readiness Planning Cost Benefit Analysis																						
Alternatives and Evaluation Criteria Definition Workshop											X											
Findings Workshop																X						
Recommendations Workshop																			X			
Preliminary Geotechnical Memorandum											D					DF						
Basis of Design											D					DF						
Environmental (Permitting Compliance Matrix & Strategy)													D			DF						
Evaulation Criteria Development											D		F									
Alternatives Development (Sketches)											D					DF						
Constructability																						
Evaluation Results																DF			F			
Risk/Opportunity Assessment (including funding)																DF			F			
Cost Estimating																DF			F			
Reporting																			D			F

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- **Workshops:** 3 remote workshops
  - Workshop #1 – get confirmation on alternatives and criteria being evaluated
  - Workshop #2 – discuss initial findings of the CBA
  - Workshop #3 – present recommendations

Agenda

West Seattle High-Rise Bridge

- Workshop Objective
- Cost-Benefit-Analysis Framework
- Alternatives Definition
- Evaluation Process
- **Attributes (Value Definition)**
- Action Items

Attributes Definition

West Seattle High-Rise Bridge

Identified Potential Attributes:

- Funding Opportunity
- Forward Compatibility
- Constructability
- Environmental
- Local Business Impacts
- Mobility Impacts
- Multi-Modal Impacts
- Maintenance and Inspection

Attributes Definition

West Seattle High-Rise Bridge

Eliminated Potential Attributes

- Utility Impacts
  - Must be accommodated regardless of alternative chosen
  - Can be evaluated in terms of cost

Attributes Definition

West Seattle High-Rise Bridge

Attribute	Measurable
Funding Opportunities	• Amount of funding available • Viability of funding sources
Forward Compatibility	• Flexibility for future roadway configurations • Ability to support light rail or other transit
Constructability	• Schedule impacts, • Complexity • Specialized construction equipment

Attributes Definition

West Seattle High-Rise Bridge

Attribute	Measurable
Environmental	• Air and Water Quality • Wildlife Impacts • Critical Areas • Section 4(f) Resources
Multi-modal Impacts	• Public transit • Pedestrian and bicycle access • Emergency Access
Maintenance and Inspection	• Inspection frequency and access • Painting requirements

Attributes Definition

West Seattle High-Rise Bridge

Attribute	Measurable
Local Business Impacts	• Traffic impacts • Closures • Temporary construction easements • Utility interruptions
Mobility Impacts	• Social economic • Travel time • Road user impacts • Safety

Agenda

West Seattle High-Rise Bridge

- Workshop Objective
- Cost-Benefit-Analysis Framework
- Alternatives Definition
- Evaluation Process
- Attributes (Value Definition)
- **Action Items**

Action Items (Workshop Questions)

West Seattle High-Rise Bridge

- Questions for Workshop Participants
  - Foundation Strengthening – now or in “x” years? If “x”, what should be assumed?
  - What should be assumed for the anticipated service life of the rehabilitation?
  - Is the shown evaluation criteria sufficient? Should anything be modified? Should additional criteria be added?
  - Is there anything missing in the alternative definitions?
  - What design contingency do we want to assume?
  - Input from SDOT on O&M costs to assume (annual).

Thank You!!!

West Seattle High-Rise Bridge

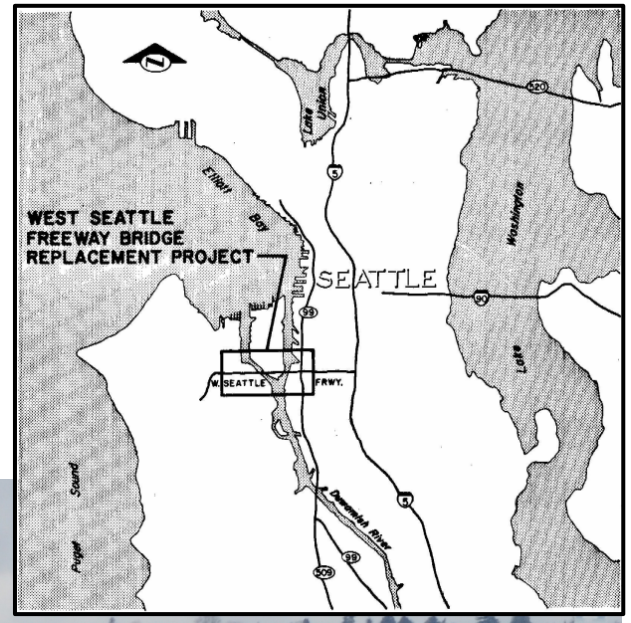
WORKSHOP PRESENTATIONS

WORKSHOP NO. 2

West Seattle High-Rise Bridge

N52

Cost Benefit Analysis – Workshop No. 02



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Client: City of Seattle | Location: Seattle, WA | Date: 20 August 2020

Pre-workshop Provided Materials

West Seattle High-Rise Bridge

- Alternative Figures
- Evaluation Matrix
- Capital Cost and BLCCA Spreadsheets
- Risk Spreadsheet

Agenda

West Seattle High-Rise Bridge

- **CBA & Workshop Objective**
- Attribute Weighting
- Attribute Definition & Measurables
- Alternative ROM Cost Ranges
- Next Steps

CBA & Workshop Objective

West Seattle High-Rise Bridge

- **Cost Benefit Analysis -** be prepared to make informed decisions related to further rehabilitation of the existing bridge or pivoting towards replacing the bridge with the lens of an accelerated replacement on alignment and matching existing profile.

- **Workshop No. 02** – solicit input from the CITY, TAP, and CTF on the performance evaluation and to-date cost information.
  - Outcome: Is there consistency between entities (i.e. CITY, TAP, CTF, WSP)? If not, initial performance findings will allow for focused follow-up discussions ahead of making a decision regarding further repair or replace.

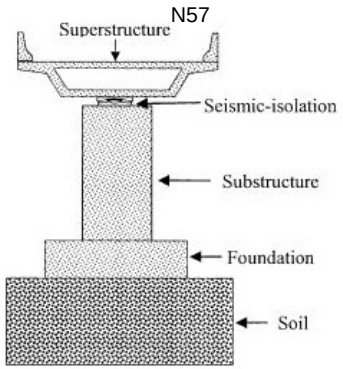
CBA & Workshop Objective

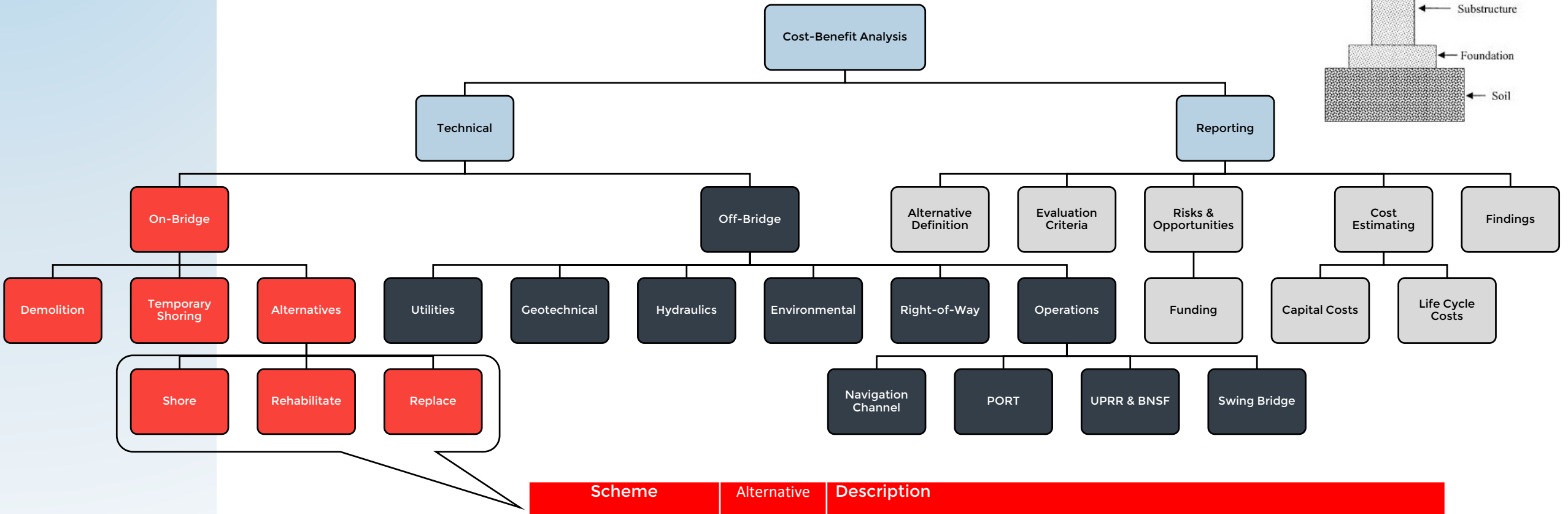
West Seattle High-Rise Bridge

- **The Ask…**
  - Fill-out Performance Rankings for each Attribute
    - 1,3,5,7,9 ranking relative to baseline alternative
  - Review of Cost Development
    - O&M – input on difference between alternatives
    - Risk Monetization
    - Right-of-Way – value based on identified properties

Cost Benefit Analysis - Framework

West Seattle High-Rise Bridge





Scheme	Alternative	Description
Shore	1	Temporary Shoring to Restore Live Load (3 to 5 years)
Rehabilitate	**2** **BASELINE**	Direct Rehabilitation to Restore Live Load (15 to 40 years)
Replace "In-Kind"	4	Accelerated Superstructure Replacement (50 to 75 years)
	5	Accelerated Bridge Replacement (75+ years)
Immersed Tube Tunnel	6	Off-Alignment Immersed Tube Tunnel (75+ years)

Note:
Arterial/corridor planning study assuming bridge is rehabilitated not considered as part of RP-CBA.

Agenda

West Seattle High-Rise Bridge

- CBA & Workshop Objective
- **Attribute Weighting**
- Attribute Definition & Measurables
- Alternative Performance “Benefit”
- Alternative ROM Cost Ranges
- Summary of Findings

Attribute Weighting

West Seattle High-Rise Bridge

Which evaluation criterion contributes more to the overall success of the project?

