



Enhanced Seismic Retrofit Technical Report

Multnomah County | Earthquake Ready
Burnside Bridge

Portland, Oregon

January 29, 2021



Earthquake Ready Burnside Bridge Enhanced Seismic Retrofit Technical Report

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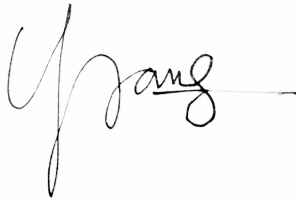
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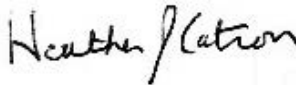
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Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ADA	Americans with Disabilities Act
API	Area of Potential Impact
BES	Bureau of Environmental Services
C/D	capacity-to-demand ratio
CFR	Code of Federal Regulations
CIP	capital improvement plan
City	City of Portland, Oregon
Consultant	HDR Engineering, Inc. and subconsultant partners
County	Multnomah County, Oregon
Criteria	seismic design criteria
CSO	combined sewer overflow
CSZ	Cascadia Subduction Zone
DSS	dynamic soil-structure interaction
EB	eastbound
EIS	environmental impact statement
EQRB	Earthquake Ready Burnside Bridge
ERS	earthquake resisting system
FO	full operation
LO	limited operation
LRFD	load and resistance factor design
LRT	light rail transit
NAVD	North American vertical datum
NEPA	National Environmental Policy Act
ODOT	Oregon Department of Transportation
OSSPAC	Oregon Seismic Safety Policy Advisory Commission
PLC	programmable logic controller
Project	Earthquake Ready Burnside Bridge Project
PS&E	plans, specifications and estimates
RCDG	reinforced concrete deck girder
ROW	right-of-way

RSA	response spectrum analysis
SDC	seismic design criteria
UPRR	Union Pacific Railroad
USCG	U.S. Coast Guard
UPS	uninterruptible power supply
WB	westbound

Executive Summary

Objectives

This Burnside Bridge Enhanced Seismic Retrofit Alternative technical report discusses the following topics:

- Seismic vulnerabilities of the existing bridge.
- Feasible concepts for seismic retrofit alternatives.
- Description of a conceptual-level seismic retrofit analysis of the existing Burnside Bridge. The analysis includes a list of feasible seismic retrofit strategies for the bridge to withstand major seismic events as defined in the Project's Seismic Design Criteria.
- Necessary rehabilitation measures to improve the bridge load rating capacity to meet current standards.

To establish a consistent and reasonable set of alternative impacts, benefits, and construction costs prior to performing detailed designs, structural typical sections were developed. They do not represent a decision on bridge width, lane configurations, lane allocations, or even structure type. Instead, they serve as a basis of design in order to establish bridge footprint, verify ability to meet clearances, evaluate seismic demands, and impacts related to construction. These parameters are expected to change and evolve during the design phase.

Enhanced Seismic Retrofit Alternative

The major seismic vulnerabilities of the existing bridge are identified in Section 5 and Appendix B of this report. The load rating deficiencies of the existing bridge are identified in Section 6.

The Enhanced Seismic Retrofit Alternative was developed to achieve the same performance and safety standards as the bridge replacement alternatives.

A temporary diversion bridge would be required to keep the Burnside Street route open during the construction unless the traffic could be detoured.

The navigational channel would remain open during construction, except for temporary closures, such as during the lifting of the bascule leaves.

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1 Introduction

Multnomah County will be directing the study and development of an environmental impact statement (EIS) as part of the National Environmental Policy Act (NEPA) assessment for the Earthquake Ready Burnside Bridge (EQRB) river crossing. The following summarizes the EQRB Project (Project) background, the problem being addressed, and the Project's intent.

1.1 Background and Bridge Description

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

The existing Burnside Bridge (see Figure 1-1) carries a total of 35,000 vehicles per day, with 19,000 eastbound and 16,000 westbound vehicles (traffic counts are from 2019). Built in 1926, the Burnside Bridge is an aging structure requiring increasingly frequent and significant repairs and maintenance. The Burnside Bridge crosses the Willamette River, Interstate Highway 5 (I-5), multiple City of Portland streets, parking lots, parks, TriMet MAX lines, and other facilities along Burnside Street. The bridge carries three eastbound and two westbound lanes of vehicle traffic, as well as bike lanes and sidewalks in each direction. The total bridge length is approximately 2,307 feet and consists of three structures (see Figure 1-2 and Figure 1-3) (Multnomah County 1924):

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

The bridge is designated a historically significant structure and is listed on the National Register of Historic Places.

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



1.1.1 The Need for Seismic Resilience

Geologically, Oregon is located in the Cascadia Subduction Zone (CSZ), making it subject to some of the world's most powerful recurring earthquakes. The last major earthquake in Oregon occurred over 300 years ago in 1700, a timespan that exceeds 75 percent of the intervals between the major earthquakes to hit Oregon over the last 10,000 years. There is a significant risk that the next event will occur relatively soon. Such an earthquake will cause major ground shaking, settling, and landslides, and it is expected to result in major and widespread damage to buildings, utilities, and transportation facilities (OSSPAC 2013), leaving the City of Portland divided and isolating members of the community.

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

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

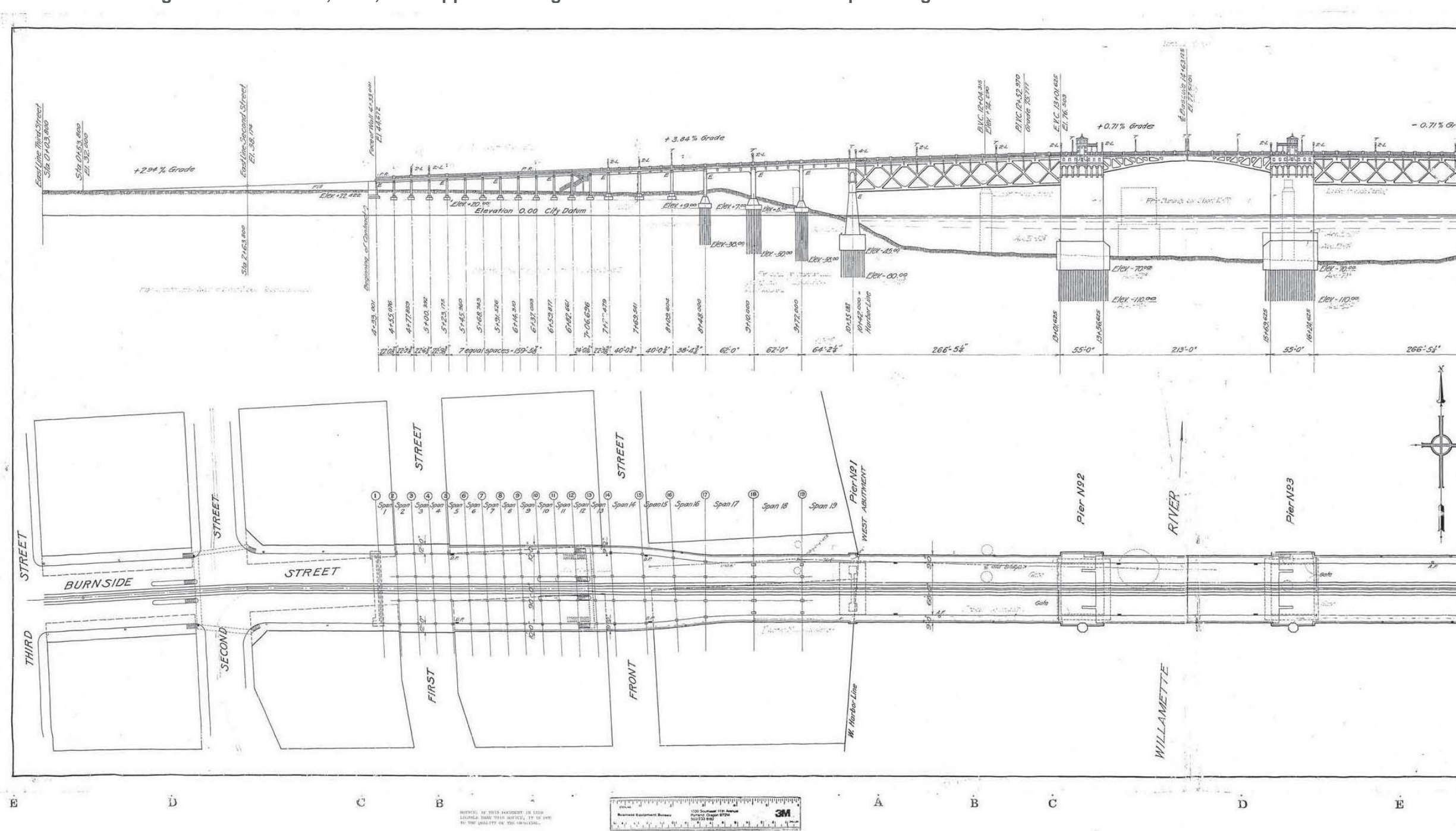
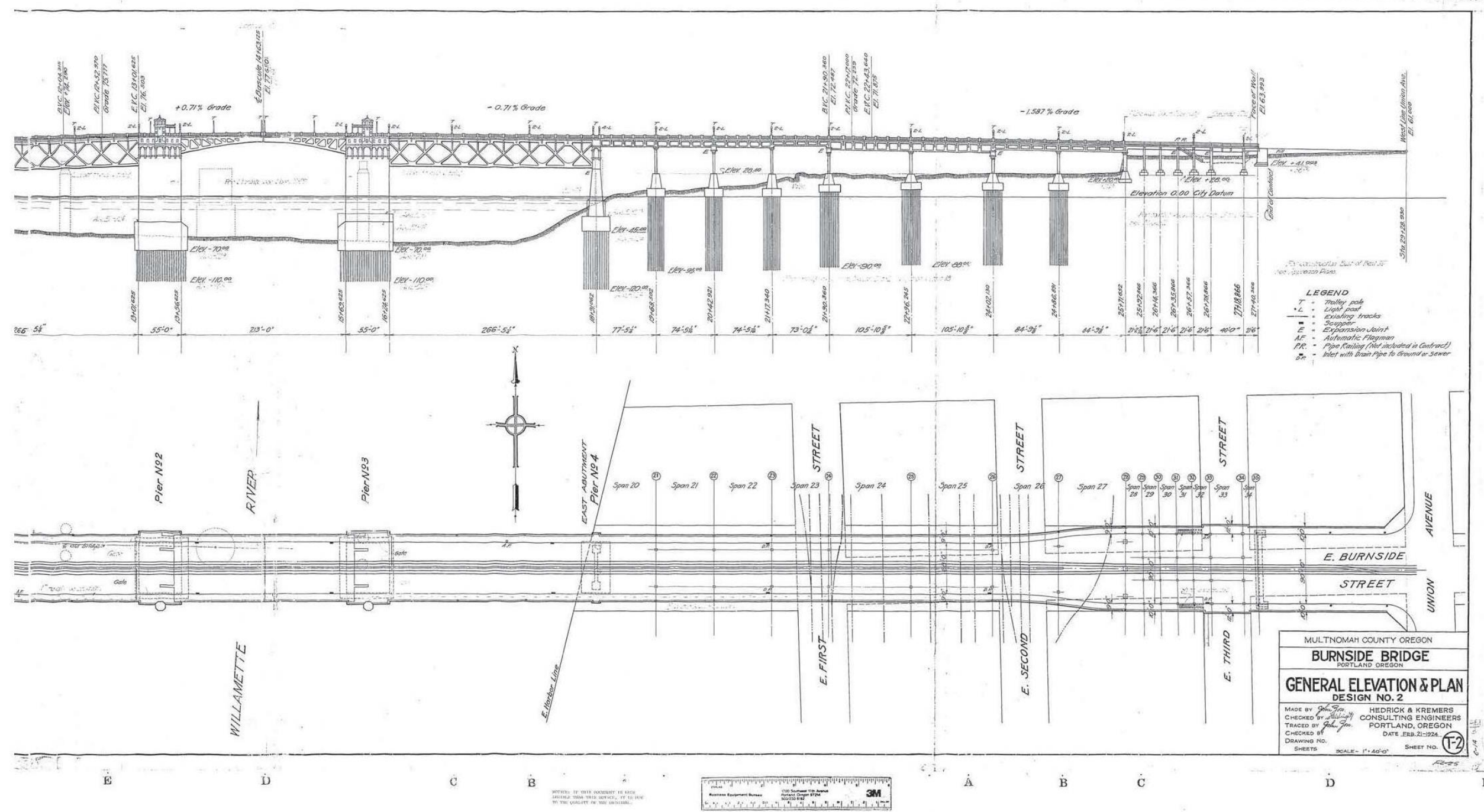


Figure 1-3. Burnside Original As-Built Plans, 1924, a Portion of the Main River Span Bridge and the East Approach Bridge



In response to this risk from a future seismic event, Multnomah County recently completed its 20-year Willamette Bridges Capital Improvement Plan (CIP) in 2015. This plan was a comprehensive study of the County's six bridges crossing the Willamette River, focusing mainly on the four downtown structures, and provided a high-level assessment of their conditions and a list of required improvements to promote safety and reliability for those critical transportation infrastructures. The CIP identified the Burnside Bridge seismic resiliency as a top priority for Multnomah County in the next 20 years.

1.1.2 Burnside Street Lifeline Designation

The Burnside Bridge is designated as the only County-owned Primary Emergency Transportation Route across the Willamette River in downtown Portland in a 1996 report to Metro's Regional Emergency Management Group. This group was formed by intergovernmental agreement among the region's cities, counties, Metro, and the Red Cross to improve disaster preparedness, response, recovery, and mitigation plans and programs. (Regional Emergency Transportation Routes, Portland Metropolitan Region, 1996).

The Burnside Street emergency route is approximately 18.7 miles long and extends from SW 57th Avenue in Washington County to U.S. Highway 26 in Gresham, crossing the Willamette River via the Burnside Bridge.

Other agency plans have also identified Burnside Street as an important lifeline route. For example, the City of Portland's Citywide Evacuation Plan addresses evacuation needs for general disasters. The plan identifies Burnside Street as a secondary east-west evacuation route and an emergency transportation route (City of Portland 2017).

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

- "Enhance the proposed (state) Highway Lifeline Maps by considering the use of highway segments owned by cities and counties to provide access to critical facilities. Prioritize local routes to provide access to population centers and critical facilities from the identified (state) Tier-1 routes" (OSSPAC 2013, 105-109).
- "When developing projects for seismic retrofit of (state) highway facilities, consider whether a local agency roadway may offer a more cost-effective alternative for all or part of a lifeline route" (OSSPAC 2013, 105-109).
- Recommendation for "seismically upgrading lifeline transportation routes into and out of major business centers statewide by 2030" (OSSPAC 2013, xiii).

1.1.3 Project Intent

The primary purpose of the Project is to build a seismically resilient Burnside Street lifeline crossing over the Willamette River that will remain fully operational and accessible for vehicles and other modes of transportation following a major CSZ earthquake. The Burnside Bridge would provide a reliable crossing for emergency response, evacuation, and economic recovery after an earthquake. Additionally, the bridge would provide a long-term safe crossing with low maintenance needs. It will enable the following:

- Emergency medical, fire, and life safety response
- Evacuation of survivors to safe locations
- Reunification of families and households
- Post-disaster restoration of services, and
- Regional recovery

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

The Project would be compatible with existing major infrastructure.

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

1.1.4 Enhanced Seismic Retrofit Technical Report Intent

This Enhanced Seismic Retrofit Technical Report documents the technical aspects of the enhanced retrofit concepts. Technical options and alternatives were developed during the feasibility study phase and are documented in the *Earthquake Ready Burnside Bridge Feasibility Study Report* (Multnomah County 2018). The seismic retrofit alternatives were narrowed down to a preferred Enhanced Seismic Retrofit Alternative that has been further developed in this NEPA phase of the Project.

The enhanced retrofit concepts include the following seismic retrofit measures and improvements to the bridge load rating capacities:

- Identify the seismic vulnerabilities of the existing bridge.
- Identify the load rating deficiencies of the existing bridge.
- Develop feasible concepts for seismic retrofit alternatives.
- Develop feasible concepts for improving load rating capacities.
- Perform a conceptual-level enhanced retrofit analysis of the existing Burnside Bridge. Develop a list of feasible of enhanced retrofit strategies, including any necessary rehabilitation measures, to withstand major seismic events as defined in the Project's Seismic Design Criteria.

This technical report does not represent a decision on bridge type, size, and location; it serves as a basis of design in order to establish a bridge footprint, verify the ability to

meet clearances, and to evaluate seismic demands as well as impacts related to construction.