											TOTAL	%
Bridge Maintenance, Inspection & Operation	A	1/8	2/7	4/5	3/6	2/7	3/6	2/7	2/7	0/9	3.1	5.7%
Constructability	B	5/4	6/3	6/3	8/1	7/2	4/5	5/4	2/7		6.7	12.1%
Environmental	C	5/4	6/3	6/3	6/3	3/6	4/5	3/6			5.9	10.7%
Equity	D	5/4	6/3	6/3	5/4	5/4	2/7				5.6	10.1%
Forward Compatibility	E	5/4	5/4	3/6	6/3	2/7					5.1	9.3%
Funding Opportunities	F	6/3	4/5	3/6	2/7						4.7	8.5%
Local Business Impacts	G	2/7	4/5	1/8							4.1	7.5%
Mobility Impacts	H	7/2	2/7								6.4	11.7%
Multi-modal Impacts	I	1/8									5.1	9.3%
Seismic/Safety	J										8.3	15.2%
											55.0	100%

CTF

Which evaluation criterion contributes more to the overall success of the project?

											TOTAL	%
Bridge Maintenance, Inspection & Operation	A	10/10	11/9	5/15	11/9	8/12	14/6	5/15	8/12	6/14	4.9	8.9%
Constructability	B	9/11	7/13	11/9	9/11	13/7	7/13	7/13	6/14		5.0	9.0%
Environmental	C	5/15	10/10	10/10	9/11	9/11	10/10	3/17			4.8	8.7%
Equity	D	15/5	13/7	14/6	10/10	11/9	5/15				6.6	11.9%
Forward Compatibility	E	6/14	8/12	5/15	5/15	2/18					4.0	7.2%
Funding Opportunities	F	10/10	7/13	8/12	2/18						5.1	9.2%
Local Business Impacts	G	2/18	8/12	2/18							4.2	7.6%
Mobility Impacts	H	13/7	6/14								6.7	12.2%
Multi-modal Impacts	I	4/16									5.7	10.4%
Seismic/Safety	J										8.2	14.9%
											55.0	100%

SDOT

N59

Which evaluation criterion contributes more to the overall success of the project?

											TOTAL	%
Bridge Maintenance, Inspection & Operation	A	5/2	6/1	7/0	5/2	4/3	3/4	0/7	3/4	5/2	6.4	11.7%
Constructability	B	4/3	7/0	5/2	3/4	4/3	0/7	5/2	4/3		5.9	10.6%
Environmental	C	4/3	2/5	2/5	1/6	1/6	1/6	1/6			3.3	6.0%
Equity	D	1/6	2/5	1/6	0/7	2/5	0/7				2.3	4.2%
Forward Compatibility	E	4/3	2/5	0/7	4/3	3/4					5.0	9.1%
Funding Opportunities	F	3/4	0/7	4/3	5/2						5.6	10.1%
Local Business Impacts	G	1/6	6/1	2/5							6.3	11.4%
Mobility Impacts	H	7/0	6/1								9.6	17.4%
Multi-modal Impacts	I	2/5									4.7	8.6%
Seismic/Safety	J										6.0	10.9%
											55.0	100%

TAP

Combined

	TOTAL	%	Top 6
Bridge Maintenance, Inspection & Operation	14.4	8.8%	
Constructability	17.5	10.6%	3
Environmental	14.0	8.5%	
Equity	14.4	8.7%	
Forward Compatibility	14.1	8.5%	
Funding Opportunities	15.3	9.3%	5
Local Business Impacts	14.6	8.8%	6
Mobility Impacts	22.7	13.8%	1
Multi-modal Impacts	15.5	9.4%	4
Seismic/Safety	22.5	13.7%	2
	165.0	100%	

Attribute Weighting

West Seattle High-Rise Bridge

N60

	SDOT			TAP			CTF			Combined		
	TOTAL	%	Top 3	TOTAL	%	Top 3	TOTAL	%	Top 3	TOTAL	%	Top 6
Bridge Maintenance, Inspection & Operation	4.9	8.9%		6.4	11.7%	2	3.1	5.7%		14.4	8.8%	
Constructability	5.0	9.0%		5.9	10.6%		6.7	12.1%	2	17.5	10.6%	3
Environmental	4.8	8.7%		3.3	6.0%		5.9	10.7%		14.0	8.5%	
Equity	6.6	11.9%	3	2.3	4.2%		5.6	10.1%		14.4	8.7%	
Forward Compatibility	4.0	7.2%		5.0	9.1%		5.1	9.3%		14.1	8.5%	
Funding Opportunities	5.1	9.2%		5.6	10.1%		4.7	8.5%		15.3	9.3%	5
Local Business Impacts	4.2	7.6%		6.3	11.4%	3	4.1	7.5%		14.6	8.8%	6
Mobility Impacts	6.7	12.2%	2	9.6	17.4%	1	6.4	11.7%	3	22.7	13.8%	1
Multi-modal Impacts	5.7	10.4%		4.7	8.6%		5.1	9.3%		15.5	9.4%	4
Seismic/Safety	8.2	14.9%	1	6.0	10.9%		8.3	15.2%	1	22.5	13.7%	2
	55.0	100%		55.0	100%		55.0	100%		165.0	100%	

Attribute Weighting

West Seattle High-Rise Bridge

Direction: Kept Original 10 Attributes for Initial Evaluation N61

- **Attribute Condensing Considerations –**
  - **Environmental** - addressed in the assumptions made (for example, all alternatives will comply with environmental regulations; no new foundations can be constructed within the wetted perimeter of the ordinary high water; etc). The effects will show up in terms of construction schedule (working around work windows) and the overall type and layout of the structure assumed.
  - **Forward Compatibility** - focused on incorporation of ST on a joint structure. This is very similar to the multi-modal attribute, and thus, will be considered a measurable in the multi-modal attribute.
  - **Funding Opportunities** - acknowledge that the funding opportunities are acknowledged in the cost analysis and defined as opportunities to offset costs associated with monetized risk.
  - **Bridge Maintenance, Inspection and Operations** - acknowledge that these are really a cost and show up in the bridge life cycle cost analysis.

Agenda

West Seattle High-Rise Bridge

- CBA & Workshop Objective
- Attribute Weighting
- **Attribute Definition & Measurables**
- Alternative ROM Cost Ranges
- Next Steps

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Maintenance, Inspection & Operation

A measure of the services and systems necessary for the operation and preservation of the bridge and corridor.

Measurables	Unit of Measure	Alt #1 Shoring		Alt #2 Rehabilitation		Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
		5 Years (Shoring)	75+ Years (Alt 5)	40 Years (Rehab)	75+ Years (Alt 5)			
Inspection Requirements	Frequency & Level-of-Effort (High/Med/Low) Special Inspections (Yes/No)	6-12 Mo High Yes	12-24 Mo High Yes	6-12 Mo High Yes	12-24 Mo High Yes	24 Mo Med No	12-24 Mo High Yes	24 Mo Med Effort Yes
Intelligent Transportation Systems Required	Yes/No	Yes	No	No	No	No	No	Yes
Structural Health Monitoring Systems Required	Yes/No How many systems required?	Yes 3	No	Yes 3	No	No	No	No
Painting/UV protection Required	Yes or No	No	Yes	Yes	Yes	No	Yes	No
SCORE	**(1,3,5,7,9)**			**5**				

Other Considerations: Alternative 5 (

nt including dampers, cables and anchorages

First Ask: Provide 1, 3, 5, 7, 9 Score
Alternative 2 = Baseline (gets a score of 5)
Other Alternatives = 1 if baseline is strongly preferred
= 3 if baseline is preferred
= 5 if is equal to the baseline
= 7 if alternative is preferred over the baseline
= 9 if alternative is strongly preferred over the baseline

Attributes Definition & Measurables

N64

West Seattle High-Rise Bridge

Constructability

A measure of the ease and efficiency an alternative can be built relative to the schedule and potential means/methods permitted by the contract.

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Schedule impacts	Duration of project	39 Mo	12 Mo	42 Mo	45 Mo	
Complexity	Standard construction or complex	Standard	Complex	Standard	Complex	
Specialty Contractors and Equipment	Are any required and if so how many	Yes, 2	Yes, 5	Yes, 5	Yes, 5	
Utility Relocations	Impacts on existing utilities, scope and magnitude of relocations	Average	Minor	Major	Major	
In Water Work	Amount of in water work and how many in water windows are needed	None	Foundation Retrofit - 2 Windows	Foundation Retrofit - 2 Windows	None -Ex. Piers Remain	
Demolition	Amount and complexity of demolition required	Complex & Truss Demo	Complex	Complex	Complex	
Poor soil conditions	Is substantial foundation work required?	No	Yes	Yes	Yes	
Staging/Laydown area required	Required footprint	Average	Minimal	Average	Large	
SCORE	**(1,3,5,7,9)**		**5**			

Other Considerations Determined Not to Influence Decision: List of other considerations; etc.

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Environmental

A measure of potential adverse effects to the built or natural environments

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Noise and Vibration	Pile driving be required?	No	Yes	Yes	No	
Duwamish Waterway	In water work required (timing and duration) Proximity of construction ground disturbance to shoreline (1) Limited to temporary barges during demolition and construction (2) New foundations and piers for replacement structure are located away from waterway. (3) Foundation retrofit/strengthening requires ground disturbance within the shoreline and near the waterway	(1) and (2)	(1), (2), and (3)	(1), (2), and (3)	(1) and (2)	
Section 4(f) Resources	Use of parks, trails, open space, wildlife refuges Proximity of construction to known archaeological sites and known historic structures listed or eligible for the National Register of Historic Places (1) No use of parks, trails, open space or wildlife refuges (2) Potential use of Nucor Steel NHRP Building or Pacific Forge historic district (see list)	(1) and (2) List: Shoring trusses, and In kind bridge replacement piers	(1) and (2) List: Foundation retrofit, and In kind bridge replacement piers	(1) and (2) List: In kind bridge replacement piers	(1) and (2) List: In kind bridge replacement piers	
Emissions	Tons of GHG (from VMT) x construction duration	210,000 more tons	128,000 more tons	161,000 more tons	161,000 more tons	
Wildlife Impacts	Timing and duration of construction proposed during falcon/Great Blue Heron breeding season	Two phases of construction	Three phases of construction	Two phases of construction	Two phases of construction,	
SCORE	(1,3,5,7,9)		5			

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Equity

The incremental impacts resulting from bridge closure and construction that disproportionately affect historically and currently marginalized communities.