1.2 Major Transportation Facilities and Critical Infrastructure

The seismic resiliency of the Burnside Bridge is impacted by the adjacent major transportation facilities and buildings (see Figure 1-4). The alternatives considered the following existing facilities during the conceptual design process:

1. The Tri-County Metropolitan Transportation District of Oregon (TriMet) light rail lines run under the west approach of the bridge at 1st Avenue on the west side.
2. The City of Portland roadway facilities: Naito Parkway runs under the west approach of the bridge; 2nd and 3rd Avenues run under the east approach spans; and Martin Luther King Jr. Boulevard and Grand Avenues are adjacent to the east approach.
3. The City of Portland large-diameter combined sewer overflow (CSO) pipes run under both the west approach and east approach bridge spans.
4. I-5 south- and northbound main lines and the ramps to and from Interstate Highway (I-84) run under the east approach of the bridge.
5. Union Pacific Railroad (UPRR) lines run under the east approach of the bridge.
6. River navigation channel for the U.S. Coast Guard (USCG) and other river users.
7. The Portland Streetcar runs just east of the bridge on Martin Luther King Jr. Boulevard and Grand Avenue.
8. The west and east approaches of the bridge are within close proximity to adjacent buildings, some having sidewalk access from Burnside Street.

Figure 1-4. Adjacent Major Transportation Facilities and Buildings of Burnside Bridge



2 Design Criteria and Other Considerations

The Enhanced Seismic Retrofit Alternative will be designed to current City, County, State, and national standards as applicable for the features and components of the Alternative. The Alternative will be designed for a minimum 100-year design life.

Subsequent sections describe the Project-specific technical reports and applicable criteria and design considerations documented within those reports.

2.1 Bridge Design Criteria

The relevant design specifications and guidelines that are the basis of the Enhanced Seismic Retrofit Alternative can be found in the *EQRB Bridge Design Criteria* (Multnomah County 2021) (Appendix A). The criteria provide design loading and specific clearance requirements and considerations for the enhanced retrofit alternatives being studied during the NEPA phase. The following unique loading criteria have been taken into consideration:

- Removal of load restrictions across the Burnside Bridge by including emergency vehicles into the design criteria
- Able to accommodate Portland Streetcar

2.2 Seismic Design Criteria

The relevant seismic design and guidelines that are the basis of the Bridge Enhanced Retrofit Alternative can be found in the *EQRB Seismic Design Criteria* (Multnomah County 2021) (Appendix A). The design criteria identify the minimum requirements for seismic design for the NEPA-phase design assessment.

Seismic performance goals defined for this Project are as follows:

Full Operation – Damage sustained is negligible. Only minimal, superficial repairs and maintenance activities will be required post-earthquake without interruption to traffic. All traffic modes are able to use the bridge, immediately after the earthquake. Full operation of the movable span will be possible within weeks of the CSZ seismic event.

Limited Operation – Damage sustained is minimal. The bridge allows for emergency vehicles (after inspection and removal of debris). Movable components may not be operable without repairs. Damage is repairable but may have short-term traffic impacts.

2.3 Roadway Geometry

Roadway design standards are developed to support safety and mobility goals. Roadway deficiencies have a critical impact on the safe and efficient use of the road by all travelers. The deficiencies of the existing Burnside Bridge and approach roadway have been identified in the *EQRB Existing Roadway Deficiency Memo* (Multnomah County 2021) (Appendix A). The proposed roadway geometrics defined in the *EQRB Facilities Standards List* (Multnomah County 2021) (Appendix A) by using applicable AASHTO,

Oregon Department of Transportation (ODOT), and County design standards are primarily for the replacement alternatives. The roadway improvements for the Enhanced Retrofit Alternative (Appendix C) are restricted by the existing bridge.

2.4 Geotechnical Conditions

The results of the geotechnical research, field explorations, laboratory testing, analyses, and design recommendations for the Enhanced Seismic Retrofit Alternative can be found in the *EQRB Geotechnical Report* (Multnomah County 2021) (Appendix A). Geotechnical analyses and recommendations presented in that report expand on the preliminary geotechnical work performed during the EQRB Feasibility Study. Foundation recommendations, as well as seismic hazard mitigation, have been identified for the Enhanced Retrofit Alternative. These recommendations have also been discussed and summarized in Section 7 below.

2.5 Multimodal/Transit Considerations

As a part of the preparation of the EIS for the Project, the *EQRB Transportation Technical Report* (Multnomah County 2021) was prepared to identify and evaluate transportation within the in the Project's Area of Potential Impact (API). Transportation modes evaluated are automobiles, bus, light rail, streetcar, freight, bicycles, and pedestrians. Direct effects caused by proposed alternatives were evaluated within the direct impact area, whereas the indirect impact area was used to evaluate broader transportation implications for all modes during construction.

2.6 Navigation Clearances

2.6.1 Navigation Clearance

The commercial, recreational, and government vessel traffic that transit the Willamette River under the Burnside Bridge has been summarized in the *EQRB Preliminary Navigation Study* (Multnomah County 2021) (Appendix A). River user impacts, if any, have been identified for the Enhanced Seismic Retrofit Alternative. Furthermore, elevation and horizontal clearance requirements are discussed; these have been identified as elevation 167.1 (NAVD 88), 147-foot vertical clearance (above ordinary high water elevation 20.1) and 205-foot-wide horizontal clearance. Ultimately, the USCG requirement is to enable 100 percent of vessel traffic to safely transit the bridge.

2.6.2 Bascule Span Open and Close

The existing bridge bascule span can only open to approximately 55 degrees from horizontal because of restrictions from previous repair work. One of the objectives of the enhanced retrofit is to restore the bascule span leaf opening angle to the original design angle: 73 degrees.

2.7 Railroad Considerations

The Project site is located over UPRR tracks. At the time of this report, railroad coordination and input has not been initiated. Once coordination begins, items to discuss include, but are not limited to:

- Temporary access to facilitate demolition of the existing bridge adjacent to and over the UPRR tracks.
- Temporary track crossings to facilitate construction of the proposed enhanced retrofit.
- UPRR flagging requirements and third-party inspector and Project site.

2.8 Right-of-Way

Per preliminary right-of-way (ROW) investigations, it has been determined that in addition to the County's current easements and resolutions, additional ROW acquisitions are anticipated from parcels on both the west and east approaches of the proposed bridge retrofit alternative. Additionally, temporary construction easements would need to be secured to construct the Enhanced Seismic Retrofit Alternative and roadway improvements. As the design for the Project progresses, HDR will work closely with the County to determine the extents of the permanent and temporary ROW needs. Preliminary ROW impact maps have been identified and detailed within the *EQRB Right-of-Way Technical Report* (Multnomah County 2021).

2.9 Utilities

Reasonable attempts have been made to avoid utility infrastructure with the Enhanced Seismic Retrofit Alternative layout where practical. Foundation elements have been located to avoid the large-diameter CSO pipes. Smaller utilities that are near the surface have been avoided where practical, but some temporary utility relocations may be required.

Expected temporary impacts include:

- Temporary relocation of sewer lines running along the sea wall behind and adjacent to the existing Pier 1.
- Temporary disruption to TriMet's overhead catenary lines attached to existing Bent 3.
- Abandonment or temporary relocation of all other utilities directly attached to the existing bridge structure.

For further discussion about these impacts and their need, see the *EQRB Construction Approach Technical Report* (Multnomah County 2021) and the *EQRB Utilities Technical Report* (Multnomah County 2021).

2.10 Hydraulic Considerations

At the time of this report, a design hydraulic study has not been conducted. Preliminary analysis and water surface elevations will need to be determined for the design flood events. Freeboard for the proposed structure will need to meet Federal Highway Administration and ODOT criteria for both the 50-year and 100-year flood events. Analysis will be done to determine the preferred alternatives' impact on the base flood elevation. The Project is expected to have only minor flood elevation increases for the final condition, though temporary conditions during construction may have impacts that would require mitigation. If the enhanced retrofitted bridge contributes to a net increase in the 100-year base flood elevation, the Project may require conveyance offsets or may request revision to the base flood elevation to accommodate the post-retrofit conditions. A Letter, or Conditional Letter, of Map Revision (LOMR/CLOMR) would be required for Federal Emergency Management Agency flood insurance maps.

2.11 Constructability

The anticipated approach to construct the Enhanced Seismic Retrofit Alternative can be found in the *EQRB Construction Approach Technical Report* (Multnomah County 2021) (Appendix A). The purpose of this report is to identify the potential phasing and staged construction considerations for the duration of retrofit construction. Project-specific construction activities have been investigated for the Enhanced Seismic Retrofit Alternative being studied for the EIS.

2.12 Aesthetics and Urban Design

Although not specifically identified at the time of this report, it is anticipated that architectural aesthetics for this Project will be of significant importance. Additionally, design features that would fit the urban context will be developed. As the design for this Project progresses, HDR will work closely with the County to define the extents of the aesthetic and urban design needs and incorporate them into the design of the Project.

3 Existing Site Conditions

3.1 Geotechnical Conditions

To support this EQRB Environmental Impact Study (NEPA), Shannon & Wilson prepared the *EQRB Geotechnical Report* (Multnomah County 2021) that is part of Appendix A of this document.

The report provides geotechnical information and recommendations as follows:

- Project Area subsurface conditions that include geotechnical soil and groundwater conditions
- Seismic ground motions and hazard evaluations, including the recommended seismic design ground motions
- Existing foundation resistance and stiffness
- Conceptual seismic mitigation ground improvement design
- Foundation resistance for post-retrofit conditions

The following geotechnical report figures are of particular interest:

- Figure 4 – The Project Area subsurface profile
- Figures 5 to 8 – Recommended response spectra
- Figure 9 – Post-retrofit Project Area subsurface profile

The following geotechnical report exhibits are of particular interest:

- Exhibit 8-3 – Estimated Liquefaction-Induced Settlement at Existing Spread Footing Foundations
- Exhibit 8-4 – Estimated Liquefaction-Induced Settlement at Existing Pile Group Foundations
- Exhibit 9-1 – Recommended Unfactored Static and Seismic Soil Parameters for Existing Spread Footings and Pile Caps
- Exhibit 9-2 – Recommended Nominal Static and Seismic Axial and Uplift Resistance for Existing Piles
- Exhibit 11-1 – Summary of Spread Footing Foundations for Preferred Retrofit and Seismic Mitigation Alternative
- Exhibit 11-2 – Recommended Unfactored Post-Seismic/Reduced Strength Soil Parameters for Spread Footings and Pile Caps for Preferred Retrofit and Seismic Mitigation Alternative
- Exhibit 11-3 – Summary of Drilled Shaft Group Foundations and Estimated Downdrag Loads for Preferred Retrofit and Seismic Mitigation Alternative

3.2 Utilities

The utilities found underground and on the Burnside Bridge structure are generally described below. For details, see the *EQRB Utilities Technical Report* (Multnomah County 2021) prepared by HDR and Casso Consulting.

3.2.1 West Approach

The westside utilities include multiple pipes under the streets and in the areas between the streets. The underground pipes accommodate telecommunication, natural gas, electricity, water, sewer, and foul air in structures constructed from clay, ductile iron, PVC, and conduit. Typical pipe sizes range from 1 inch in diameter up to 60 inches for the City of Portland CSO. Of particular note is the 168-inch City of Portland CSO line located between Bents 17 and 18. Constructability-related challenges with utilities near the Ankeny Pump Station are noted in Section 12. CenturyLink has four 3-inch-diameter conduits attached to the west approach to Pier 2. The west approach bridge structure also carries various overhead conduits and utilities for the TriMet MAX line including the train overhead catenary lines attached to Bent 3.

3.2.2 Bascule Spans

At Pier 2, the CenturyLink conduits carried from the west approach become a 6-inch submarine conduit that crosses between Pier 2 and Pier 3 in the Willamette River. The conduit is attached to Pier 3 where it comes out of the water to continue east on the east approach.

3.2.3 East Approach

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

3.3 Waterway Navigation Channel

The vessel navigation channel of the Willamette River is under the bridge's bascule span. More detailed descriptions of the navigational channel can be found in the EQRB Preliminary Navigation Study that is included as part of the Appendix A of this document.

3.4 Adjacent Facilities

3.4.1 Building Adjacent to West Approach Spans

The locations and proximity of adjacent buildings can be seen in Figure 3-1.

Figure 3-1. Private Building Locations



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

On the south side of the west approach spans and retaining walls, the Portland Saturday Market occupies space under Span 1. The market sets up a large number of booths under the bridge on weekends, March 1 through December 31. The Salvation Army also occupies a building immediately adjacent to the approach retaining walls on the south side.

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

3.4.2 Water Facility at Pier 1

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

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

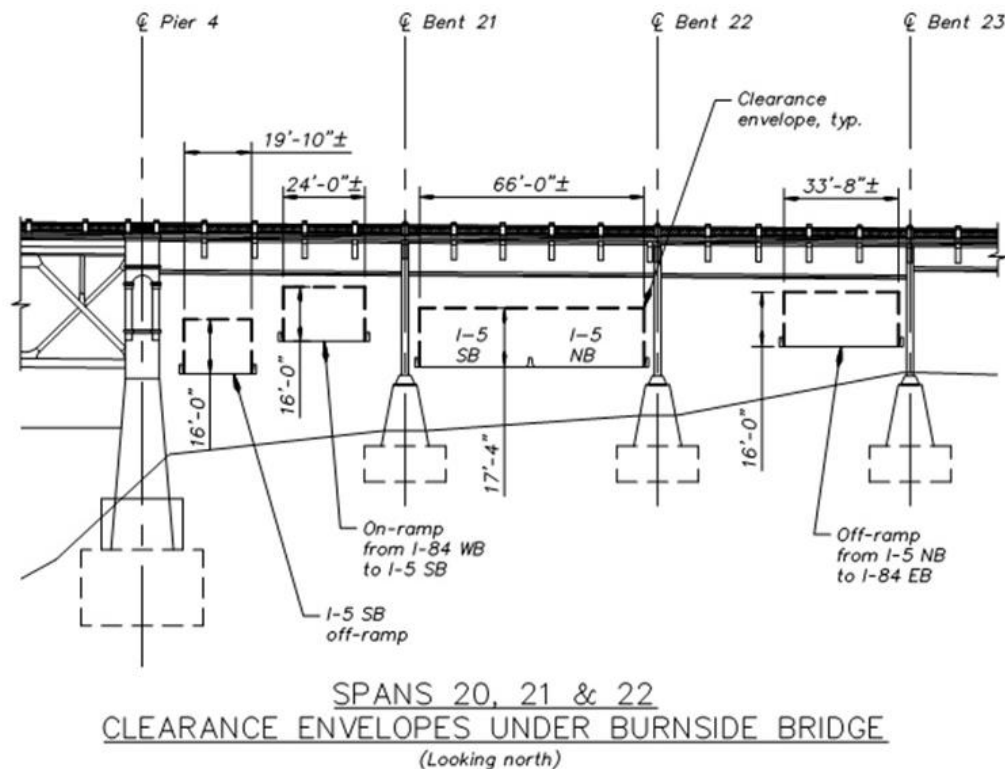
On the north side of the bridge, within Waterfront Park adjacent to Bent 19, BES has two below-grade odor-control vaults. The 19-foot by 19-foot vault contains mechanical equipment, and the 25-foot by 26-foot vault contains media for air treatment. Foul air from the Ankeny wet well and Ankeny shaft is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

The seawall is recessed into Waterfront Park on the west side of Pier 1 (see Figure 3-7). Two sewer force mains running north from the Ankeny Pump Station (one 30-inch and one 42-inch) are attached to the exposed side of the seawall adjacent to Pier 1. The force mains are stacked above each other and follow the seawall recess, turning on the north side of Pier 1, then following the seawall to the north before crossing under the river to the east.

3.4.3 Highway Ramps under East Approach Spans

I-5 and associated ramps pass under existing Spans 20 to 22 can be seen in Figure 3-2. The interstate and ramps are all bridges that were built after the Burnside Bridge with foundations on either side of the existing 86-foot Burnside Bridge width at this location. The structures are within inches of the existing bridge bents, including the I-5 southbound bridge and its on-ramp from I-84 to both sides of existing Bent 21, the I-5 northbound bridge to the west side of existing Bent 22, and the I-5 northbound off-ramp to I-84 to the west side of existing Bent 23.

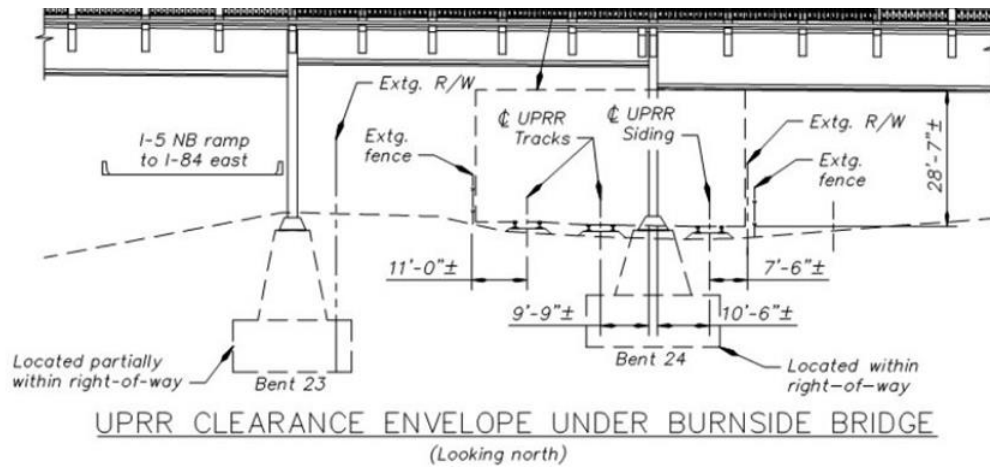
Figure 3-2. ODOT Highway Clearances



3.4.4 Railway Lines under East Approach Spans

UPRR main lines and a railroad spur line pass under existing Spans 23 and 24 and can be seen in Figure 3-3. The main lines pass to the west side of existing Bent 24, while the railroad spur line, which does not appear to be in use any longer, passes to the east side existing Bent 24.

Figure 3-3. Union Pacific Railroad Clearances



3.4.5 TriMet Light Rail under West Approach Spans

The TriMet MAX Red and Blue lines pass under the west approach in Spans 3 and 4, and the Skidmore Fountain station is located under the bridge. The overhead catenary system used to electrify the lines is currently supported by the bridge structure. TriMet light rail clearances are shown in Figure 3-4 and Figure 3-5.

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

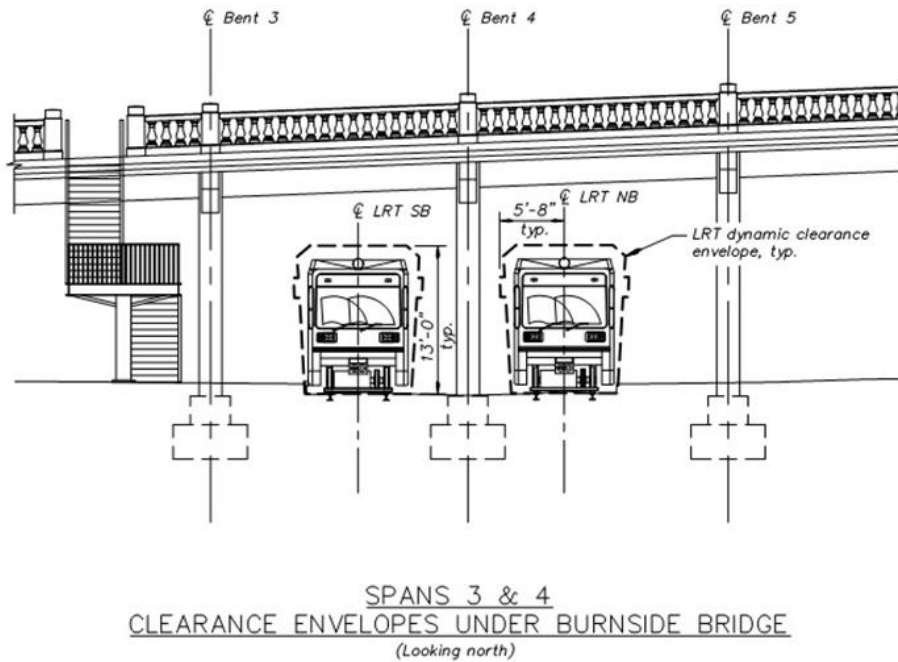
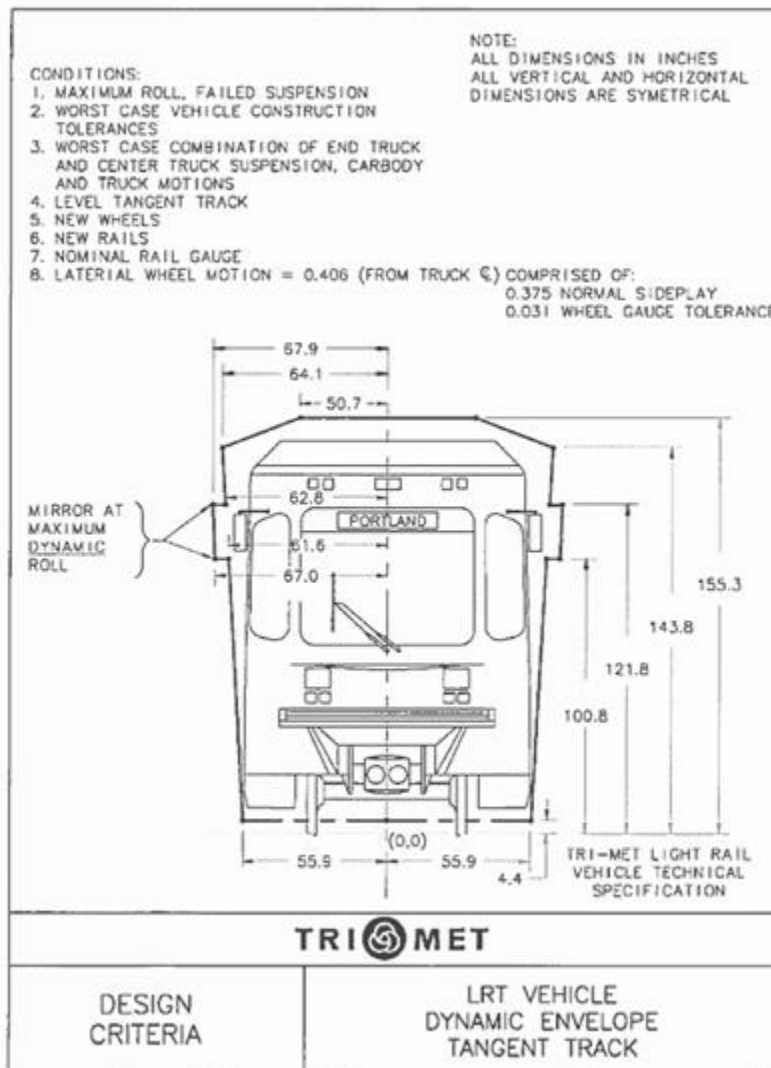


Figure 3-5. TriMet Light Rail Vehicle Dynamic Envelope Tangent Track



3.4.6 City of Portland Facilities

Naito Parkway passes under the west approach in Spans 14 and 15, and the Waterfront Park Trail passes under Span 19. Waterfront Park, which houses many community events, extends under the west approach Spans 17 through 19. NW/SW 2nd Avenue passes under the east approach in Span 26, and NW/SW 3rd Avenue passes under Span 33. City of Portland facility clearances are shown in Figure 3-6, Figure 3-7, Figure 3-8, and Figure 3-9.

Figure 3-6. Clearance Envelopes under the Burnside Bridge – Spans 14 and 15

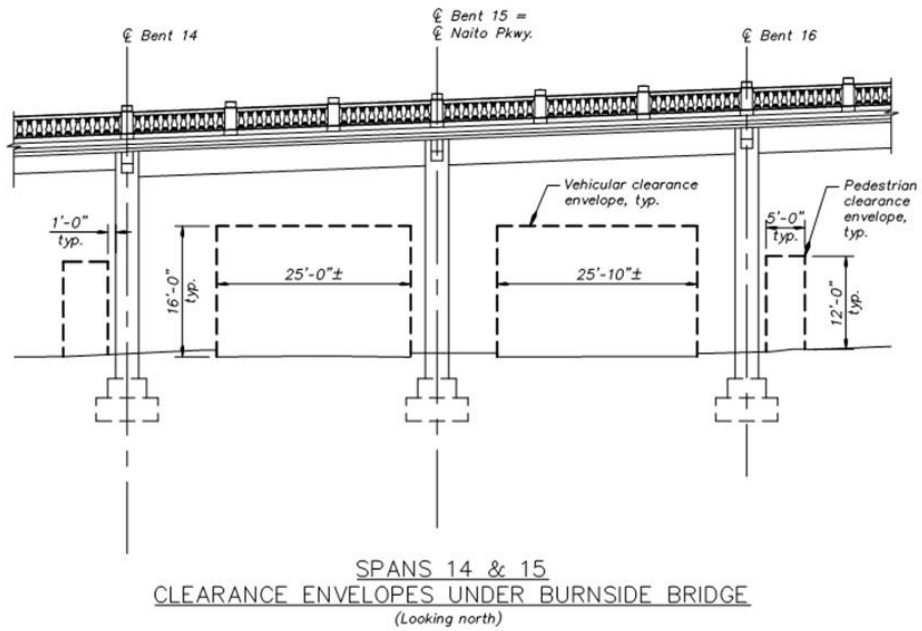


Figure 3-7. Clearance Envelope under the Burnside Bridge – Span 19

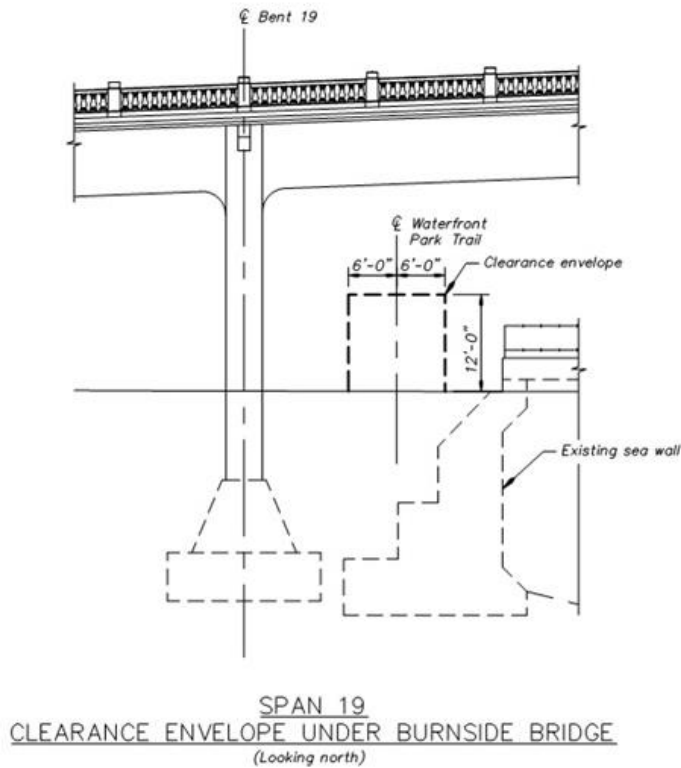
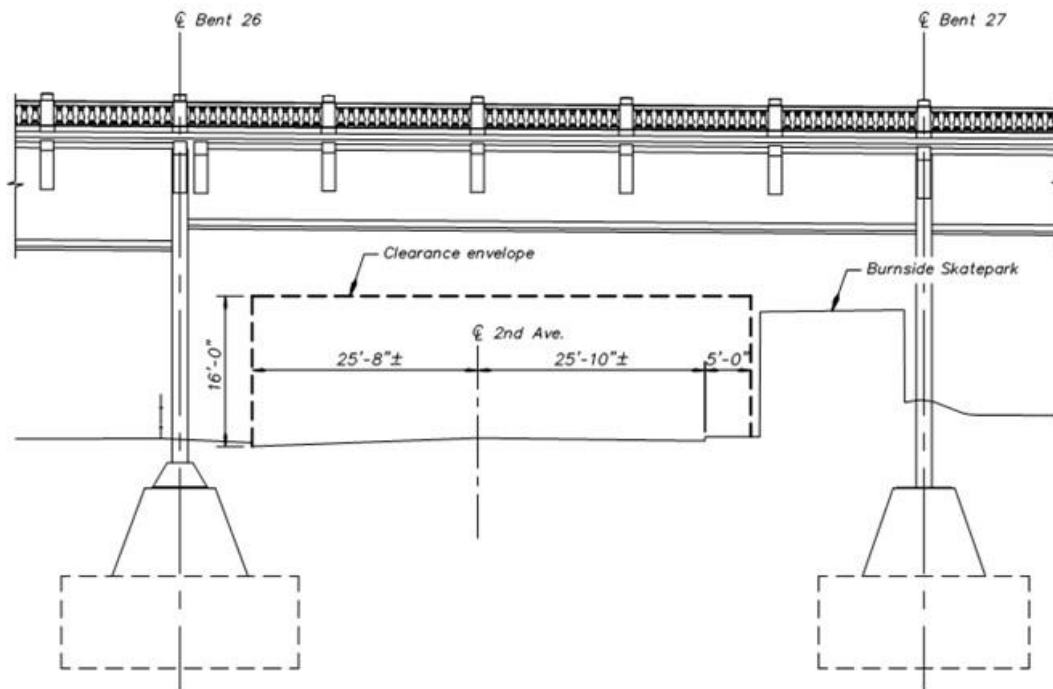
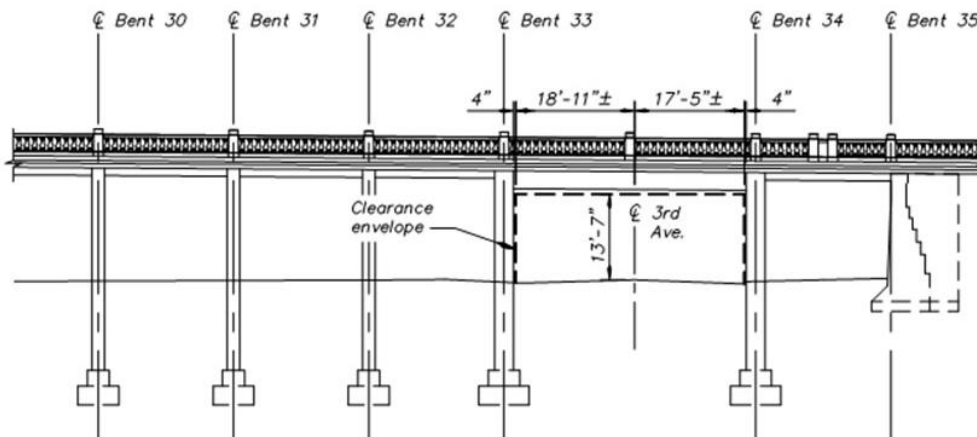


Figure 3-8. Clearance Envelope under the Burnside Bridge – Span 26



SPAN 26
CLEARANCE ENVELOPE UNDER BURNSIDE BRIDGE
 (Looking north)

Figure 3-9. Clearance Envelopes under the Burnside Bridge – Spans 30, 31, 32, and 34



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

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4 Structural Analysis Methodology

4.1 Design and Analysis Methodologies

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

- Review various as-built plans and previous rehabilitation/retrofit plans.
- Review previous bridge rehabilitation and retrofit study memos and reports.
- Identify seismic vulnerabilities based on engineering judgement and analysis results.
- Perform conceptual-level analysis that includes hand calculations, spreadsheets, and dynamic model analysis to support the identification of seismic vulnerabilities.
- Develop enhanced bridge seismic retrofit concepts according to design code requirements.
- Perform dynamic model analysis to support the development of enhanced retrofit schemes. Engineering judgement was applied for selecting representative structural elements for analysis. For example, instead of analyzing every stringer and floor beam, selected stringers and floor beams were analyzed for the purpose of conceptual engineering.

The enhanced retrofits include the structural rehabilitation of the existing structures to meet the current load rating requirements of AASHTO, Multnomah County, and ODOT.

The bridge structure was analyzed using the finite element software SAP2000, version 18.1 and the multimodal spectral method to determine force and displacement demands on the critical elements of the structure. Capacities for the critical elements were developed in accordance with the *EQRB Seismic Design Criteria* (Multnomah County 2021) (Appendix A) and compared to the analysis demands. Capacity-to-demand (C/D) ratios were developed where data were available. A C/D ratio less than one indicates a structural deficiency.

4.2 Analysis Models

4.2.1 Existing Structure Models

Models for the existing structure were developed for the west approach, east approach, and main river spans. Roadway slabs were modeled with a 0.5-inch sacrificial wearing surface, assigned to the roadway as additional mass and dead load.

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

The existing structure deficiency analysis was based on the acceleration response spectra and soil property recommendations provided in the *Draft Geotechnical Report – Burnside Bridge Seismic Feasibility Study* dated September 13, 2017. The site-specific acceleration response spectra and soil properties were subsequently further refined and updated by the geotechnical engineers and reported in the *EQRB Geotechnical Report* (Multnomah County 2021) (Appendix A). However, based on comparisons and engineering assessment, the identified structural deficiencies are still valid; thus, no re-analysis was necessary for the purpose of identifying structural deficiencies.

For concrete columns in the pre-retrofit analysis, gross section properties were used to determine CSZ event demands, and cracked section properties were used to determine 1000-year event demands. Consideration was given to using cracked sections for the CSZ event in an attempt to improve the C/D ratios for that event, but since C/D ratios for column displacement in the 1000-year event were also low, indicating that column retrofit was required, it was deemed not necessary to analyze the CSZ event with cracked column sections.

4.2.2 Post-Retrofit Structure Model

A model that includes west and east approach structures and the main river spans was developed for post-retrofit analysis. Boundary conditions at the expansion joints were defined according to the retrofit schemes. Conceptual seismic retrofits were developed and initially sized based on engineering experience and followed up with computer model analysis to verify the retrofit could be reasonably expected to resolve the deficiencies in final design. The retrofit concepts were updated as needed according to the post-retrofit analysis results.

The post-retrofit analysis was based on the updated acceleration response spectra and soil parameters provided in the *EQRB Geotechnical Report* (Multnomah County 2021) (Appendix A).