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Duration of bridge closure	Time to restore full capacity of bridge operations[1] (years full closure starting 1/2021)	3 lanes: 2024 New Bridge: 2033 (7.5 yrs)	All lanes: 2022 New bridge: 2066 (5.25 yrs)	New bridge: 2026 (5.25 yrs)	New bridge: 2026 (5.25 yrs)	
Incremental Vehicle-Miles Travelled	Incremental VMT during closure through marginalized communities[2]	Higher	Low	Medium	Medium	
Incremental Vehicle Travel	Incremental vehicular travel through marginalized communities (crossing the Duwamish Waterway)[3]	85.3 million more trips	58.8 million more trips	62.5 million more trips	62.5 million more trips	
Incremental Travel Time	Cumulative PHT during construction through marginalized communities[4]	Higher	Medium	Lower	Lower	
Number of Construction Events / Community Disruption	Repeat closure events due to preservation or construction marginalized communities would experience in a 75 year timeframe[5]	2 events within 3 years totaling 7.5 years	2 events within 40 years totaling 5.25 years	1 event totaling 5.25 years	1 event totaling 5.25 years	
SCORE	**(1,3,5,7,9)**		**5**			

1 - Time to restore full capacity of bridge operations; captures number of occasions a community would be subjected to more than one bridge closure episode lasting >1 year; begins on March 23, 2020.
2 - This is intended to capture associated incremental emissions (air quality impacts and GHG), risk of incremental severe crashes (mortality, severe injuries), noise and other negative indirect externalities associated with vehicular travel through marginalized communities.
3 - Vehicle trips (all types) crossing the Duwamish Waterway through diversion routes (through marginalized communities) x Construction Duration
4 - Cumulative PHT during construction and PHT in final configuration (tunnel is only difference) through marginalized communities through SDOT diversion routes. Measurable is intended to also capture qualitatively incremental vehicular delays during bridge closure through marginalized communities, and impacts on travel reliability for transit (speed and reliability).
5 - Measurable does not capture OTHER construction projects in the area happening in tandem; assumes haul routes will not go through marginalized communities.

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Forward Compatibility

Ability to accommodate future roadway configurations including light rail

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Future roadway configurations	Maintain a minimum of existing configuration and improve design features (Year)	3 lanes 2024-2029 7 lanes starting 2033	7 lanes 2022 (less 2063-2066)	7 lanes 2026 (some lane closures 1 year)	7 lanes 2026 (some lane closures 1 year)	
Accommodate light rail	Yes or No: Ability to accommodate light rail on bridge by 2032 (new bridge)	NO	NO	YES	YES	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Funding Opportunities

Alternative eligibility for various funding sources and evaluation of funding and financing opportunities.

Measurables	Unit of Measure (Weighted Ranking 1-9; Except Eligibility)	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Project eligibility	Yes or No: Count of eligible of funding sources (out of 42)	29	29	29	29	
Funding revenue generation potential	Historical experience on revenue generation ranges	5.7	5.4	5.7	5.7	
Stability of the funding source	Funding sources are scored based on known continuation, likely continuation, and likely elimination	6.2	5.9	6.2	6.2	
Likelihood of funding	Historical experience on competitiveness of funding sources and successful probability ranges	2.0	1.9	2.0	2.0	
Timing for availability	Availability of funding using ranges based on typical years to secure funds	7.0	6.5	7.0	7.0	
Administrative requirements	Existing administrative structure in place, incremental requirements, no structure in place	7.7	7.3	7.7	7.7	
Legal authority	Existing legal authority, Legal at the State level but City of Seattle is not authorized, no State or City authorization	8.8	8.3	8.8	8.8	
Income and racial equity	Measure is progressive, progressive or regressive depending on implementation, or regressive	7.6	7.1	7.6	7.6	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Local Business Impacts

Impacts to adjacent and local business during construction and in the final configuration

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Bridge Closures Impacts	Trip reduction x discretionary spending per trip (could impact BIPOC-owned businesses)	Net reduction of $3,5 million	Net increase of $6.7 million	Net reduction of $0.9 million	Net reduction of $0.9 million	
Direct/Indirect Economic Construction from construction	Capital expenditures	7,451 FTEs	7,976 FTEs	6,766 FTEs	9,548 FTEs	
Temp Const. Easements	Number/duration of TCE's (could impact BIPOC-owned businesses)	1 TCE for 1 yr + Alt 4/5 TCE's for 2.75 yrs	1 TCE for 1 yr + Alt 4/5 TCE's for 3.75 yrs	1 large TCE for 3.75 yrs	2 TCE's for 3.75 yrs	
Utility interruption	# of times impact utilities on and under bridge	For Demo and new bridge construct (Alt 4/5)	For Demo and new bridge construct (Alt 4/5)	For Demo and new bridge between piers 15-18 (east line)	For Demo and new bridge between piers 13-18 (east & west lines)	
Access impacts to local properties	Impacts to existing property access during construction and final configuration	No restrictions	Min. restriction during construction to S. Harbor Island via SW Manning	Potential restrictions to S. Harbor Island via SW Manning	Potential restrictions to S. Harbor Island via SW Manning	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Mobility Impacts

The effects of construction on regional mobility including travel times, routes and mode shifts as well as increased mobility of final facility.

Measurables	Unit of Measure (total over forecast period)	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Travel Time	Cumulative PHT during construction and PHT in final configuration (tunnel is only difference)	42.9 million more hours	40.0 million more hours	30.1 million more hours	30.1 million more hours	
Travel Distance	VMT x duration during construction and VMT in final configuration (tunnel is only difference)	563.9 million more miles	353.6 million more miles	398.8 million more miles	398.8 million more miles	
Non-Vehicle Trips	Trips (mode shift to bikes from car) during construction. This is a health benefit.	60.9 million more bike/ped trips	45.7 million more bike/ped trips	44.1 million more bike/ped trips	44.1 million more bike/ped trips	
Regional Mobility Impacts	Impacts to other regional construction projects. Does other construction occur when bridge is closed and there is more traffic on surface streets?	E Marginal: bridge closed; WSBLE: ½ during bridge closure	E Marginal: ½ during bridge closure; WSBLE: bridge open	E Marginal: bridge closed; WSBLE: bridge open	E Marginal: bridge closed; WSBLE: bridge open	
Safety	Crashes/VMT on detour routes compared to W Seattle Bridge (during construction), and typical tunnel versus bridge (final configuration)	553 more crashes	347 more crashes	391 more crashes	391 more crashes	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

N71

West Seattle High-Rise Bridge

Multi-modal Impacts

Impacts to emergency vehicle and transit during construction as well as pedestrians and bicycles in the final configuration

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Incremental Transit Service during Construction	Increase transit capacity	1,280 daily units	753 daily units	982 daily units	982 daily units	
Bicycle Traffic Accommodation	High/med/low (no bike accommodation on bridge)	Low	Low	Low	Low	
Pedestrian Traffic Accommodation	High/med/low (no ped accommodation on bridge)	Low	Low	Low	Low	
Emergency Access	Total Response time increase during construction	38 hours	27 hours	27 hours	27 hours	
Freight Mobility	High/med/low	422,000 hours	360,000 hours	295,000 hours	295,000 hours	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

N72

West Seattle High-Rise Bridge

Seismic/Safety

Expected post-earthquake service and damage levels relative to the design seismic event including consideration of implementation.

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Seismic Hazard Levels	100-Year Return Period 210-Year Return Period 975-Year Return Period	 X X	X X	 X X	 X X	 X X
Operational Classification *Post design earthquake service* *(No, Limited, or Full Service)*	100-Year Return Period 210-Year Return Period 975-Year Return Period	 Full Limited	Full No	 Full Limited	 Full Limited	 Full Limited
In-Ground Hinging Permitted	Yes or No	No	Yes	No	No	No
Vertical Excitations Considered	Yes or No	Yes	No	Yes	No	Yes
Seismic Compliance Established	Year	2033	2032	2025	2026	TBD
SCORE	**(1,3,5,7,9)**		**5**			

Other Considerations: Owner specified project specific criteria needed to be developed for replacement structures considered as “essential” or “critical”.

Agenda

West Seattle High-Rise Bridge

- CBA & Workshop Objective
- Attribute Weighting
- Attribute Definition & Measurables
- **Alternative ROM Cost Ranges**
- Summary of Findings

Alternative ROM Cost Ranges – General Assumptions

West Seattle High-Rise Bridge

- ROM cost ranges are in part metrics based, and in-part based on actual engineering and calculated quantities.
- Limited estimate based on assumptions of the CBA
  - On-Alignment and matching existing profile
  - High-Rise Only (no consideration of approach costs)
- Capital costs are reported in 2021 dollars
- Capital costs are inclusive of:
  - 30% design contingency
  - 10% construction contingency
- Social economic impacts are not quantitatively reported in the reported cost ranges. Social economic impacts have been qualitatively defined in the performance (i.e. benefits).

Alternative ROM Cost Ranges – General Assumptions

West Seattle High-Rise Bridge

- Types of Costs
  - Capital Costs
  - Life Cycle Costs

Alternative ROM Cost Ranges – General Approach

N76

West Seattle High-Rise Bridge

- Types of Costs
  - **Capital Costs**
  - Life Cycle Costs

- Construction Costs
  - Developed Material Quantities to varying levels of degree and supported with metrics from recent and relevant experience
  - Design and Construction Contingencies
- Right-of-Way
  - Developed map of impacted properties
- Risk Monetization
  - Have risk registry and identified risks that could be monetized and method for monetizing the risk.
  - Assigned a probability based on likelihood the risk could materialize.
- Design and Planning Costs
  - Termed as “Soft Costs” or “Other Costs” in the estimates
  - Taken as 30% of construction costs
  - Considers engineering, CA&I, CSS, third party review costs, Owner costs, etc.

Second Ask: Cost input on impacted properties

Third Ask: Are there risks missing? Do we appropriately capture/monetize the risks within the context of the CBA?