For post-retrofit analysis, live load applications were defined according to the *EQRB Bridge Design Criteria* (Multnomah County 2021) and *EQRB Seismic Design Criteria* (Multnomah County 2021) (Appendix A).

The Enhanced Retrofit Alternative includes the bridge live load capacity upgrade to meet Project requirements. Load rating analyses were performed to identify the member rehabilitation and strengthening requirements.

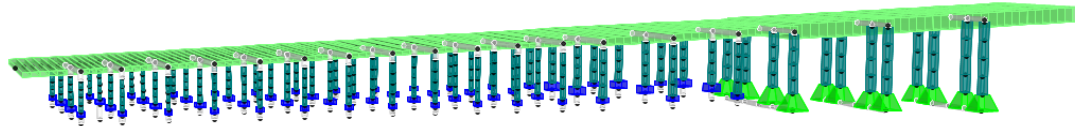
For concrete columns in the post-retrofit analysis, section properties were developed according to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (AASHTO 2012).

4.2.3 Approach Spans

Boundary conditions for the analysis models are described below.

4.2.3.1 West Approach

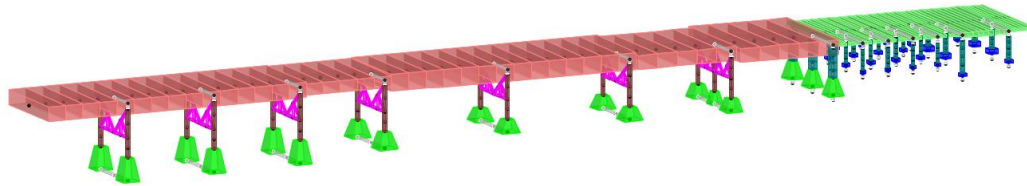
Figure 4-1. West Approach Analysis Model



- Expansion joints at Bents 1, 5, 8, 11, 14, 16, 17, 18, and 19 were modeled as closed (seismic restrainers were installed during Phase 1 seismic retrofit in 2001) with superstructure moment releases at the following eight locations:
 1. Span 4 at Bent 5
 2. Span 7 at Bent 8
 3. Span 10 at Bent 11
 4. Span 13 at Bent 14
 5. Span 15 at Bent 16
 6. Span 17 at Bent 17
 7. Span 18 at Bent 18
 8. Span 19 at Bent 19
- Seismic demands from elastic analysis may be limited by overstrength of the columns.
- For the reinforced concrete deck girder spans, average section properties were developed to be representative of the average girder spacing and deck width for that span. The cantilevered sidewalk and fascia beam were treated as dead weight only and did not contribute to the superstructure section properties.
- Column heights were defined from bottom of end floor beam to top of pile cap or spread footings. As-built column heights were used at all locations.
- East end of Span 19 is fixed horizontally on top of Pier 1 as a pin connection. The effects from the steel truss span were modeled as a longitudinal roller support on top of Pier 1.

4.2.3.2 East Approach

Figure 4-2. Existing East Approach Analysis Model



- Moments were released at the top of all columns at Bent 28 to capture the behavior of the pinned bars at Bent 28.
- Expansion joints at Bents 22, 24, 26, 31, 33, and 34 were modeled as closed (seismic restrainers were installed during Phase 1 seismic retrofit in 2001) with superstructure moment releases at the following six locations:
 1. Span 21 at Bent 22
 2. Span 23 at Bent 24
 3. Span 25 at Bent 26
 4. Span 30 at Bent 31
 5. Span 32 at Bent 33
 6. Span 34 at Bent 34
- Seismic demands from elastic analysis may be limited by overstrength of the columns.
- For the plate girder spans, average section properties were developed to be representative of the plate sizes, girder spacing, and deck width for that span. The deck was considered to be composite. The cantilevered sidewalk and fascia beam were treated as dead weight only and did not contribute to the superstructure section properties.
- For the reinforced concrete deck girder spans, average section properties were developed to be representative of the average girder spacing and deck width for that span. The cantilevered sidewalk and fascia beam were treated as dead weight only and did not contribute to the superstructure section properties.
- Column heights were defined from bottom of end floor beam to top of pile cap or spread footings. As-built column heights were used at all locations.
- The concrete encasement of Spans 20 to 27 and Bents 21 to 27 was treated as dead load only and did not contribute to the section properties of the steel elements.
- The west end of Span 20 is fixed on top of Pier 4. The effects from the steel truss span are modeled as a set of springs on top of the Pier 4.

4.2.3.3 East Approach Span Replacement

Because of the site constructability restrictions, the seismic retrofit requires replacing the existing Bents 21 to 24, including the superstructure supported by these bents, with a new three-span continuous structure, so the post-retrofit east approach would have the following span layout:

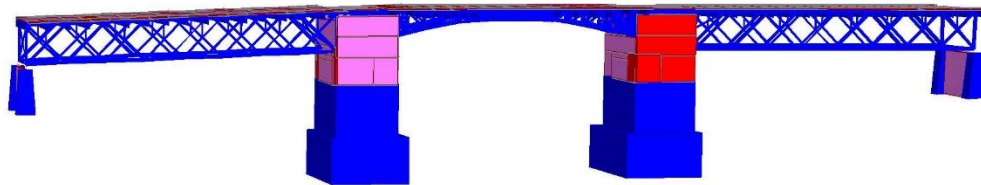
- Bent 23 – New replacement bent that supports the east end of the existing steel truss span and the west end of the replacement spans
- Bent 24 – New replacement bent that supports the new replacement spans
- Bent 25 to Bent 34 and east abutment are retrofitted bents

4.2.4 Main River Spans

As previously stated, seismic demand analysis was performed on a continuous three-dimensional SAP (Version 18.1) analysis model for the fixed steel truss spans, bascule spans, and Piers 1 to 4 (see Figure 4-3).

For post-retrofit analysis, the three-span model for the in-river spans was integrated with the west and east approach models, with boundary conditions defined according to the retrofit schemes.

Figure 4-3. Main River Span Analysis Model



- Steel trusses were modeled as two-force members with moments released at both ends, for both axes, except where noted.
- The two-force members were modeled with the gross area noted in the plans dated December 22, 1923 (Bascule Spans) (Multnomah County 1923), and February 5, 1924 (Fixed Spans) (Multnomah County 1924). Where gross area was not available, it was calculated using dimensions from shop drawings.
- Flexural members (brackets, floor beams, stringers, beginning bascule members, end bascule members, trunnion posts, trunnion struts, and live load supports) were first modeled in AutoCAD to determine relevant section properties. These members were then defined in SAP as "generic members" with the section properties input manually.
- All slabs, sidewalks, and walls were input as "thin shell" elements with the appropriate as-built thicknesses, at the structural center of gravity of the element.
- Columns were modeled as continuous frame elements.

- Slabs and sidewalks were modeled without structural stiffness in the "dead load" model.
- Slabs in the composite models, walls, and columns were modeled with 50 percent structural stiffness to approximate cracked conditions.
- Structural systems were connected using rigid links (slab to truss, truss to piers, etc.).
- Relatively stiff support elements (buttresses, bearings, and pedestals) were modeled as links, with weights and masses assigned to node points.
- Piers 2 and 3 used body constraints at the bottoms of walls and tops of columns to simulate relative stiffness of the structures.
- The footings of Piers 1 and 4 were modeled with links from the bottoms of columns to tops of piles.
- Non-structural elements (control towers, rails, sidewalk and roadway stringers, etc.) were added to the model and assigned to the nearest structural elements as loads and masses.
- Pit deck stringer longitudinal supports were modeled according to the as-built plans available that include 2005 Main Span Rehabilitation (#00511), drawings 70418, 70420, and 76211; and in the 1924 plans, sheets T-34, S-7 and S-18. End restraints were modeled as expansion joints where connected to the fixed span and as fully supported where connected to the bascule span.

4.2.4.1 Fixed Spans

- Fixed spans were connected to piers with "roller" type links on the shore side (Piers 1 and 4), and standard links on the river side (Piers 2 and 3).
- Self-load was determined using the 15 kip per linear foot truss weight estimate on page T30 of the February 1924 design drawings (Multnomah County 1924). These loads were verified by comparing the load effects calculated in the "original configuration" SAP model with the loads originally shown on page T30.
- Sway bracing on the shore side was noted in the inspection reports but was not detailed in the design documents. Bracing was assumed to be similar to adjacent bracing and was included in the model.
- Roadway slab height and thickness were adjusted based on ODOT rehabilitation drawings dated September 2001 (Multnomah County 2001).

4.2.4.2 Bascule Spans

- Self-load was calculated using "lifting load" estimates from the shop drawings. These loads were verified by comparing the load effects calculated in the "original configuration" SAP model with the loads originally shown on page S2 of the December 1923 design drawings (Multnomah County 1923).

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

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

4.3 Boundary Conditions

The west approach and east approach models used soil parameters, provided by the geotechnical engineers, for vertical and horizontal displacements at the bottoms of all columns. After a sensitivity study of foundation springs, rotations in all directions of the column bases were set as “fixed.” Where the approach structures meet the main river spans (Piers 1 and 4), models used springs for displacement and rotations along and about the horizontal axes to approximate the stiffness of the main spans for the as-built structure analysis. Vertical displacements and rotations about the vertical axis were fixed. Column embedment depth effects during seismic events shall be considered during the detailed design when the soil information is available. At the abutments, models used springs to approximate the stiffness of the soil behind the abutment.

All the main span and bascule piers (Piers 1, 2, 3, and 4) were modeled as fixed at the tops of piles for dead load models. For seismic models, rotation was assumed to be

relatively fixed, while lateral movement was restrained by springs determined by matching maximum seismic displacements with the load-displacement graphs provided by the geotechnical engineers. Iterations were used for each lateral spring. Vertical springs were determined by matching dead load reactions with load-displacement graphs.

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

5 Existing Structure and Seismic Vulnerabilities and Deficiencies

Burnside Bridge Seismic Deficiency Plans are attached as Appendix B for reference.

5.1 West Approach Structures

The west approach is 602 feet long and consists of 19 spans: Span 1 through Span 19. It crosses over City of Portland streets, the TriMet MAX line, and the Governor Tom McCall Waterfront Park.

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

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

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

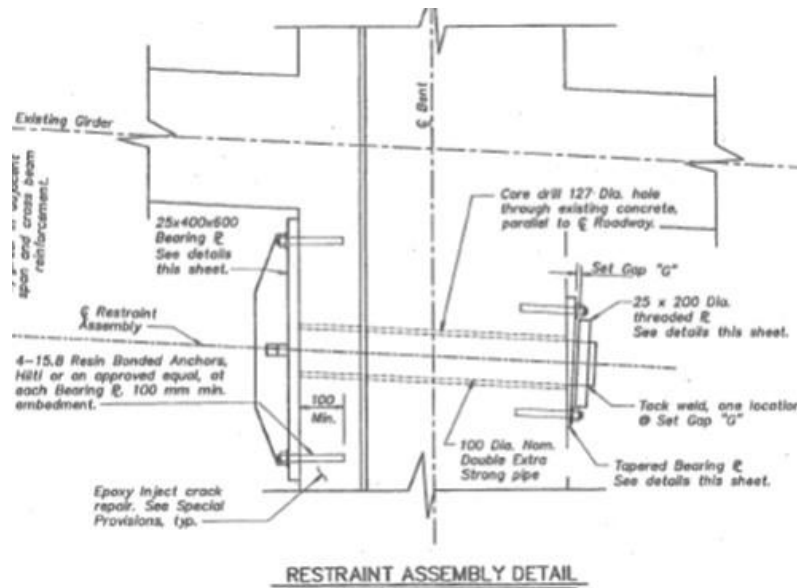
Spans 1 to 19 consist of reinforced concrete deck girder (RCDG) spans in two main configurations. Spans 1 to 13 consist of three- and four-span continuous units with constant-width RCDG spans framing into end floor beams at each bent. End floor beams are supported by four reinforced concrete columns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

As part of the enhanced retrofit analysis, load rating analysis performed on the existing structures revealed that the existing bridge deck on the west approach structures does not provide the required strength to meet the requirements of current load rating standards and the planned streetcars on this bridge.

The following seismic vulnerabilities were identified within the west approach structure:

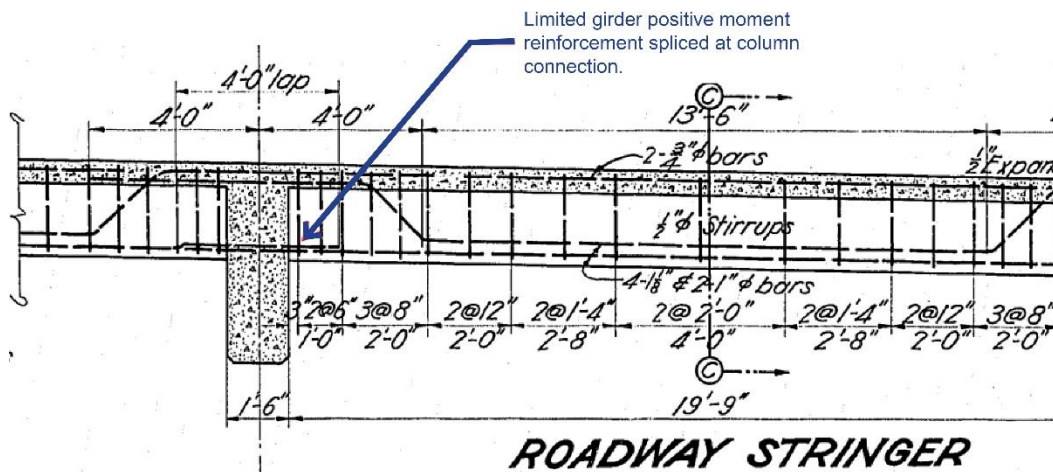
Seismic Restrainer – Insufficient strength. The existing restrainers at expansion bents are inadequate due to increased seismic loading demand requirements that have been developed in the years since the original Phase I seismic retrofit. Modification of the restrainers would also be required due to the floor-beam strengthening that is described below; therefore, analysis of the demands and capacities of the existing restrainers was not required (see Figure 5-1).

Figure 5-1. Seismic Restrainer Vulnerability



Superstructure Flexural or Shear Strength – Insufficient strength. The girder’s positive moment reinforcement is spliced at the column connection, limiting moment capacity due to inadequate development length. Analysis of the existing structure for the CSZ event shows the C/D ratios are less than 1.0 at multiple locations (see Figure 5-2). In addition, load rating analysis results indicated that the stringers do not have sufficient strength to meet the load rating requirements per current design standards.

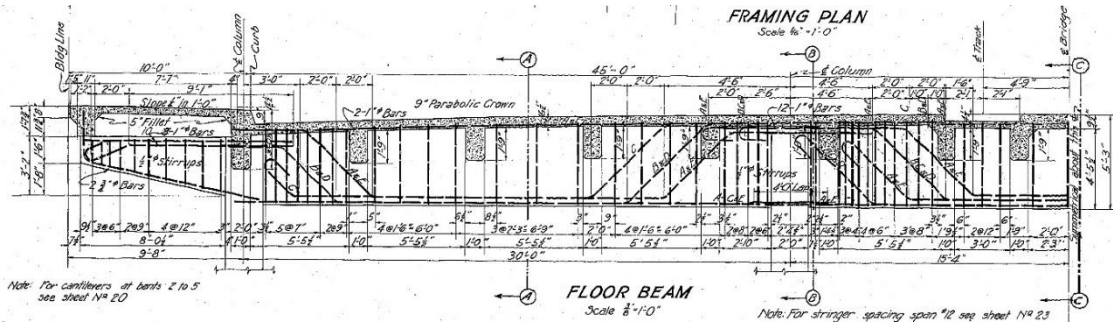
Figure 5-2. Superstructure Vulnerability



Floor-Beam Flexural or Shear Strength – Poor seismic detailing. The floor-beam positive moment reinforcement is spliced and/or has limited embedment at column connection, limiting its moment capacity due to inadequate development length. The

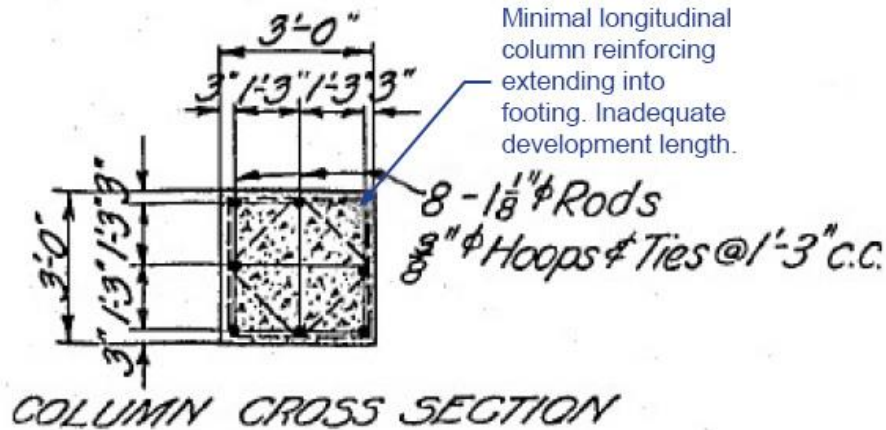
negative moment capacities at supports are inadequate to maintain elastic behavior for a design-level seismic event. Analysis of the existing structure for the CSZ event shows the C/D ratios at multiple locations are less than 1.0 (see Figure 5-3). In addition, load rating analysis results indicated that the floor beams do not have sufficient strength to meet the load rating requirements per current design standards.

Figure 5-3. Floor-Beam Vulnerability



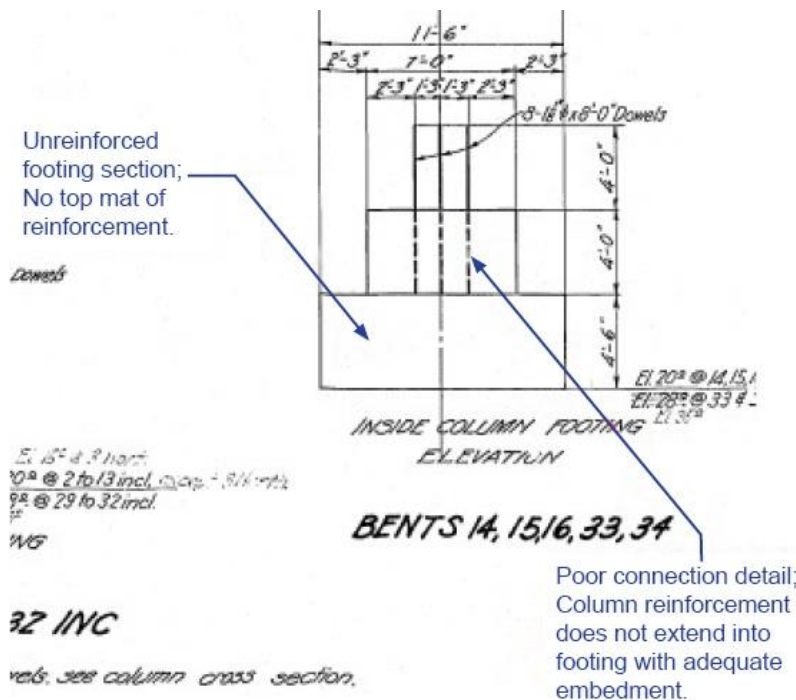
Column Flexural or Shear Strength – Insufficient strength. There is very little longitudinal column reinforcing extending into the footing, compounded by inadequate development length. There is poor confinement and a lack of seismic hooks, with ties and hoops at 1-foot 3-inch spacing. C/D ratios for column flexure and shear for the CSZ event are less than 1.0 (see Figure 5-4).

Figure 5-4. Column Vulnerability



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

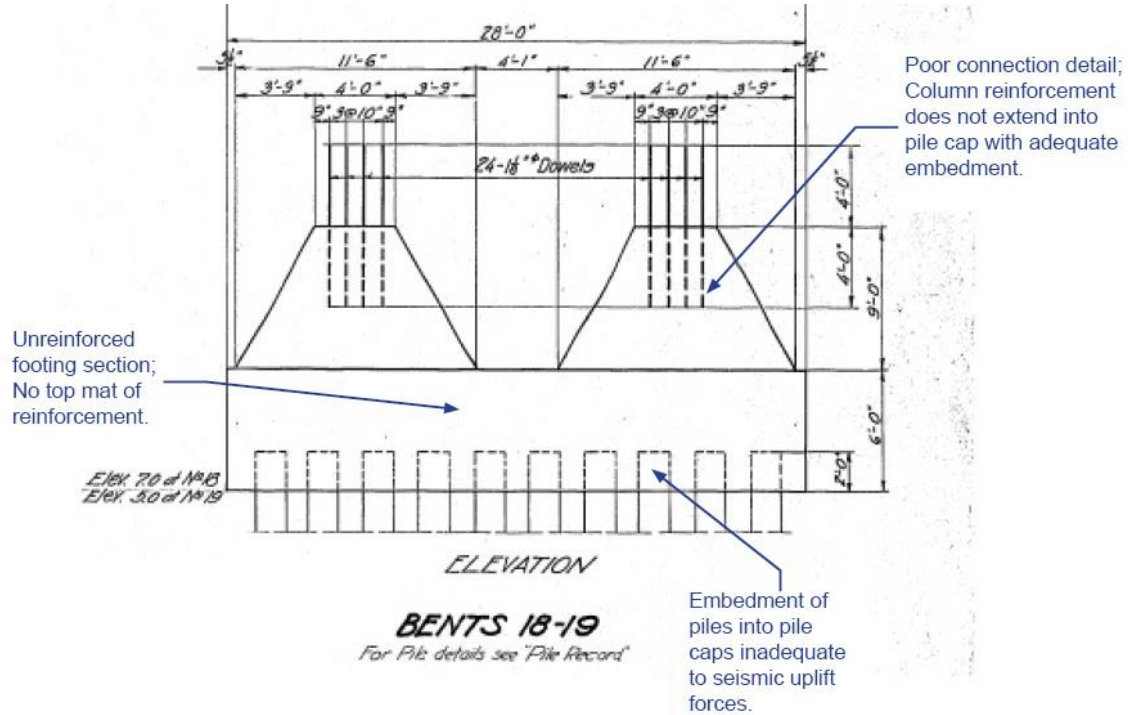
Figure 5-5. Spread Footing Vulnerability



Timber Pile Lateral Strength and Uplift Capacity – Insufficient pile strength. The pile group capacity is inadequate to limit settlement from liquefaction, resist uplift and downdrag forces, and resist displacements and forces from lateral spreading. The unreinforced footing section has no top mat of reinforcement. It has a poor connection detail as the column reinforcement does not extend into the pile cap with adequate embedment. Embedment of piles into pile caps is inadequate to resist seismic uplift forces. Due to the amount of liquefaction and lateral spread at Bents 17 through 19, it

was determined that foundation retrofit is required because a reasonable load path for seismic forces in these elements does not exist (see Figure 5-6).

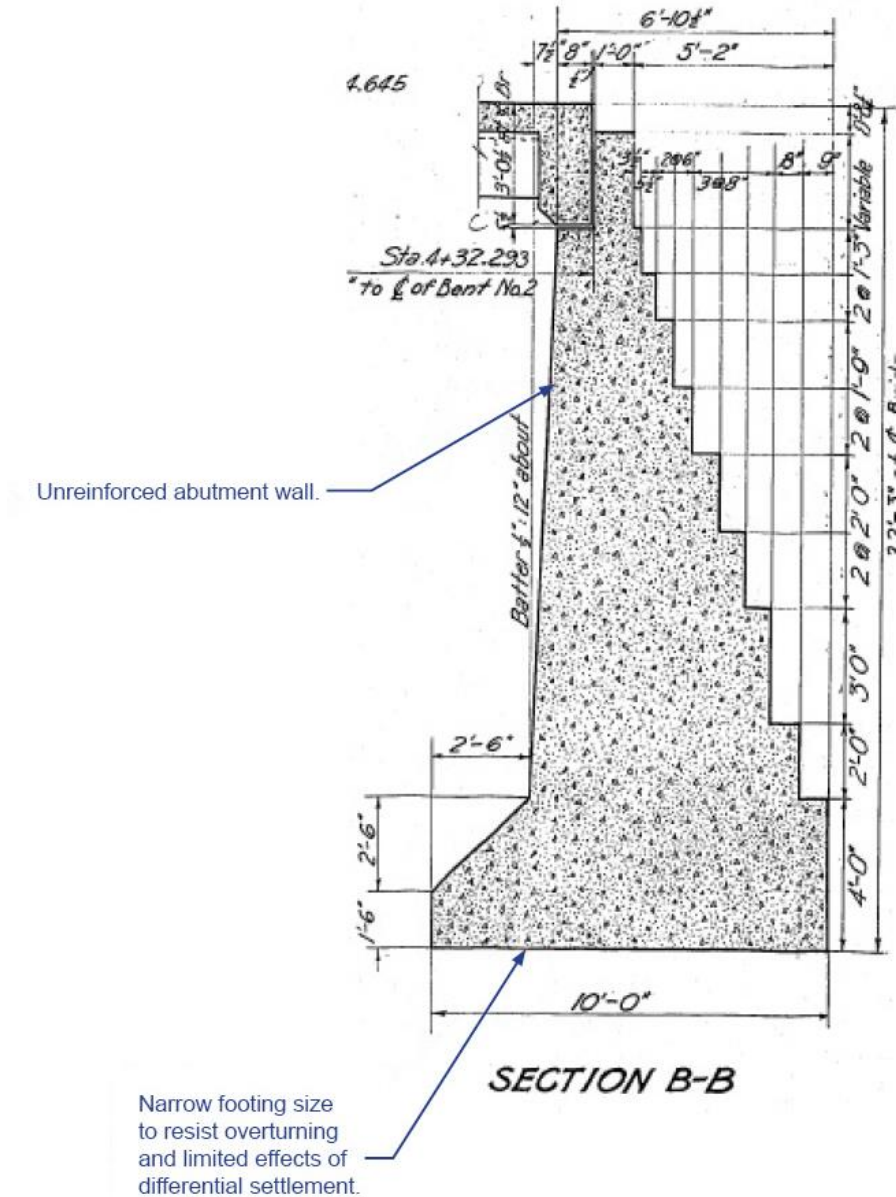
Figure 5-6. Pile Foundation Vulnerability (Bents 21 to 27 Similar)



Tall Abutment Retaining Wall Footing Size – Insufficient strength. The abutment’s narrow footing size is inadequate to resist overturning and limit effects of vertical and differential settlement from liquefaction to a desirable level. Additionally, the abutment wall is unreinforced. Due to the lack of reinforcing in the abutment and the liquefaction

effects, it was determined that a retrofit is required because a reasonable load path for seismic forces in these elements does not exist (see Figure 5-7).

Figure 5-7. Abutment Vulnerabilities



Liquefiable Soils and Lateral Spreading – Geotechnical hazards. Bents 1 to 16 are on spread footings with limited bearing capacity to resist overturning and liquefaction-induced settlement. Bents 17, 18, and 19 are on timber piles with limited capacity to resist liquefaction-induced settlement and lateral forces and displacements due to lateral spreading. Additional detail regarding the liquefaction and lateral spread can be found in the *EQRB Geotechnical Report* (Multnomah County 2021) (see Appendix A).

5.2 East Approach Structures

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

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

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

The east approach structures (existing Spans 20 to 34) consist of two configurations: (1) concrete-encased steel-plate girder spans (Spans 20 to 27) and (2) RCDG spans (Spans 28 to 34).

As part of the enhanced retrofit analysis, load rating analysis performed on the existing structures revealed that the existing bridge deck on the east approach structures does not provide the required strength to meet the requirements of current load rating standards and the planned streetcars on this bridge.

5.2.1 East Approach Existing Spans 20 to 27

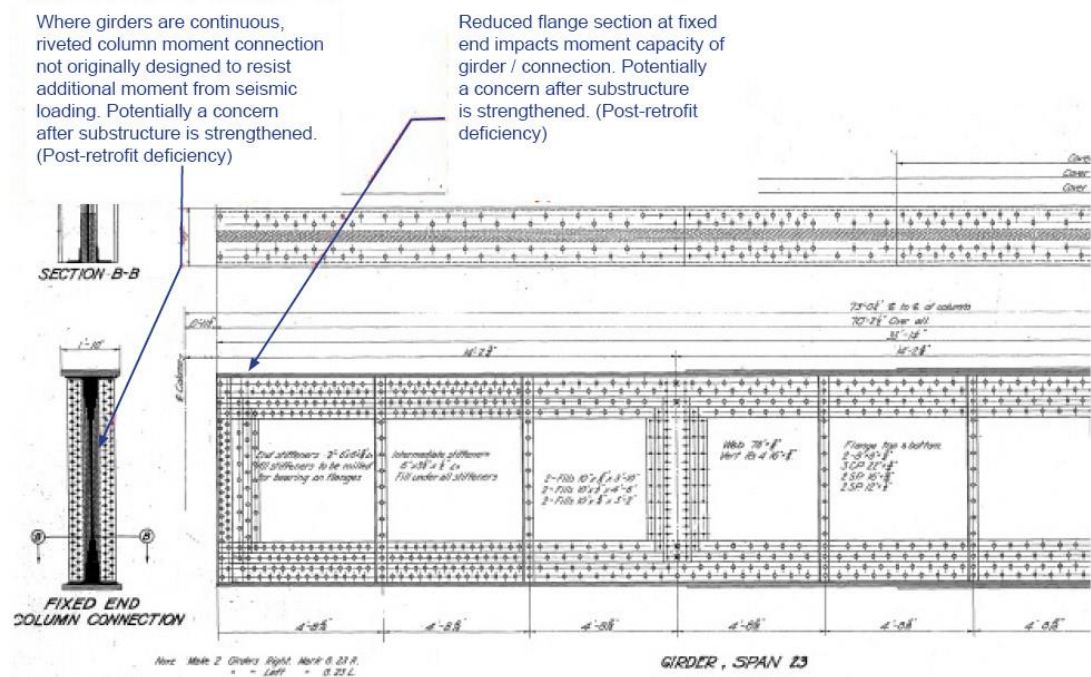
Existing Spans 20 to 27 are two-span continuous units of deep steel-plate girders encased in concrete. The plate girders support the concrete-encased steel floor beams, the reinforced concrete stringers, and the concrete deck. The floor beams are supported on concrete-encased steel bents with diagonal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

The following seismic vulnerabilities were identified within the east approach steel girder spans (Spans 20 to 27).

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

Superstructure Flexural or Shear Strength – The strength of the superstructure is a concern. Where girders are continuous, the riveted column moment connection was not originally designed to resist additional moment from seismic loading. The reduced flange section at the fixed end impacts the moment capacity of the girder/floor-beam connection. Analysis of the existing structure for the CSZ event resulted in a minimum C/D ratio less than 1.0 (see Figure 5-8). In addition, load rating analysis results indicated that the stringers do not have sufficient strength to meet the load rating requirements per current design standards.

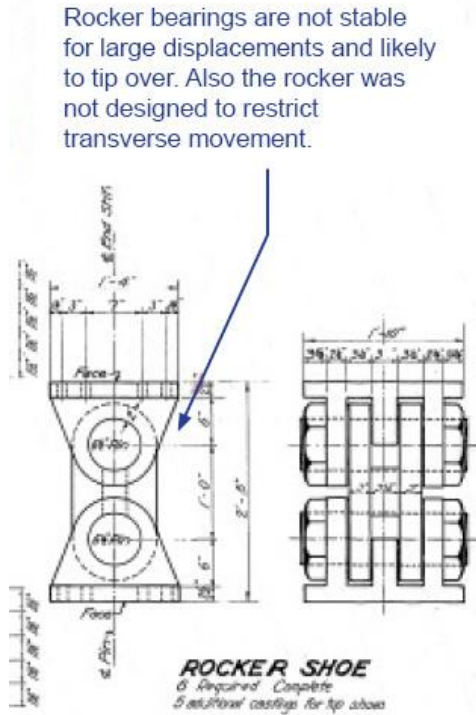
Figure 5-8. Superstructure to Column Connection Vulnerability



Steel Rocker Bearings – Rocker bearings are not stable for larger displacements and are likely to tip over. Also, the rocker was not designed to restrict transverse movement. Longitudinal restrainers installed in the early 2000s tied the superstructure together near deck level and do not restrict transverse movement of the superstructure. Retrofit of the steel rocker bearings is required based on the lack of a reasonable load path for seismic

loading; therefore, a detailed analysis of the demands and capacities of the existing rocker bearings was not required (see Figure 5-9).