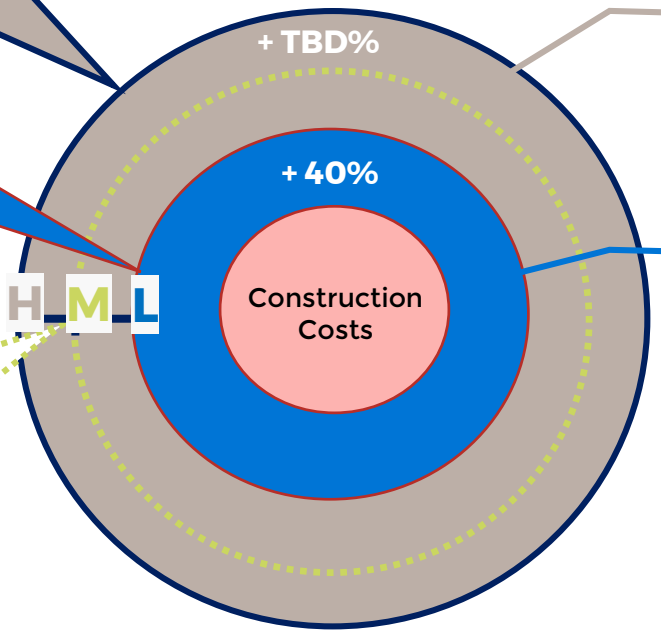
Alternative ROM Cost Ranges – General Approach

West Seattle High-Rise Bridge

High-end of cost range (H)
ROM construction cost estimate with a risk factor over the ROM construction cost estimate with contingencies.

Low-end of cost range (L)
ROM construction cost estimate with a design and construction contingency allowance

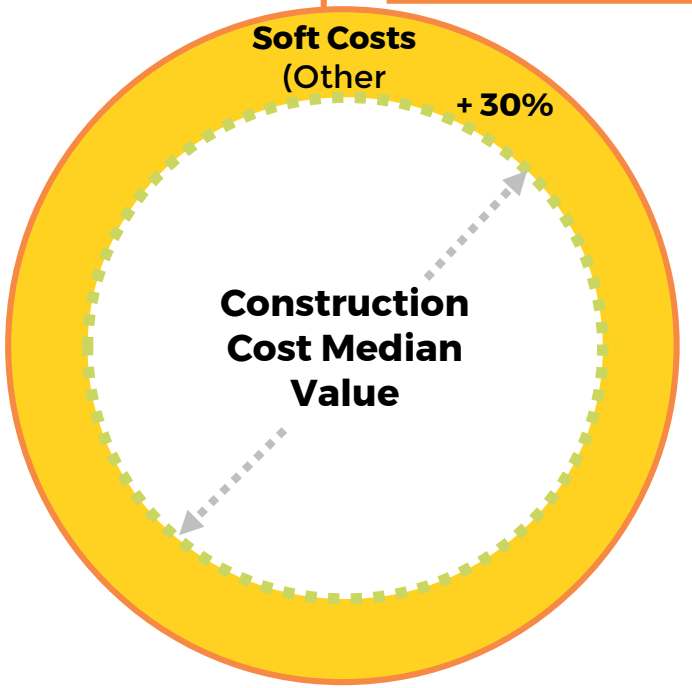
Construction Cost Weighted Average Value (M)
Weighted average value of total calculated monetized risk.



A **risk factor** on top of the contingency allowance accounts for uncertainty and site complexity at this level of preliminary planning. May be different for each alternative.

A **contingency** allowance is included to account for design and construction contingencies given the preliminary level of design used in this study (<10% design).

Project Costs
Includes 30% for "soft costs" (i.e. project costs) applied to the median value of construction costs estimated; this estimate was used for the purpose of the life cycle cost analysis.



Step 1. Estimate the range of **construction costs.**

Step 2. Estimate **project costs** for the life cycle analysis.

Alternative ROM Cost Ranges – General Approach

N78

West Seattle High-Rise Bridge

- Types of Costs
  - Capital Costs
  - **Life Cycle Costs**

- Construction Durations
  - Developed preliminary schedule
  - Considers when in time work would occur
- Anticipated Asset Life
  - Equivalent durations formulated around AASHTO and FHWA 75 year design life
  - Start in year 2021 when shoring starts (Alternative No. 01)
  - Ends in year 2101 when 75 year design life of replacement structure (Alternative No. 05) is reached
- Rates
  - Inflation = 3% | Discount = 2%
- Operating and Maintenance (O&M) Costs
  - Based on input received from the CITY
  - Scaled based on extrapolation of understanding of provided O&M costs for each alternative
- Repair and Replacement (R&R) Costs
  - Repair/rehabilitation 50-years into their life. Repair assumed to be $50M.

Fourth Ask: Concurrence on O&M values based on the context of the CBA?

Alternative ROM Cost Ranges – Capital Cost Parameters

N79

West Seattle High-Rise Bridge

Parameter	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Bridge Importance Category	Essential	Normal	Essential	Essential	
Seismic Hazard 100-year 210-year 975-year	 X X	 X X	 X X	 X X	
Permissible In-Ground Hinging	No	Yes	No	No	
Vertical Excitations	Yes	No	Yes	Yes	
Structure Type	Cable-Stayed	N/A	CIP Conc Box Girder	Cable-Stayed	
Inclusive of Sound Transit Yes/No If so, what year?	 Yes 2033	 No – See Note --	 Yes 2026	 Yes 2026	
No. of Lanes Each Direction	4	4	4	4	
Approach Modifications	No	No	No	Yes	
Life Cycle	2021 - 2101				

NOTE: Could potentially accommodate if reconfigured channelization (i.e. use of reversible lanes.

Alternative ROM Cost Ranges – Life Cycle Parameters

N80

West Seattle High-Rise Bridge

Parameter	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Life Cycle	2021 - 2101				
Shoring to Restore Live Load	**2021-2024**	--	--	--	TBD
Demolition	2029	2063	**2022-2023**	**2022-2023**	
Rehabilitation	--	**2021-2022**	2075	2076	
Foundation Retrofit	--	2032	**2024**	--	
Superstructure Replacement	--	--	**2024-2025**	--	
“In-Kind” Bridge Replacement	2030-2033	2063-2066	--	**2024-2026**	
Tunnel					

BOLD = Considered in Capital Cost Calculation

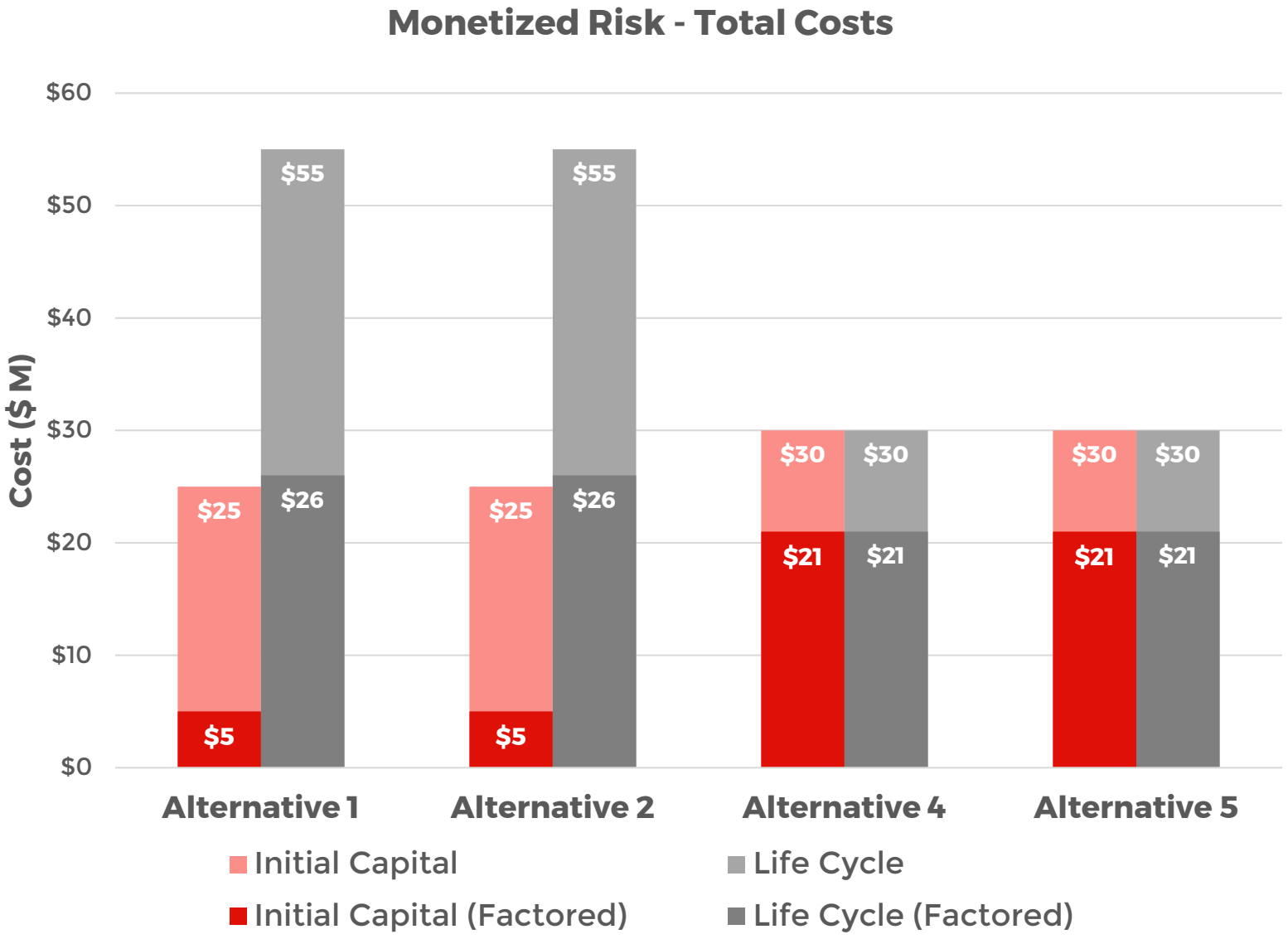
Alternative ROM Cost Ranges – Monetized Risk Items

West Seattle High-Rise Bridge

Key Risk Items*:

- Change in geotechnical standards for seismic acceleration
- Bridge Stabilization measure performance
- Bridge Importance classification
- FAA height restrictions
- USCG navigation clearance