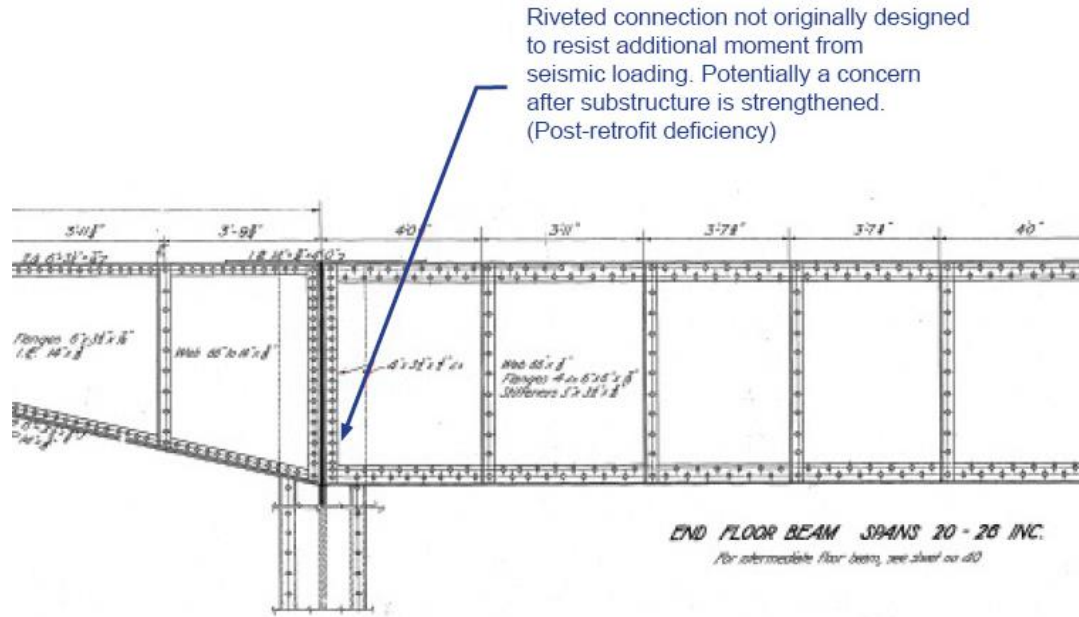
Figure 5-9. Rocker Bearing Vulnerability



End Floor-Beam Flexural or Shear Strength – The superstructure strength becomes deficient after the substructure columns is strengthened. The riveted connection was not originally designed to resist additional moment from seismic loading, and this is exacerbated by needing to be capacity-protected against the column. There is a relatively long cantilever supporting a portion of the roadway. Analysis of the existing structure for the CSZ event resulted in C/D ratios for all end floor beams of less than 1.0 at the columns (see Figure 5-10). In addition, load rating analysis results indicated that

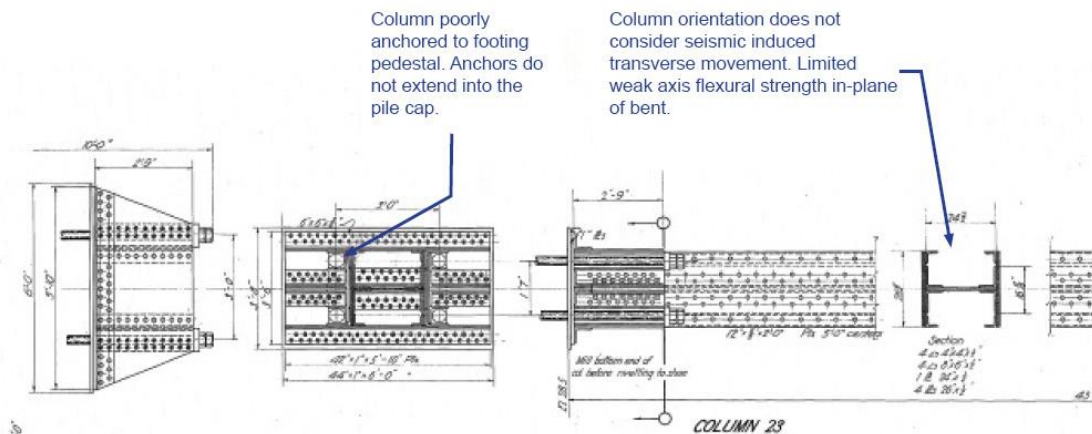
the floor beams do not have sufficient strength to meet the load rating requirements per current design standards.

Figure 5-10. Steel Floor-Beam Connection Vulnerability



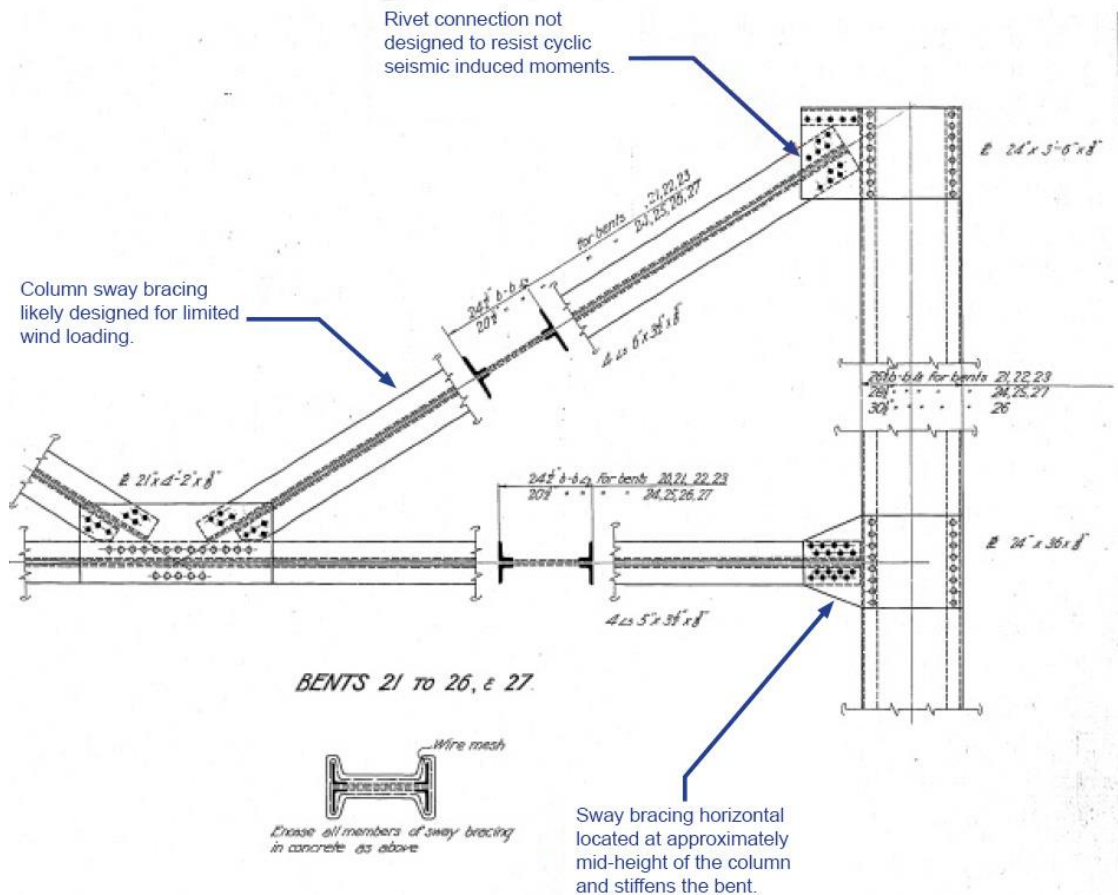
Column Flexural or Shear Strength – Columns are poorly anchored to footing pedestals. Anchors do not extend into the pile caps. Column orientations do not accommodate seismic-induced transverse movement. Weak axis flexural strength in the plane of the bent. Analysis of the existing structure for the CSZ event resulted in C/D ratios for all steel columns of less than 1.0 when evaluating axial-flexure interaction in the columns and tension in the anchor bolts (see Figure 5-11).

Figure 5-11. Steel Column Vulnerability



Column Sway Bracing Strength – Column sway bracing was likely designed for limited wind loading. The rivet connections were not designed to resist cyclic seismic-induced moments. The sway bracing horizontal is located at approximately mid-height of the column and stiffens the bent. Analysis of the existing structure for the CSZ event resulted in C/D ratios less than 1.0 for the steel column bracing (see Figure 5-12).

Figure 5-12. Column Bracing Vulnerability



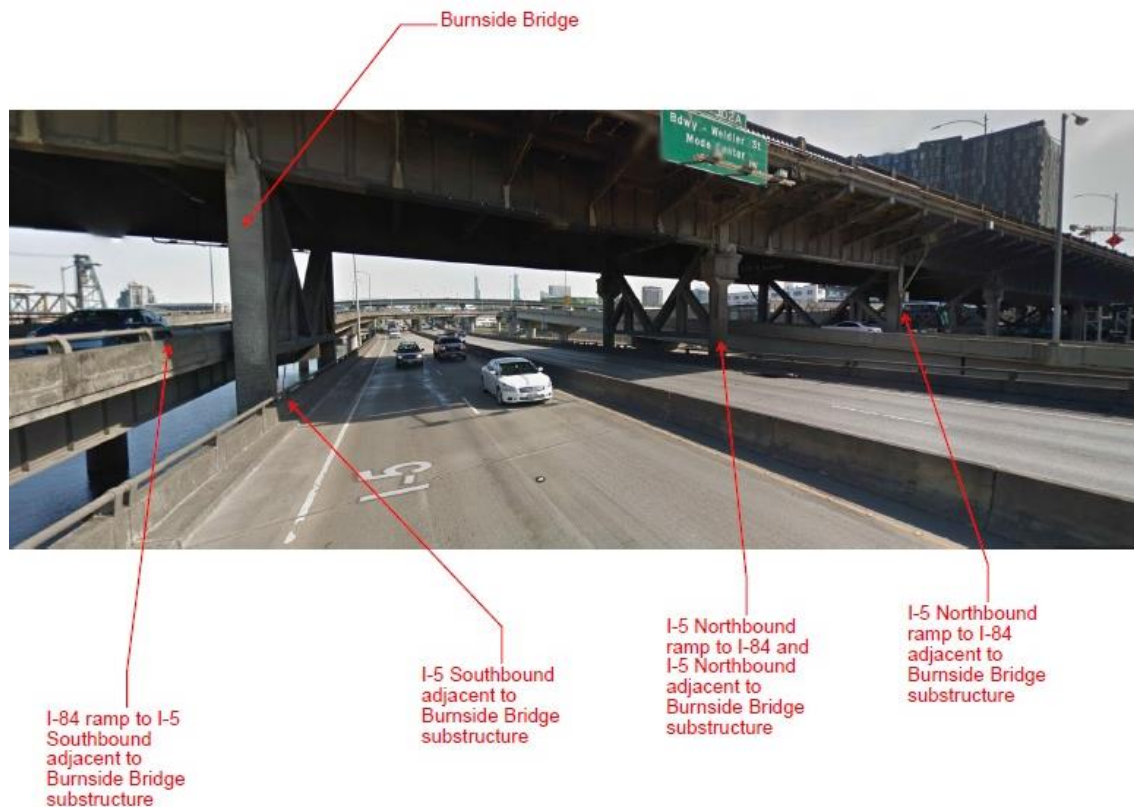
Timber Pile Lateral Strength and Uplift Capacity – Pile group capacity is inadequate to limit settlement due to liquefaction, to resist uplift and downdrag forces, and to resist displacements and forces from lateral spreading. Unreinforced footing section with no top mat of reinforcement. Poor connection detail as column anchorage does not extend into pile cap with adequate embedment. Embedment of piles into pile caps is inadequate to resist seismic uplift forces. Due to the amount of liquefaction and lateral spread at Bents 21 through 27, foundation retrofit is required because a reasonable load path for seismic forces in these elements could not be found, and structural analysis of the demands and capacities of the timber piles and associated pile caps was not required (see Figure 5-6).

Liquefiable Soils and Lateral Spreading – Bents 21 to 27 are on timber piles with limited capacity to resist liquefaction-induced settlement and lateral forces and

displacements due to lateral spreading. Additional detail regarding the liquefaction and lateral spread analysis can be found in the *EQRB Geotechnical Report* (Multnomah County 2021) (Appendix A).

Damage from Adjacent Structure – Bents 21 to 23 are adjacent to various highway structures (on-ramp from I-84 westbound to I-5 southbound, I-5 north- and southbound, and I-5 northbound to I-84 eastbound), which could impact each other during a seismic event resulting in catastrophic damage to the columns of Bents 21 to 23 (see Figure 5-13).

Figure 5-13. Vulnerability to Impact from Adjacent Structures



5.2.2 East Approach Spans 28 to 34

Spans 28 to 34 are RCDG spans that match the description of Spans 1 to 13 in the west approach structure (provided earlier). The east abutment (Bent 35) is a gravity-type wall abutment. The following seismic vulnerabilities were identified with these spans.

Superstructure Flexural or Shear Strength – Poor seismic detail. Girder positive moment reinforcement spliced at column connection thus limiting moment capacity due to inadequate development length. Analysis of the existing structure for the CSZ event shows the C/D ratios for superstructure flexure at the bents to be less than 1.0 (see Figure 5-2). In addition, load rating analysis results indicated that the stringers do not

have sufficient strength to meet the load rating requirements per current design standards.

End Floor-Beam Flexural or Shear Strength – Poor seismic detailing. Floor-beam reinforcement has a short splice length or has limited embedment at column connection, thus limiting moment capacity due to inadequate development length. Inadequate negative moment capacity at supports to maintain elastic behavior for CSZ event. Analysis of the existing structure for the CSZ event shows the C/D ratios at the columns are less than 1.0 (see Figure 5-3). In addition, load rating analysis results indicated that the floor beams do not have sufficient strength to meet the load rating requirements per current design standards.

Column Flexural or Shear Strength, Poor Confinement Detailing – Minimal longitudinal column reinforcing extending into footing with inadequate development length. Poor confinement and lack of seismic hooks, with ties and hoops at 1-foot 3-inch spacing. C/D ratios for columns are less than 1.0 (see Figure 5-4).

Footing Size and Strength, Unreinforced Footings – Bents 28 to 35 are on small spread footings with limited bearing capacity to resist overturning. Unreinforced footing section with no top mat of reinforcement. Poor connection detail as column reinforcement does not extend into the footing with adequate embedment. Due to the lack of reinforcing in the footing, foundation retrofit is required because a reasonable load path for seismic forces in these elements could not be found, and structural analysis of the demands and capacities of the footings was not required (see Figure 5-5).

Tall Abutment Retaining Wall Footing Size – Narrow footing size to resist overturning. Fixed end connection between superstructure and abutment imparts seismic loads on unreinforced abutment wall. Due to the lack of reinforcing in the abutment, it was determined that retrofit is required because a reasonable load path for seismic forces in these elements could not be found, and structural analysis of the demands and capacities of the abutment was not required (see Figure 5-6).

5.3 Steel Truss Fixed River Spans

Two steel truss fixed river spans connect the east and west approach structures to the main river bascule span over the water. The west truss span connects the west approach structure at Pier 1 to the bascule span structure at Pier 2. The east truss span connects the east approach structure at Pier 4 to bascule span structure at Pier 3. Steel truss fixed-span elevation and section views are shown in Figure 5-14.

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

The analysis was conducted step by step to identify the required retrofit. Table 5-1 below indicates representative members that have C/D ratios less than 1.0 under a CSZ event; therefore, those members need to be strengthened.

Figure 5-14. Steel Truss Span Elevation and Sway Bracing

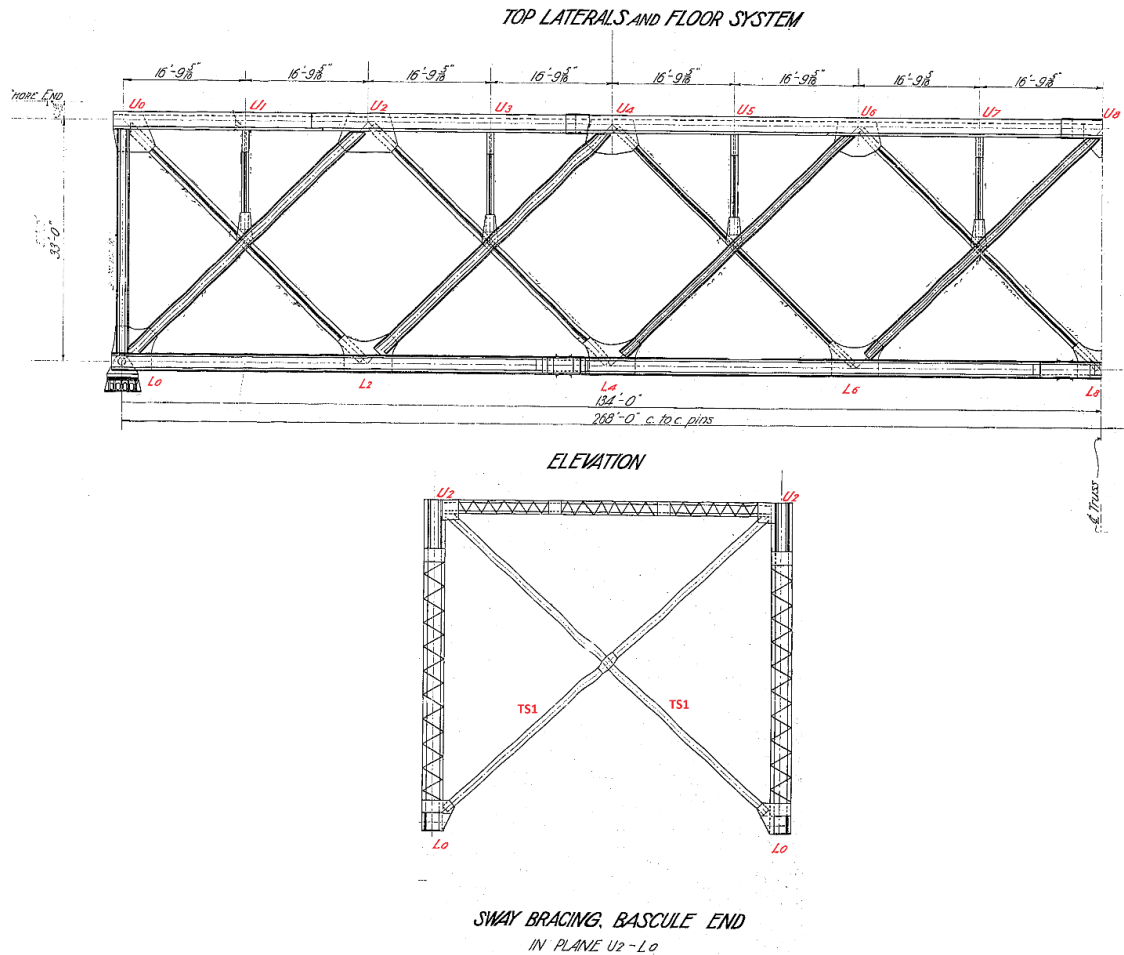


Table 5-1. C/D Ratio Summary Existing Fixed Truss Span

Drawing Member #	Member Location	Failure Mode	C/D Ratio
U8U9	Top chord at midspan	Compression	2.04
L8L9	Bottom chord at midspan	Tension yield	2.12
L8L9	Bottom chord at midspan	Tension fracture	3.28
U8L9	Diagonal near midspan	Compression	1.99
TS1	Out-of-plane bracing near bascule	Compression	0.08
TS1	Out-of-plane bracing near bascule	Tension yield	0.40
TS1	Out-of-plane bracing near bascule	Tension fracture	0.61
U16L16	End post near bascule	Compression	3.37
	Fixed End Support Anchor Bolts	Shear	0.22