*Selected risk items have been presented here. Additional risk items which may have monetary impact are included in the full risk register



Alternative ROM Cost Ranges – Project Costs

West Seattle High-Rise Bridge

N82

Project Costs	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Construction Costs	$53.9M	$30.1M	$208.8M	$308.7M	TBD
Contingency (40%)	$21.6M	$12.1M	$83.5M	$123.5M	
Subtotal	$75.5M	$42.2M	$292.3M	$432.2M	
Right-of-Way (TCE's)	$xx.xM	$xx.xM	$xx.xM	$xx.xM	
Subtotal	$75.5M	$42.2M	$292.3M	$432.2M	
Monetized Risk	$5.0M	$5.0M	$21.0M	$21.0M	
Subtotal	$80.5M	$47.2M	$313.3M	$453.2M	
Other Costs (30%)	$24.2M	$14.2M	$94.0M	$136.0M	
Total Project Costs	**$104.7M**	**$61.4M**	**$407.3M**	**$589.2M**	

Alternative ROM Cost Ranges – Life Cycle Costs

N83

West Seattle High-Rise Bridge

Costs	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Project Costs	$104.6M	$61.3M	$407.3M	$589.2M	
Operating & Maintenance	$32.8M	$37.5M	$22.4M	$30.1M	
Repair & Rehabilitation	$479.0M	$561.1M	$130.1M	$164.7M	
Total Life Cycle Costs	**$616.4M**	**$659.9M**	**$559.8M**	**$783.9M**	

Alternative ROM Cost Ranges – O&M and R&R

West Seattle High-Rise Bridge

O&M Cost Types	Alt #1 Shoring		Alt #2 Rehabilitation		Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
	5 Years (Shoring)	75+ Years (Alt 5)	40 Years (Rehab)	75+ Years (Alt 5)			
Inspections	$150k/yr	$200k/2yrs	$150k/yr	$200k/2yrs	$100k/2yrs	$200k/2yrs	
Intelligent Transportation Systems	$100k/yr						
Structural Health Monitoring Systems Required	$100k/yr						
Drainage	$40k/yr	$45k/yr	$40k/yr	$45k/yr	$45k/yr	$45k/yr	
Routine Maintenance	$350k/yr	$250k/yr	$350k/yr	$250k/yr	$200k/yr	$250k/yr	

R&R Cost Types	Alt #1 Shoring		Alt #2 Rehabilitation		Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
	5 Years (Shoring)	75+ Years (Alt 5)	40 Years (Rehab)	75+ Years (Alt 5)			
Structure (Bridge)		$479.0M		$561.1M	$130.1M	$164.7M	
Painting/Coating		$11.5M/25yrs		$11.5M/25yrs		$11.5M/25yrs	

Agenda

West Seattle High-Rise Bridge

- CBA & Workshop Objective
- Attribute Weighting
- Attribute Definition & Measurables
- Alternative ROM Cost Ranges
- **Next Steps**

Next Steps

West Seattle High-Rise Bridge

- Fill-out Performance Rankings for each Attribute
  - 1,3,5,7,9 ranking relative to baseline alternative
- Review of ROM Cost Range Development
  - O&M – input on difference between alternatives
  - Risk Monetization
  - Right-of-Way – value based on identified properties

- Upcoming Meeting to Discuss…
  - Performance “Benefit” Findings
  - ROM Cost Range Comments

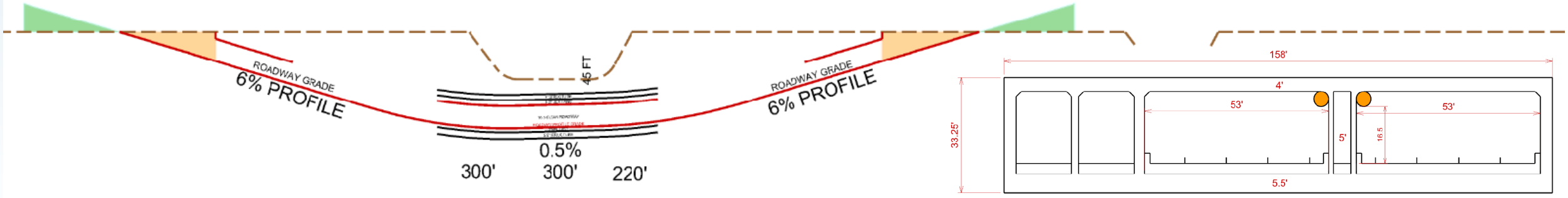
Thank You!!!

West Seattle High-Rise Bridge

Alternative No. 6 – Immersed Tube Tunnel

N88

West Seattle High-Rise Bridge



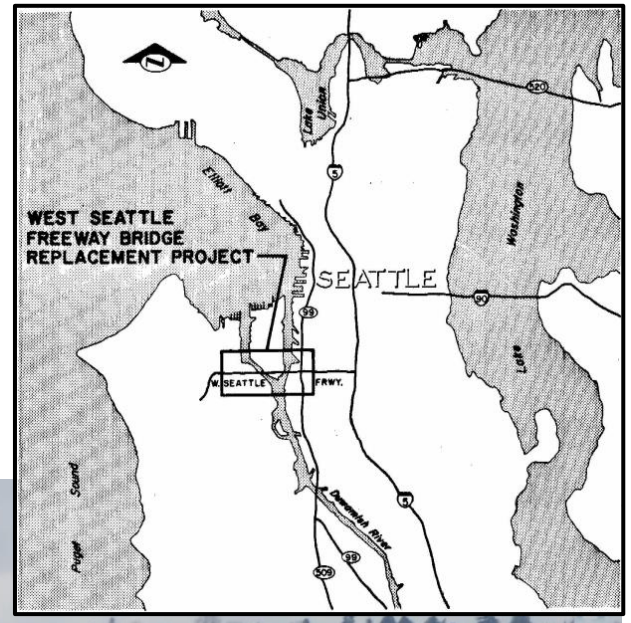
WORKSHOP PRESENTATIONS

WORKSHOP NO. 3

West Seattle High-Rise Bridge

N90

TAP Meeting – Cost Benefit Analysis



Client: City of Seattle | Location: Seattle, WA | Date: 08 September 2020

Agenda

West Seattle High-Rise Bridge

- **Attribute Weighting**
- Performance Scores “Value/Benefit”
- Alternative ROM Cost Ranges
- Summary of Findings

Attribute Weighting

West Seattle High-Rise Bridge

Which evaluation criterion contributes more to the overall success of the project?

											TOTAL	%
Bridge Maintenance, Inspection & Operation	A	1/8	2/7	4/5	3/6	2/7	3/6	2/7	2/7	0/9	3.1	5.7%
Constructability	B	5/4	6/3	6/3	8/1	7/2	4/5	5/4	2/7		6.7	12.1%
Environmental	C	5/4	6/3	6/3	6/3	3/6	4/5	3/6			5.9	10.7%
Equity	D	5/4	6/3	6/3	5/4	5/4	2/7				5.6	10.1%
Forward Compatibility	E	5/4	5/4	3/6	6/3	2/7					5.1	9.3%
Funding Opportunities	F	6/3	4/5	3/6	2/7						4.7	8.5%
Local Business Impacts	G	2/7	4/5	1/8							4.1	7.5%
Mobility Impacts	H	7/2	2/7								6.4	11.7%
Multi-modal Impacts	I	1/8									5.1	9.3%
Seismic/Safety	J										8.3	15.2%
											55.0	100%

CTF

Which evaluation criterion contributes more to the overall success of the project?

											TOTAL	%
Bridge Maintenance, Inspection & Operation	A	10/10	11/9	5/15	11/9	8/12	14/6	5/15	8/12	6/14	4.9	8.9%
Constructability	B	9/11	7/13	11/9	9/11	13/7	7/13	7/13	6/14		5.0	9.0%
Environmental	C	5/15	10/10	10/10	9/11	9/11	10/10	3/17			4.8	8.7%
Equity	D	15/5	13/7	14/6	10/10	11/9	5/15				6.6	11.9%
Forward Compatibility	E	6/14	8/12	5/15	5/15	2/18					4.0	7.2%
Funding Opportunities	F	10/10	7/13	8/12	2/18						5.1	9.2%
Local Business Impacts	G	2/18	8/12	2/18							4.2	7.6%
Mobility Impacts	H	13/7	6/14								6.7	12.2%
Multi-modal Impacts	I	4/16									5.7	10.4%
Seismic/Safety	J										8.2	14.9%
											55.0	100%

SDOT

N92

Which evaluation criterion contributes more to the overall success of the project?

											TOTAL	%
Bridge Maintenance, Inspection & Operation	A	5/2	6/1	7/0	5/2	4/3	3/4	0/7	3/4	5/2	6.4	11.7%
Constructability	B	4/3	7/0	5/2	3/4	4/3	0/7	5/2	4/3		5.9	10.6%
Environmental	C	4/3	2/5	2/5	1/6	1/6	1/6	1/6			3.3	6.0%
Equity	D	1/6	2/5	1/6	0/7	2/5	0/7				2.3	4.2%
Forward Compatibility	E	4/3	2/5	0/7	4/3	3/4					5.0	9.1%
Funding Opportunities	F	3/4	0/7	4/3	5/2						5.6	10.1%
Local Business Impacts	G	1/6	6/1	2/5							6.3	11.4%
Mobility Impacts	H	7/0	6/1								9.6	17.4%
Multi-modal Impacts	I	2/5									4.7	8.6%
Seismic/Safety	J										6.0	10.9%
											55.0	100%

TAP

Combined

	TOTAL	%	Top 6
Bridge Maintenance, Inspection & Operation	14.4	8.8%	
Constructability	17.5	10.6%	3
Environmental	14.0	8.5%	
Equity	14.4	8.7%	
Forward Compatibility	14.1	8.5%	
Funding Opportunities	15.3	9.3%	5
Local Business Impacts	14.6	8.8%	6
Mobility Impacts	22.7	13.8%	1
Multi-modal Impacts	15.5	9.4%	4
Seismic/Safety	22.5	13.7%	2
	165.0	100%	

Attribute Weighting

West Seattle High-Rise Bridge

	SDOT			TAP			CTF			Combined		
	TOTAL	%	Top 3	TOTAL	%	Top 3	TOTAL	%	Top 3	TOTAL	%	Top 6
Bridge Maintenance, Inspection & Operation	4.9	8.9%		6.4	11.7%	2	3.1	5.7%		14.4	8.8%	
Constructability	5.0	9.0%		5.9	10.6%		6.7	12.1%	2	17.5	10.6%	3
Environmental	4.8	8.7%		3.3	6.0%		5.9	10.7%		14.0	8.5%	
Equity	6.6	11.9%	3	2.3	4.2%		5.6	10.1%		14.4	8.7%	
Forward Compatibility	4.0	7.2%		5.0	9.1%		5.1	9.3%		14.1	8.5%	
Funding Opportunities	5.1	9.2%		5.6	10.1%		4.7	8.5%		15.3	9.3%	5
Local Business Impacts	4.2	7.6%		6.3	11.4%	3	4.1	7.5%		14.6	8.8%	6
Mobility Impacts	6.7	12.2%	2	9.6	17.4%	1	6.4	11.7%	3	22.7	13.8%	1
Multi-modal Impacts	5.7	10.4%		4.7	8.6%		5.1	9.3%		15.5	9.4%	4
Seismic/Safety	8.2	14.9%	1	6.0	10.9%		8.3	15.2%	1	22.5	13.7%	2
	55.0	100%		55.0	100%		55.0	100%		165.0	100%	

Agenda

West Seattle High-Rise Bridge

- Attribute Weighting
- **Performance Scores “Value/Benefit”**
- Alternative ROM Cost Ranges
- Summary of Findings

Performance Scores “Value/Benefits”

N95

West Seattle High-Rise Bridge

	SDOT	TAP	WSP	Combined
	Performance	Performance	Performance	Performance
Alt 1	333	303	425	353
Alt 2	500	500	500	500
Alt 4	582	461	593	546
Alt 5	538	435	554	509

Agenda

West Seattle High-Rise Bridge

- Attribute Weighting
- Performance Scores “Value/Benefit”
- **Alternative ROM Cost Ranges**
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Alternative ROM Cost Ranges – General Assumptions

N97

West Seattle High-Rise Bridge

- ROM cost ranges are in part metrics based, and in-part based on actual engineering and calculated quantities.
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- Social economic impacts are not quantitatively reported in the reported cost ranges. Social economic impacts have been qualitatively defined in the performance (i.e. benefits).

Alternative ROM Cost Ranges – General Assumptions

West Seattle High-Rise Bridge

- Types of Costs
  - Capital Costs
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Alternative ROM Cost Ranges – General Approach

N99

West Seattle High-Rise Bridge

- Types of Costs
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Second Ask: Cost input on impacted properties

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- Design and Planning Costs
  - Termed as “Soft Costs” or “Other Costs” in the estimates
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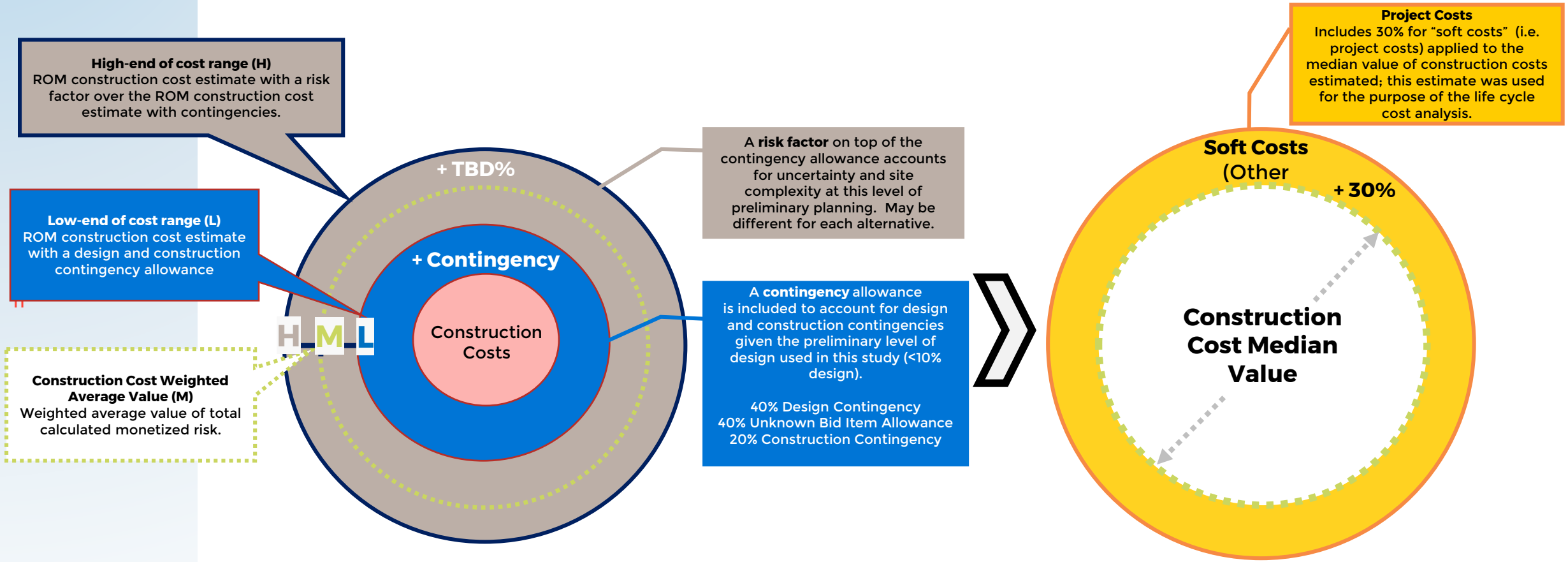
Alternative ROM Cost Ranges – General Approach

N100

West Seattle High-Rise Bridge



Step 1. Estimate the range of **construction costs.**

Step 2. Estimate **project costs** for the life cycle analysis.

Alternative ROM Cost Ranges – General Approach

N101

West Seattle High-Rise Bridge

- Types of Costs
  - Capital Costs
  - **Life Cycle Costs**

- Construction Durations
  - Developed preliminary schedule
  - Considers when in time work would occur
- Anticipated Asset Life
  - Equivalent durations formulated around AASHTO and FHWA 75 year design life
  - Start in year 2021 when shoring starts (Alternative No. 01)
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  - Repair/rehabilitation 50-years into their life. Repair assumed to be $50M.

Alternative ROM Cost Ranges – Capital Cost Parameters

West Seattle High-Rise Bridge

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Bridge Importance Category	Essential	Normal	Essential	Essential	
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Permissible In-Ground Hinging	No	Yes	No	No	
Vertical Excitations	Yes	No	Yes	Yes	
Structure Type	Cable-Stayed	N/A	CIP Conc Box Girder	Cable-Stayed	
Inclusive of Sound Transit Yes/No If so, what year?	 Yes 2033	 No – See Note --	 Yes 2026	 Yes 2026	
No. of Lanes Each Direction	4	4	4	4	
Approach Modifications	No	No	No	Yes	
Life Cycle	2021 - 2101				

NOTE: Could potentially accommodate if reconfigured channelization (i.e. use of reversible lanes.

Alternative ROM Cost Ranges – Life Cycle Parameters

West Seattle High-Rise Bridge

Parameter	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Life Cycle	2021 - 2101				
Shoring to Restore Live Load	**2021-2024**	--	--	--	TBD
Demolition	2029	2063	**2022-2023**	**2022-2023**	
Rehabilitation	2083	**2021-2022**	2075	2076	
Foundation Retrofit	--	2032	**2024**	--	
Superstructure Replacement	--	--	**2024-2025**	--	
“In-Kind” Bridge Replacement	2030-2033	2063-2066	--	**2024-2026**	
Tunnel					

BOLD = Considered in Capital Cost Calculation

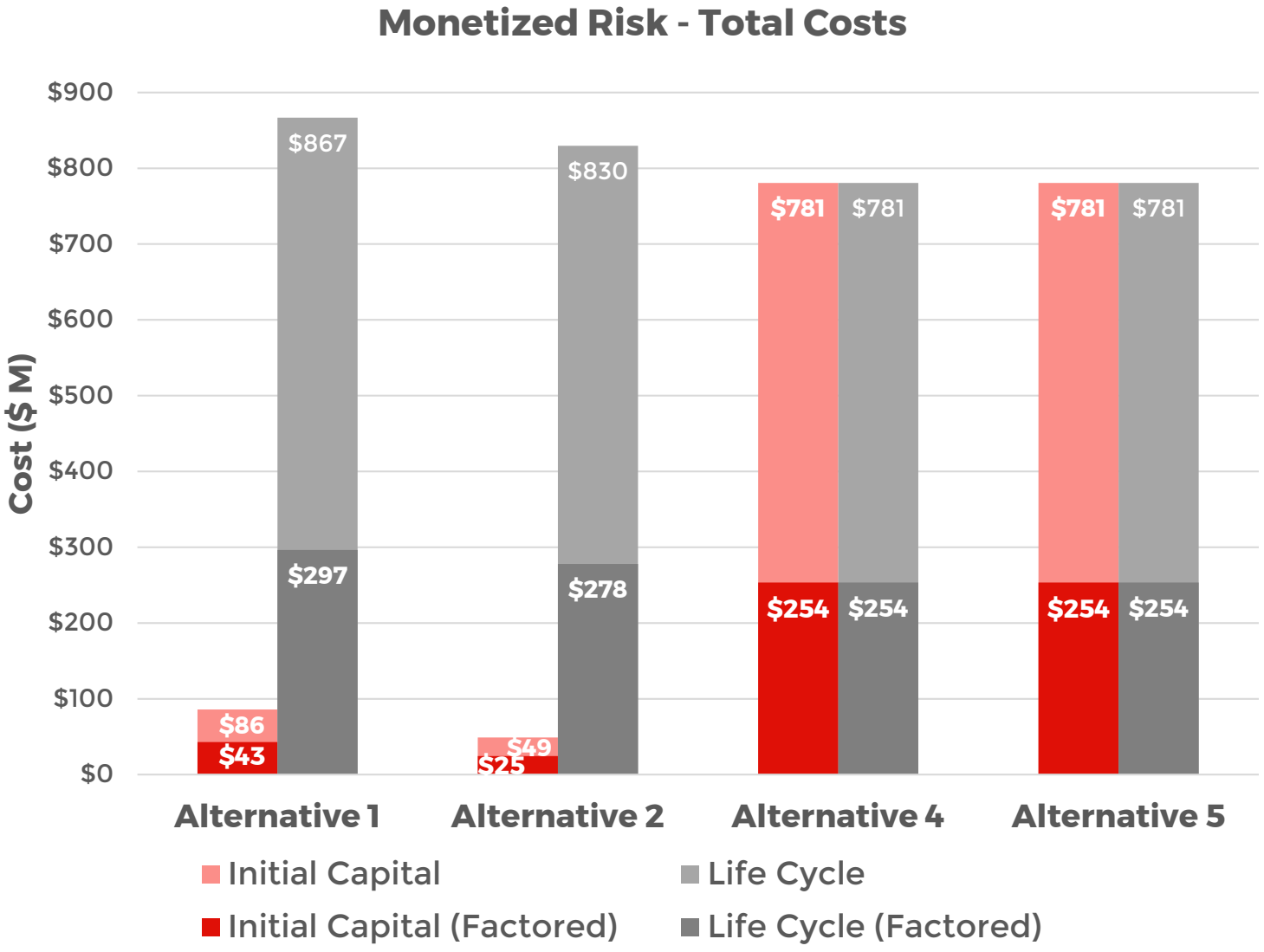
Alternative ROM Cost Ranges – Monetized Risk Items