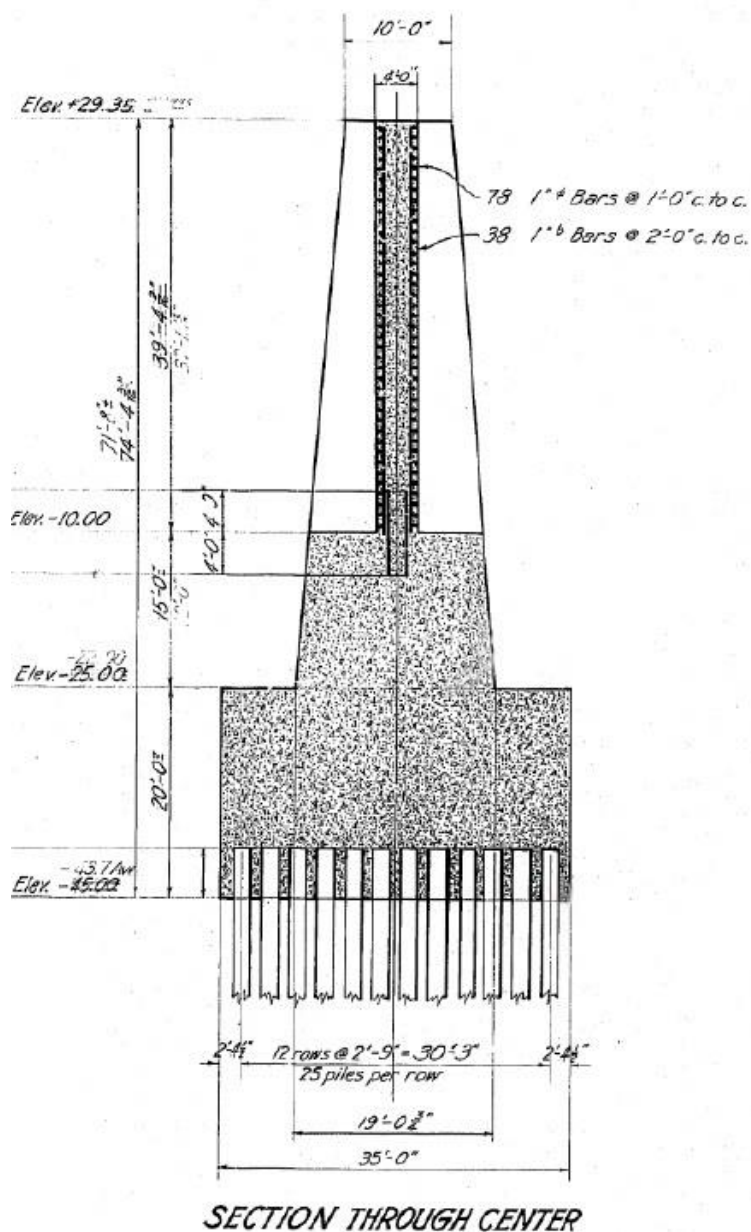
Note: Red C/D ratios less than 1.0 indicate deficiency for expected load demands.

5.3.1 Pier Foundations

5.3.1.1 Pier 1 and Pier 4 Foundations

Existing Pier 1 and Pier 4 foundations consist of unreinforced pile caps and groups of timber piles (see Figure 5-15). These foundations were neither designed nor constructed according to current seismic design requirements and detailing practices. During a design-level earthquake, these foundations can have multiple failure modes.

Figure 5-15. Piers 1 and 4 Walls and Foundations



Timber Pile Failure

Geotechnical analysis (see the *EQRB Geotechnical Report* (Multnomah County 2021) in Appendix A) indicated that liquefaction-induced settlement of the liquefiable layer and overlying soil would result in the following:

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

Pile Cap Failure

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

Foundation Collapse

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

5.3.1.2 Pier 2 and Pier 3 Foundations

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

5.3.2 Pier Column and Wall

5.3.2.1 Pier 1 and Pier 4

Each pier consists of two concrete columns supported on pile foundations (see Figure 5-16). The columns are connected by a reinforced concrete shear wall below the fixed truss bearing locations. The columns are reinforced above the top of the shear wall and are unreinforced below. The foundations consist of an unreinforced concrete pile cap supported by timber piles.

The southern column of Pier 1 has experienced significant deterioration. There is cracking, spalling, and delamination on the east face of the column, and existing patch repairs have been performed at the crack location. There is also a large vertical crack on

the south face of the column just behind the interface with the existing Harbor Wall. This deterioration was addressed as part of the Burnside Bridge Maintenance Project.¹ Two vertical steel distribution beams on each side of the shear wall were installed on the south and north faces of the column and are connected by drilled and grouted high-strength threaded rods. Steel cover plates were installed using resin-bonded anchors on the east face of the column.

Figure 5-16. Pier 4



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

- Lack of lateral confinement reinforcing in the columns. During an earthquake, the concrete is predicted to crack and fail, and the vertical main reinforcement is predicted to buckle, thus causing the pier columns to lose vertical support capacities.
- Lack of adequate rebar embedment length, lapping splice length, and seismic hook details. During an earthquake, the reinforcement will not be able to develop full strength capacity, will un-bond, and lose the load-carrying capacity. Unreinforced plain concrete is used in the lower portion of the columns and the walls. This

¹ <https://multco.us/bridges/burnside-bridge-maintenance-project>

unreinforced concrete will crack and fall apart during an earthquake (C/D ratio of 0.42), causing the piers to collapse.

- Because of the above-identified deficiencies, Pier 1 and Pier 4 are anticipated to fail in both flexure and shear during a design-level earthquake event; therefore, seismic retrofit is required.

5.3.2.2 Pier 2 and Pier 3

Pier 2 and Pier 3 are described in the Bascule Span Section 5.4.

5.3.3 Truss Supports and Pier Connections

5.3.3.1 Expansion Bearings at Pier 1 and Pier 4

The expansion ends of the steel truss spans are supported on Pier 1 and Pier 4 which also support the concrete approach spans from approach structures. The support bearings under the steel trusses are rocker-type steel bearings (see Figure 5-17). During late 2001, the bridge went through a Phase I seismic retrofit (Multnomah County 2001) that included:

- Installing seismic restrainers connecting the top chords of the steel trusses to the concrete approach spans.
- Retrofitting the rocker-type bearings by inserting bearing wedges (see Figure 5-18 and Figure 5-19).

Figure 5-17. Expansion Rocker Bearing

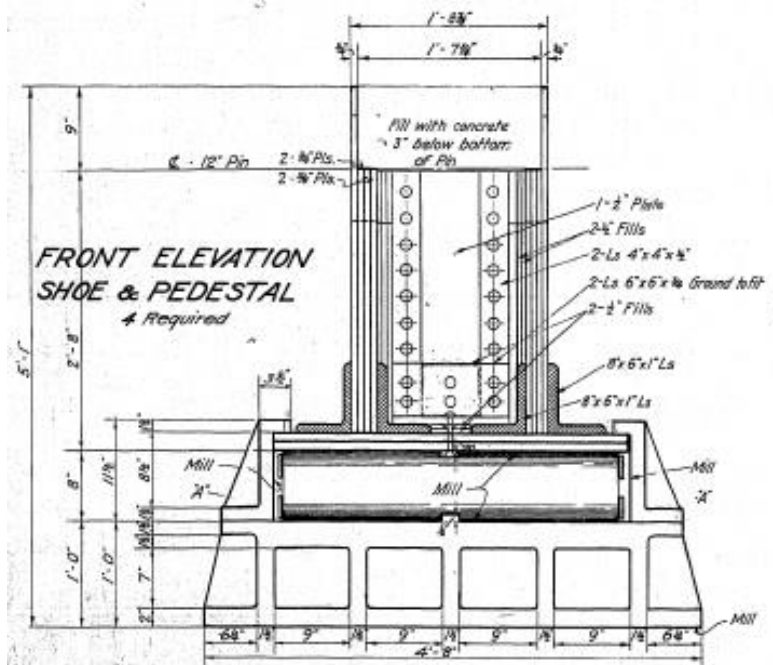


Figure 5-18. Existing Rocker Bearing Details

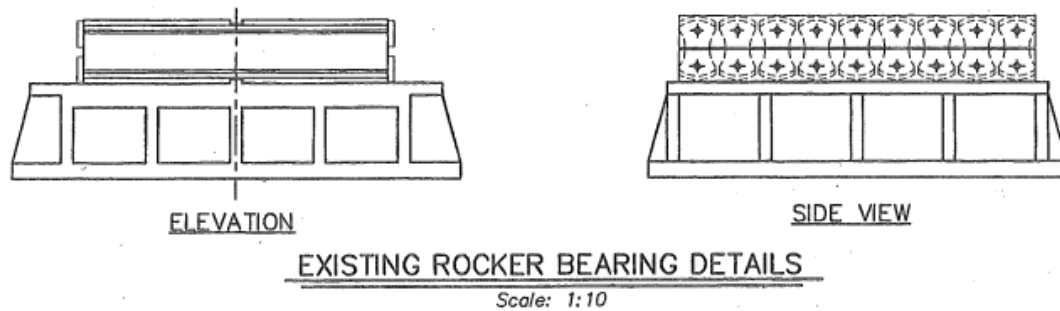
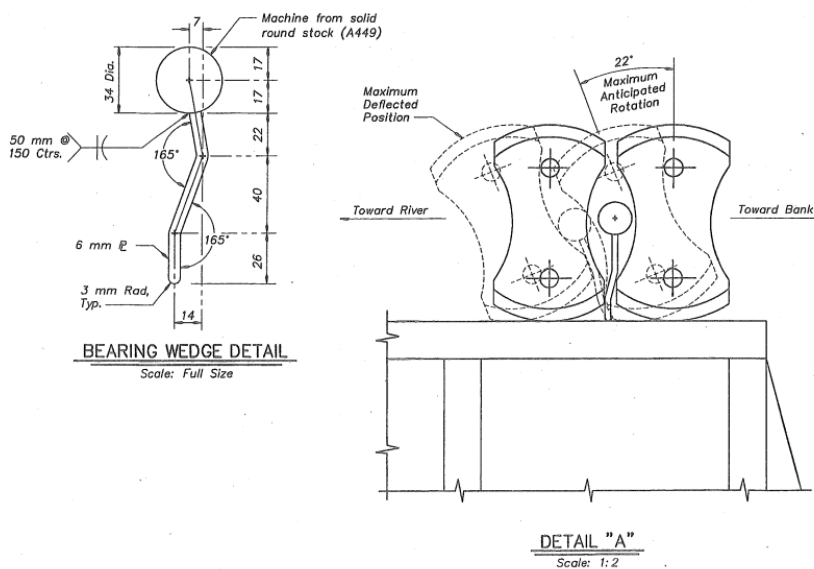


Figure 5-19. Rocker Retrofit in 2001



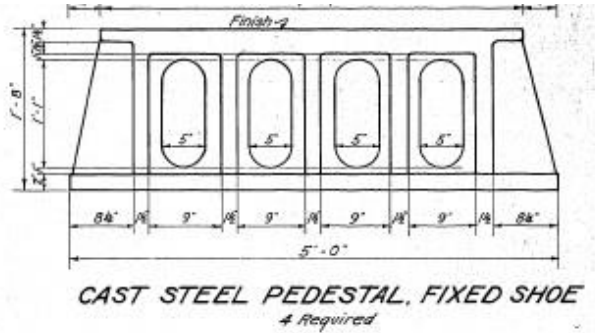
Expansion Rocker Bearing Failure

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

5.3.3.2 Fixed Bearings at Pier 2 and Pier 3

The steel truss spans are supported on fixed bearings (see Figure 5-20) at bascule Pier 2 and Pier 3.

Figure 5-20. Fixed Bearing Shoe



Fixed Bearing Anchor Bolt Shear Failure and Concrete Cracking

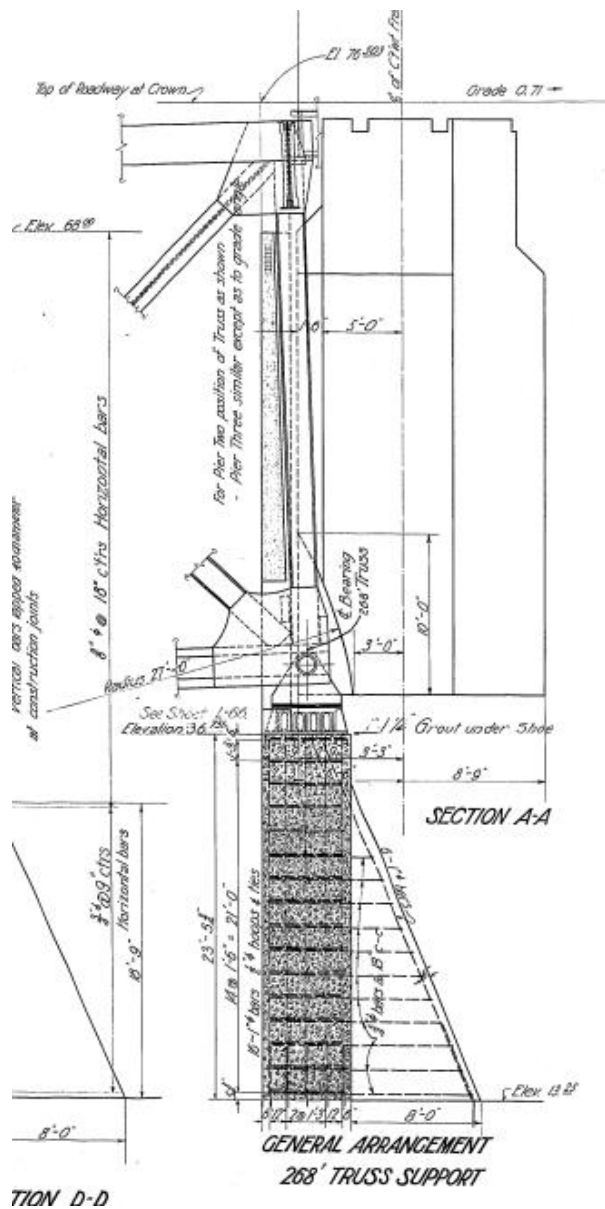
Existing anchor bolts of the fixed bearings are insufficient to resist seismic-induced horizontal forces. The concrete pier wall below the bearing is not reinforced to resist seismic loads; therefore, the concrete surrounding the anchor bolts could crack causing the anchor bolts to lose lateral resistance.

Short Seating Lengths

On the fixed support ends, the seating lengths do not conform to the current *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (AASHTO 2012) requirements for seismic design. The seating length provided is 33 inches (see Figure 5-21), which is less than the required length per current standards. The truss girders can slip off from the

support at the pier tops due to the bearing anchor bolts being sheared off and unconfined concrete cracking, as indicated in the previous section.