West Seattle High-Rise Bridge

Key Risk Items*:

- Change in geotechnical standards for seismic acceleration
- Bridge Stabilization measure performance
- Bridge Importance classification
- FAA height restrictions
- USCG navigation clearance

*Selected risk items have been presented here. Additional risk items which may have monetary impact are included in the full risk register



Alternative ROM Cost Ranges – Project Costs

N105

West Seattle High-Rise Bridge

Project Costs	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Construction Costs	$53.9M	$30.1M	$199.5M	$294.1M	TBD
Design Contingency (40%)	$21.6M	$12.0M	$79.8M	$117.6M	
Allowance (40%)	$21.6M	$12.0M	$79.8M	$117.6M	
Subtotal	$97.1M	$54.2M	$359.2M	$529.4M	
Construction Contingency (20%)	$19.4M	$10.8M	$71.8M	$105.9M	
Right-of-Way (TCE's)	$xx.xM	$xx.xM	$xx.xM	$xx.xM	
Subtotal	$116.5M	$65.1M	$431.0M	$635.3M	
Monetized Risk	$43.0M	$24.5M	$253.5M	$253.5M	
Subtotal	$159.5M	$89.6M	$684.5M	$888.8M	
Other Costs (30%)	$47.8M	$26.9M	$205.4M	$266.6M	
Total Project Costs	**$207.3M**	**$116.4M**	**$889.9M**	**$1155.4M**	

Alternative ROM Cost Ranges – Life Cycle Costs

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West Seattle High-Rise Bridge

Costs	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Project Costs	$207.3M	$116.4M	$889.9M	$1155.4M	
Operating & Maintenance	$32.8M	$37.5M	$22.4M	$30.1M	
Repair & Rehabilitation	$1523.3M	$1420.8M	$291.3M	$325.9M	
Total Life Cycle Costs	**$1763.4M**	**$1574.7M**	**$1203.6M**	**$1511.4M**	

Agenda

West Seattle High-Rise Bridge

- Attribute Weighting
- Performance Scores “Value/Benefit”
- Alternative ROM Cost Ranges
- **Summary of Findings**

Alternative Performance “Benefits”

West Seattle High-Rise Bridge

N108

Alternative	Performance Score	Ranking
Temporary Shoring to Restore Live Load (3 to 5 years)	353	4
Direct Rehabilitation to Restore Live Load (15 to 40 years)	500	3
Accelerated Superstructure Replacement (50 to 75 years)	546	1
Accelerated Bridge Replacement (75+ years)	509	2
Off-Alignment Immersed Tube Tunnel (75+ years)	TBD	TBD

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	SDOT			TAP			WSP			Combined		
	Performance	Capital Value Index	Life Cycle Value Index	Performance	Capital Value Index	Life Cycle Value Index	Performance	Capital Value Index	Life Cycle Value Index	Performance	Capital Value Index	Life Cycle Value Index
Alt 1	333	2.09	0.19	303	1.90	0.17	425	2.66	0.24	353	2.22	0.20
Alt 2	500	5.58	0.32	500	5.58	0.32	500	5.58	0.32	500	5.58	0.32
Alt 4	582	0.85	0.48	461	0.67	0.38	593	0.87	0.49	546	0.80	0.45
Alt 5	538	0.60	0.36	435	0.49	0.29	554	0.62	0.37	509	0.57	0.34

Summary of Findings

West Seattle High-Rise Bridge

Ranking Summary…

Alternative	Performance Score	Capital Costs	Life Cycle Costs	Capital Cost Value Index	Life Cycle Cost Value Index
Temporary Shoring to Restore Live Load (3 to 5 years)	4	2	4	2	4
Direct Rehabilitation to Restore Live Load (15 to 40 years)	3	1	3	1	3
Accelerated Superstructure Replacement (50 to 75 years)	1	3	1	3	1
Accelerated Bridge Replacement (75+ years)	2	4	2	4	2
Off-Alignment Immersed Tube Tunnel (75+ years)					

Thank You!!!

West Seattle High-Rise Bridge

Summary of Findings

West Seattle High-Rise Bridge

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Based on Capital Costs…

Alternative	Value Index	Ranking
Temporary Shoring to Restore Live Load (3 to 5 years)	2.22	2
Direct Rehabilitation to Restore Live Load (15 to 40 years)	5.58	1
Accelerated Superstructure Replacement (50 to 75 years)	0.80	3
Accelerated Bridge Replacement (75+ years)	0.57	4
Off-Alignment Immersed Tube Tunnel (75+ years)	TBD	TBD

Based on Life Cycle Cost Analysis…

Alternative	Value Index	Ranking
Temporary Shoring to Restore Live Load (3 to 5 years)	0.20	4
Direct Rehabilitation to Restore Live Load (15 to 40 years)	0.32	3
Accelerated Superstructure Replacement (50 to 75 years)	0.45	1
Accelerated Bridge Replacement (75+ years)	0.34	2
Off-Alignment Immersed Tube Tunnel (75+ years)	TBD	TBD

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Maintenance, Inspection & Operation

A measure of the services and systems necessary for the operation and preservation of the bridge and corridor.

Measurables	Unit of Measure	Alt #1 Shoring		Alt #2 Rehabilitation		Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
		5 Years (Shoring)	75+ Years (Alt 5)	40 Years (Rehab)	75+ Years (Alt 5)			
Inspection Requirements	Frequency & Level-of-Effort (High/Med/Low) Special Inspections (Yes/No)	6-12 Mo High Yes	12-24 Mo High Yes	6-12 Mo High Yes	12-24 Mo High Yes	24 Mo Med No	12-24 Mo High Yes	24 Mo Med Effort Yes
Intelligent Transportation Systems Required	Yes/No	Yes	No	No	No	No	No	Yes
Structural Health Monitoring Systems Required	Yes/No How many systems required?	Yes 3	No	Yes 3	No	No	No	No
Painting/UV protection Required	Yes or No	No	Yes	Yes	Yes	No	Yes	No
SCORE	(1,3,5,7,9)			5				

Other Considerations: Alternative 5 (

nt including dampers, cables and anchorages

First Ask: Provide 1, 3, 5, 7, 9 Score
Alternative 2 = Baseline (gets a score of 5)
Other Alternatives = 1 if baseline is strongly preferred
= 3 if baseline is preferred
= 5 if is equal to the baseline
= 7 if alternative is preferred over the baseline
= 9 if alternative is strongly preferred over the baseline

Attributes Definition & Measurables

N113

West Seattle High-Rise Bridge

Constructability

A measure of the ease and efficiency an alternative can be built relative to the schedule and potential means/methods permitted by the contract.

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Schedule impacts	Duration of project	39 Mo	12 Mo	42 Mo	45 Mo	
Complexity	Standard construction or complex	Standard	Complex	Standard	Complex	
Specialty Contractors and Equipment	Are any required and if so how many	Yes, 2	Yes, 5	Yes, 5	Yes, 5	
Utility Relocations	Impacts on existing utilities, scope and magnitude of relocations	Average	Minor	Major	Major	
In Water Work	Amount of in water work and how many in water windows are needed	None	Foundation Retrofit - 2 Windows	Foundation Retrofit - 2 Windows	None -Ex. Piers Remain	
Demolition	Amount and complexity of demolition required	Complex & Truss Demo	Complex	Complex	Complex	
Poor soil conditions	Is substantial foundation work required?	No	Yes	Yes	Yes	
Staging/Laydown area required	Required footprint	Average	Minimal	Average	Large	
SCORE	**(1,3,5,7,9)**		**5**			

Other Considerations Determined Not to Influence Decision: List of other considerations; etc.

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Environmental

A measure of potential adverse effects to the built or natural environments

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Noise and Vibration	Pile driving be required?	No	Yes	Yes	No	
Duwamish Waterway	In water work required (timing and duration) Proximity of construction ground disturbance to shoreline (1) Limited to temporary barges during demolition and construction (2) New foundations and piers for replacement structure are located away from waterway. (3) Foundation retrofit/strengthening requires ground disturbance within the shoreline and near the waterway	(1) and (2)	(1), (2), and (3)	(1), (2), and (3)	(1) and (2)	
Section 4(f) Resources	Use of parks, trails, open space, wildlife refuges Proximity of construction to known archaeological sites and known historic structures listed or eligible for the National Register of Historic Places (1) No use of parks, trails, open space or wildlife refuges (2) Potential use of Nucor Steel NHRP Building or Pacific Forge historic district (see list)	(1) and (2) List: Shoring trusses, and In kind bridge replacement piers	(1) and (2) List: Foundation retrofit, and In kind bridge replacement piers	(1) and (2) List: In kind bridge replacement piers	(1) and (2) List: In kind bridge replacement piers	
Emissions	Tons of GHG (from VMT) x construction duration	210,000 more tons	128,000 more tons	161,000 more tons	161,000 more tons	
Wildlife Impacts	Timing and duration of construction proposed during falcon/Great Blue Heron breeding season	Two phases of construction	Three phases of construction	Two phases of construction	Two phases of construction,	
SCORE	(1,3,5,7,9)		5			

Attributes Definition & Measurables

N115

West Seattle High-Rise Bridge

Equity

The incremental impacts resulting from bridge closure and construction that disproportionately affect historically and currently marginalized communities.

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Duration of bridge closure	Time to restore full capacity of bridge operations[1] (years full closure starting 1/2021)	3 lanes: 2024 New Bridge: 2033 (7.5 yrs)	All lanes: 2022 New bridge: 2066 (5.25 yrs)	New bridge: 2026 (5.25 yrs)	New bridge: 2026 (5.25 yrs)	
Incremental Vehicle-Miles Travelled	Incremental VMT during closure through marginalized communities[2]	Higher	Low	Medium	Medium	
Incremental Vehicle Travel	Incremental vehicular travel through marginalized communities (crossing the Duwamish Waterway)[3]	85.3 million more trips	58.8 million more trips	62.5 million more trips	62.5 million more trips	
Incremental Travel Time	Cumulative PHT during construction through marginalized communities[4]	Higher	Medium	Lower	Lower	
Number of Construction Events / Community Disruption	Repeat closure events due to preservation or construction marginalized communities would experience in a 75 year timeframe[5]	2 events within 3 years totaling 7.5 years	2 events within 40 years totaling 5.25 years	1 event totaling 5.25 years	1 event totaling 5.25 years	
SCORE	**(1,3,5,7,9)**		**5**			

1 - Time to restore full capacity of bridge operations; captures number of occasions a community would be subjected to more than one bridge closure episode lasting >1 year; begins on March 23, 2020.
2 - This is intended to capture associated incremental emissions (air quality impacts and GHG), risk of incremental severe crashes (mortality, severe injuries), noise and other negative indirect externalities associated with vehicular travel through marginalized communities.
3 - Vehicle trips (all types) crossing the Duwamish Waterway through diversion routes (through marginalized communities) x Construction Duration
4 - Cumulative PHT during construction and PHT in final configuration (tunnel is only difference) through marginalized communities through SDOT diversion routes. Measurable is intended to also capture qualitatively incremental vehicular delays during bridge closure through marginalized communities, and impacts on travel reliability for transit (speed and reliability).
5 - Measurable does not capture OTHER construction projects in the area happening in tandem; assumes haul routes will not go through marginalized communities.

Attributes Definition & Measurables

West Seattle High-Rise Bridge

Forward Compatibility

Ability to accommodate future roadway configurations including light rail

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Future roadway configurations	Maintain a minimum of existing configuration and improve design features (Year)	3 lanes 2024-2029 7 lanes starting 2033	7 lanes 2022 (less 2063-2066)	7 lanes 2026 (some lane closures 1 year)	7 lanes 2026 (some lane closures 1 year)	
Accommodate light rail	Yes or No: Ability to accommodate light rail on bridge by 2032 (new bridge)	NO	NO	YES	YES	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

N117

West Seattle High-Rise Bridge

Funding Opportunities

Alternative eligibility for various funding sources and evaluation of funding and financing opportunities.

Measurables	Unit of Measure (Weighted Ranking 1-9; Except Eligibility)	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Project eligibility	Yes or No: Count of eligible of funding sources (out of 42)	29	29	29	29	
Funding revenue generation potential	Historical experience on revenue generation ranges	5.7	5.4	5.7	5.7	
Stability of the funding source	Funding sources are scored based on known continuation, likely continuation, and likely elimination	6.2	5.9	6.2	6.2	
Likelihood of funding	Historical experience on competitiveness of funding sources and successful probability ranges	2.0	1.9	2.0	2.0	
Timing for availability	Availability of funding using ranges based on typical years to secure funds	7.0	6.5	7.0	7.0	
Administrative requirements	Existing administrative structure in place, incremental requirements, no structure in place	7.7	7.3	7.7	7.7	
Legal authority	Existing legal authority, Legal at the State level but City of Seattle is not authorized, no State or City authorization	8.8	8.3	8.8	8.8	
Income and racial equity	Measure is progressive, progressive or regressive depending on implementation, or regressive	7.6	7.1	7.6	7.6	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

N118

West Seattle High-Rise Bridge

Local Business Impacts

Impacts to adjacent and local business during construction and in the final configuration

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Bridge Closures Impacts	Trip reduction x discretionary spending per trip (could impact BIPOC-owned businesses)	Net reduction of $3,5 million	Net increase of $6.7 million	Net reduction of $0.9 million	Net reduction of $0.9 million	
Direct/Indirect Economic Construction from construction	Capital expenditures	7,451 FTEs	7,976 FTEs	6,766 FTEs	9,548 FTEs	
Temp Const. Easements	Number/duration of TCE's (could impact BIPOC-owned businesses)	1 TCE for 1 yr + Alt 4/5 TCE's for 2.75 yrs	1 TCE for 1 yr + Alt 4/5 TCE's for 3.75 yrs	1 large TCE for 3.75 yrs	2 TCE's for 3.75 yrs	
Utility interruption	# of times impact utilities on and under bridge	For Demo and new bridge construct (Alt 4/5)	For Demo and new bridge construct (Alt 4/5)	For Demo and new bridge between piers 15-18 (east line)	For Demo and new bridge between piers 13-18 (east & west lines)	
Access impacts to local properties	Impacts to existing property access during construction and final configuration	No restrictions	Min. restriction during construction to S. Harbor Island via SW Manning	Potential restrictions to S. Harbor Island via SW Manning	Potential restrictions to S. Harbor Island via SW Manning	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

N119

West Seattle High-Rise Bridge

Mobility Impacts

The effects of construction on regional mobility including travel times, routes and mode shifts as well as increased mobility of final facility.

Measurables	Unit of Measure (total over forecast period)	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Travel Time	Cumulative PHT during construction and PHT in final configuration (tunnel is only difference)	42.9 million more hours	40.0 million more hours	30.1 million more hours	30.1 million more hours	
Travel Distance	VMT x duration during construction and VMT in final configuration (tunnel is only difference)	563.9 million more miles	353.6 million more miles	398.8 million more miles	398.8 million more miles	
Non-Vehicle Trips	Trips (mode shift to bikes from car) during construction. This is a health benefit.	60.9 million more bike/ped trips	45.7 million more bike/ped trips	44.1 million more bike/ped trips	44.1 million more bike/ped trips	
Regional Mobility Impacts	Impacts to other regional construction projects. Does other construction occur when bridge is closed and there is more traffic on surface streets?	E Marginal: bridge closed; WSBLE: ½ during bridge closure	E Marginal: ½ during bridge closure; WSBLE: bridge open	E Marginal: bridge closed; WSBLE: bridge open	E Marginal: bridge closed; WSBLE: bridge open	
Safety	Crashes/VMT on detour routes compared to W Seattle Bridge (during construction), and typical tunnel versus bridge (final configuration)	553 more crashes	347 more crashes	391 more crashes	391 more crashes	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

N120

West Seattle High-Rise Bridge

Multi-modal Impacts

Impacts to emergency vehicle and transit during construction as well as pedestrians and bicycles in the final configuration

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Incremental Transit Service during Construction	Increase transit capacity	1,280 daily units	753 daily units	982 daily units	982 daily units	
Bicycle Traffic Accommodation	High/med/low (no bike accommodation on bridge)	Low	Low	Low	Low	
Pedestrian Traffic Accommodation	High/med/low (no ped accommodation on bridge)	Low	Low	Low	Low	
Emergency Access	Total Response time increase during construction	38 hours	27 hours	27 hours	27 hours	
Freight Mobility	High/med/low	422,000 hours	360,000 hours	295,000 hours	295,000 hours	
SCORE	**(1,3,5,7,9)**		**5**			

Attributes Definition & Measurables

N121

West Seattle High-Rise Bridge

Seismic/Safety

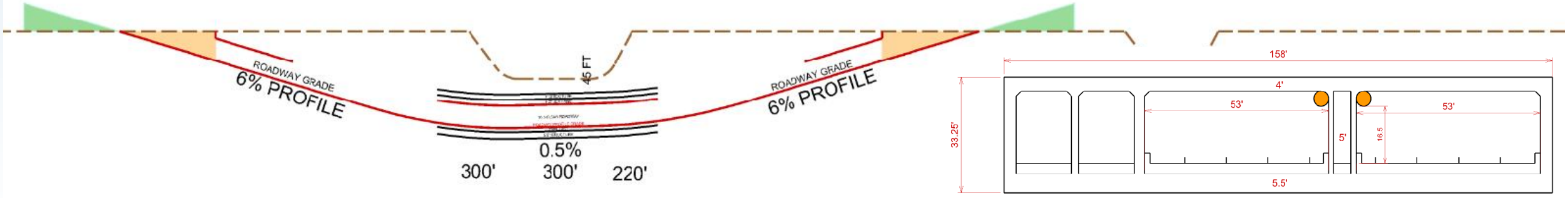
Expected post-earthquake service and damage levels relative to the design seismic event including consideration of implementation.

Measurables	Unit of Measure	Alt #1 Shoring	Alt #2 Rehabilitation	Alt #4 Replacement	Alt #5 Replacement	Alt #6 Tunnel
Seismic Hazard Levels	100-Year Return Period 210-Year Return Period 975-Year Return Period	 X X	X X	 X X	 X X	 X X
Operational Classification *Post design earthquake service* *(No, Limited, or Full Service)*	100-Year Return Period 210-Year Return Period 975-Year Return Period	 Full Limited	Full No	 Full Limited	 Full Limited	 Full Limited
In-Ground Hinging Permitted	Yes or No	No	Yes	No	No	No
Vertical Excitations Considered	Yes or No	Yes	No	Yes	No	Yes
Seismic Compliance Established	Year	2033	2032	2025	2026	TBD
SCORE	**(1,3,5,7,9)**		**5**			

Other Considerations: Owner specified project specific criteria needed to be developed for replacement structures considered as “essential” or “critical”.

Alternative No. 6 – Immersed Tube Tunnel

West Seattle High-Rise Bridge



Alternative No. 6 – Immersed Tube Tunnel

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West Seattle High-Rise Bridge