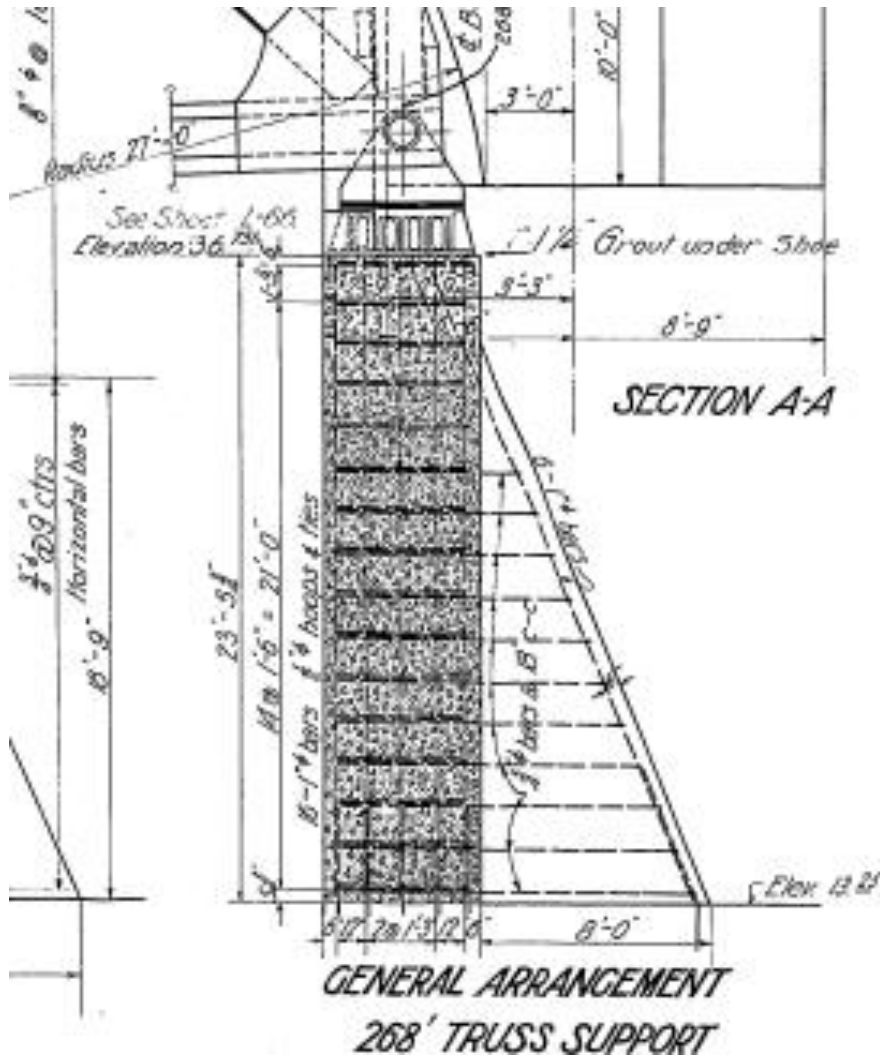
Figure 5-21. Truss Span Support at Piers 2 and 3



5.3.3.3 Lack of Effective Transverse Restrainers

The pier columns or walls at Piers 1, 2, 3, and 4 are not properly reinforced with seismic reinforcing details; therefore, these columns and walls cannot function effectively as transverse seismic restrainers during a design-level earthquake. Therefore, the truss support will pull free from the existing pier structure. Reinforcement of Piers 1 and 4 can be seen in Figure 5-15. Reinforcement of the wall under the truss supports at Piers 2 and 3 can be seen in Figure 5-22.

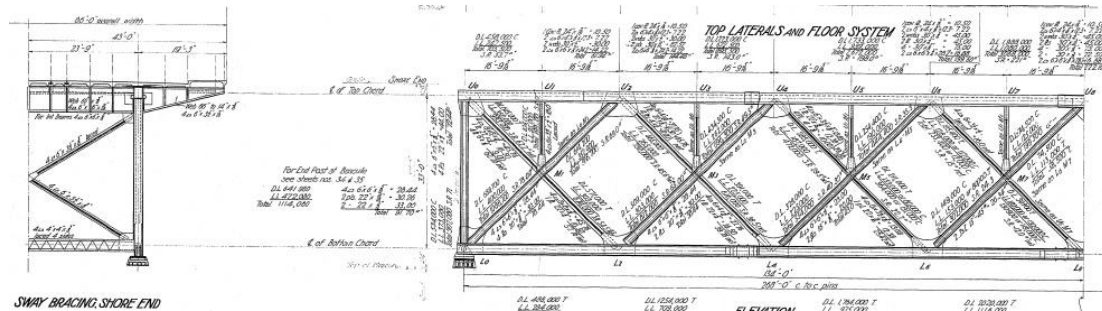
Figure 5-22. Wall Reinforcement under the Bearings at Piers 2 and 3



5.3.4 Steel Truss Superstructure

Each of the truss spans is a 268-foot-long constant depth steel deck truss (see Figure 5-23). A reinforced-concrete bridge deck is supported by steel stringers and floor beams that are connected to the main steel trusses. An analysis of the truss member was conducted to determine the seismic deficiencies. The following sections generally describe the analysis results.

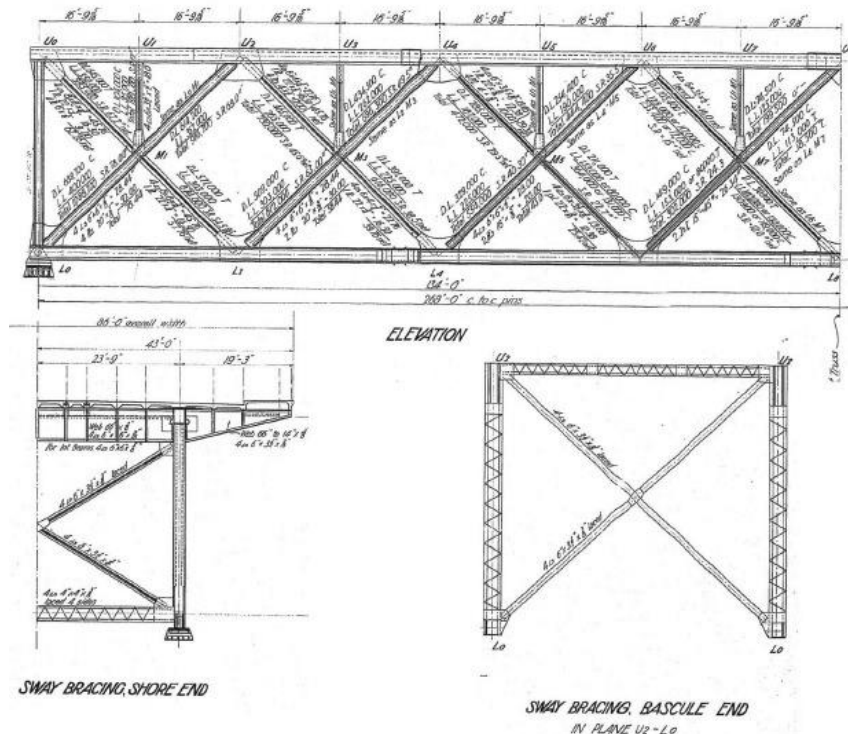
Figure 5-23. Fixed Steel Truss Span



5.3.4.1 Weak Lateral Load Paths

The existing steel truss lacks a proper lateral load transfer path that is capable of transferring the horizontal seismic-induced forces from the deck down to the support at the bearings (see Figure 5-24).

Figure 5-24. Truss Span Bracings



Sway bracing is absent from four of the eight bays in the fixed spans. Sway bracing is required in each bay to prevent the collapse of the deck and top chord caused by lateral movement. Due to the limited bracing, the bracing that currently is in place would become overloaded during a seismic event. The out-of-plane bracing near the bascule end of the fixed span has a C/D ratio of 0.08. The bridge deck to steel floor-beam connection shear capacity is insufficient based on the as-built plans, shop drawings, and rehabilitation plans. There are no connectors between the deck and floor beams of the fixed span, so the existing deck will not act compositely under seismic loading.

5.3.4.2 Insufficient Bottom Lateral Bracings

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

5.4 Bascule River Span

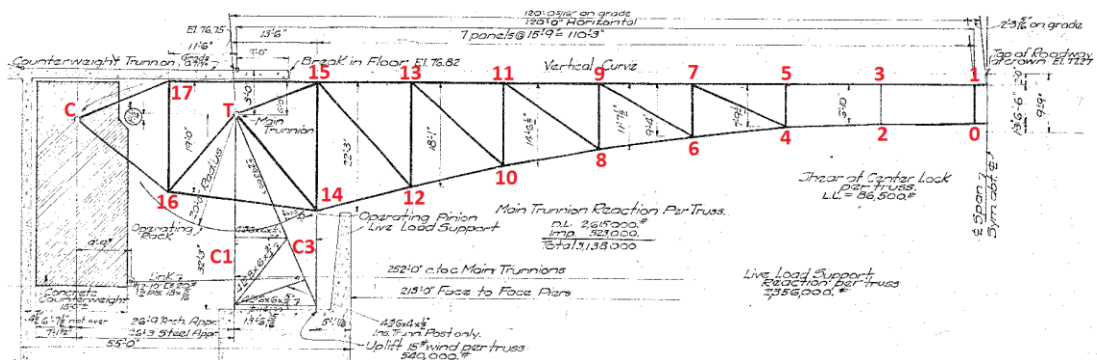
The Burnside Bridge main river span crossing the Willamette River navigation channel is a 252-foot-long (trunnion-to-trunnion) double-leaf steel deck truss bascule span. According to the original as-built plans (Multnomah County 1923; Multnomah County 1924), some of the major dimensions are described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

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

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

Reinforced concrete decks are on top of the variable-depth bascule leaves. Each of the two bascule leaves, including the counterweight (see Figure 5-25), is supported via trunnion support steel frames on concrete pedestals inside the bascule pier. The centerlines of the bascule trunnions are at an elevation of 70.85 (NAVD 88), and the supporting concrete pedestals are at an elevation of 38.6 (NAVD 88).

Figure 5-25. Bascule Leaf and Counterweight



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

The upper part of the pier is enclosed by reinforced concrete pier walls from under the bridge deck down to elevation -30.9 (NAVD 88). A pit floor inside the bascule pier is at elevation 15.35 (NAVD 88). The as-built plans show no reinforcement in the concrete pier walls from the pit floor to the top of the pile cap.

The pier walls are connected with straight dowels to the unreinforced pile caps founded on timber piles. The bottoms of the pile caps are at elevation -67.9 (NAVD 88).

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

The analysis was conducted step by step to identify the retrofit required. Table 5-2 shows members with C/D ratios less than 1.0 under a CSZ event. At a minimum, these members need to be strengthened. In Figure 5-25, the joint numbers are shown, and the members are identified in Table 5-2 by their joint-joint connection.

Table 5-2. C/D Ratio Summary for Existing Bascule Span

Drawing Member #	Member Location	Failure Mode	C/D Ratio
16-C	Counterweight Compression Support	Comp.-Moment Interaction*	0.40
16-C	Counterweight Compression Support	Shear	0.21
Counterweight Link	Counterweight Link	Compression	0.05
Counterweight Link	Counterweight Link	Tension Yield	0.31
Counterweight Link	Counterweight Link	Tension Fracture	0.34
T-C1	Trunnion Post	Compression	1.28
-	Trunnion Support Anchor Bolts	Tension	0.93
14-T	Trunnion Brace	Comp.-Moment Interaction*	1.01
14-T	Trunnion Brace	Shear	0.21
14-T	Trunnion Brace	Ten.-Moment Interaction*	0.62
15-T	Trunnion Link	Comp.-Moment Interaction*	0.77
15-T	Trunnion Link	Shear	0.14
15-T	Trunnion Link	Ten.-Moment Interaction*	0.66
14-15	Bottom Chord Bascule Truss	Compression	1.73
12-15	Diagonal Bascule Truss	Tension Yield	2.46
12-15	Diagonal Bascule Truss	Tension Net Fracture	3.30
13-15	Top Chord Bascule Truss	Tension Yield	1.79
13-15	Top Chord Bascule Truss	Tension Net Fracture	2.53

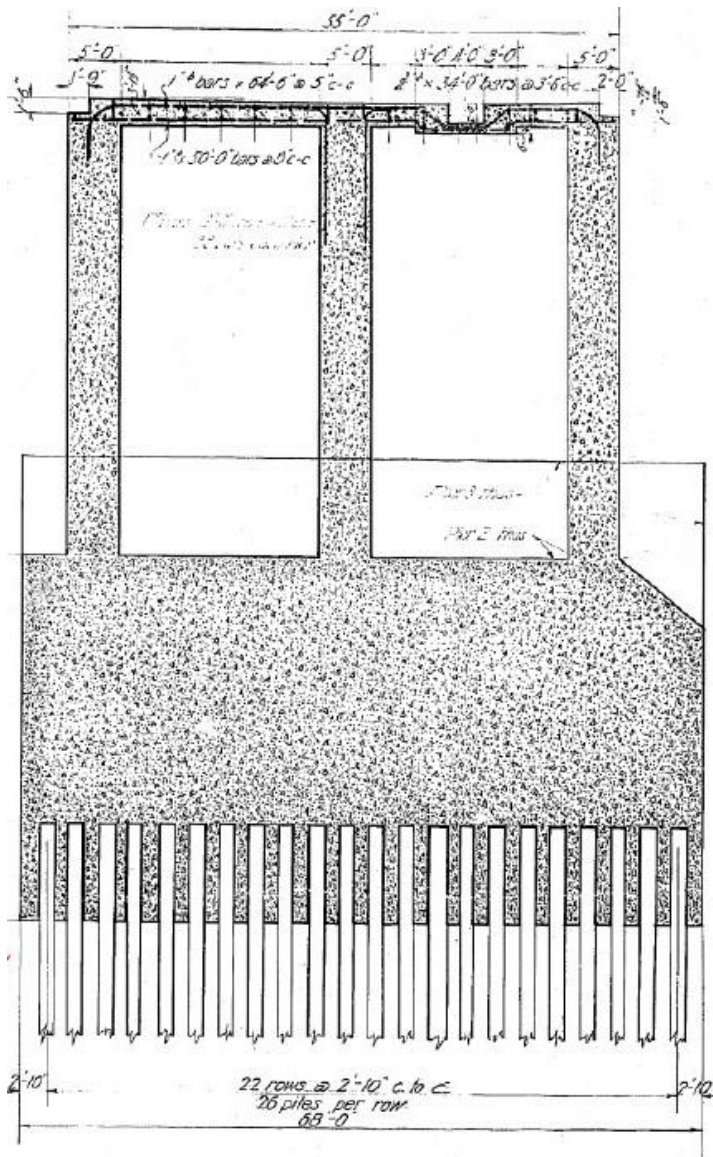
* For Interaction failure, C/D ratio shown represents 1/(Interaction Result)

Note: Red C/D ratios less than 1.0 indicate deficiency for expected load demands.

5.4.1 Pier Foundations

Existing bascule Pier 2 and Pier 3 foundations consist of unreinforced pile caps and groups of timber piles (see Figure 5-26). These foundations were neither designed nor constructed according to current seismic design requirements and detailing practices. During a design-level earthquake, these foundations would fail as described below.

Figure 5-26. Bascule Pier Foundation



5.4.1.1 Timber Pile Failure

Geotechnical analysis (see the *EQRB Geotechnical Report* (Multnomah County 2021) in Appendix A) indicated that liquefaction-induced settlement of the liquefiable soil layer will result in the following:

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

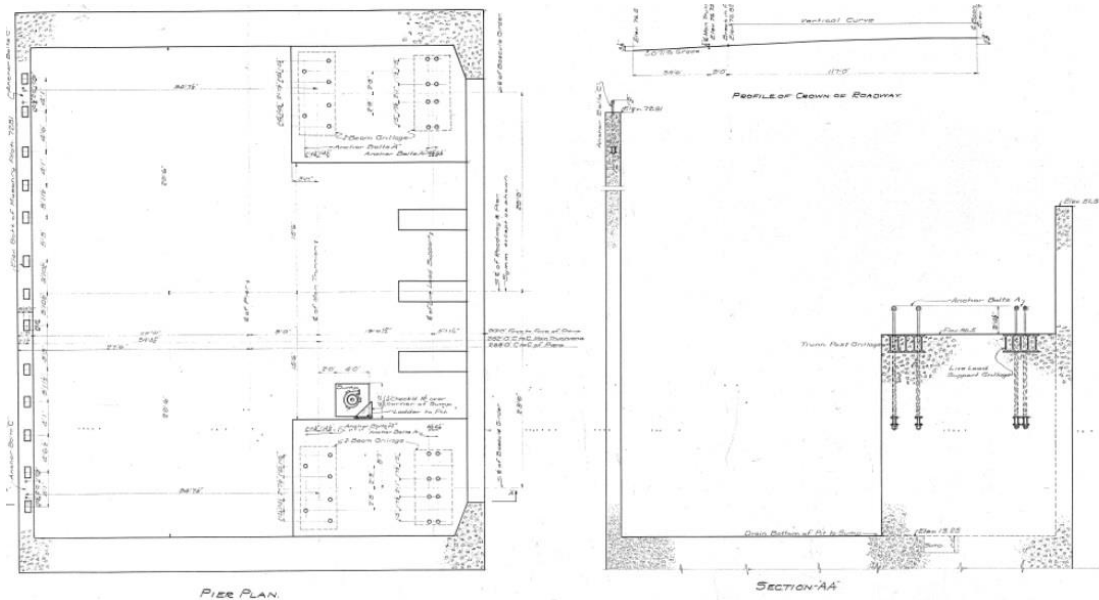
5.4.1.2 Pile Cap Failure

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

5.4.2 Pier Walls

Pier walls were designed for non-seismic lateral loads such as wind loads and gravity loads only. They are not reinforced and detailed to resist seismic forces (see Figure 5-27).

Figure 5-27. Bascule Pier Walls

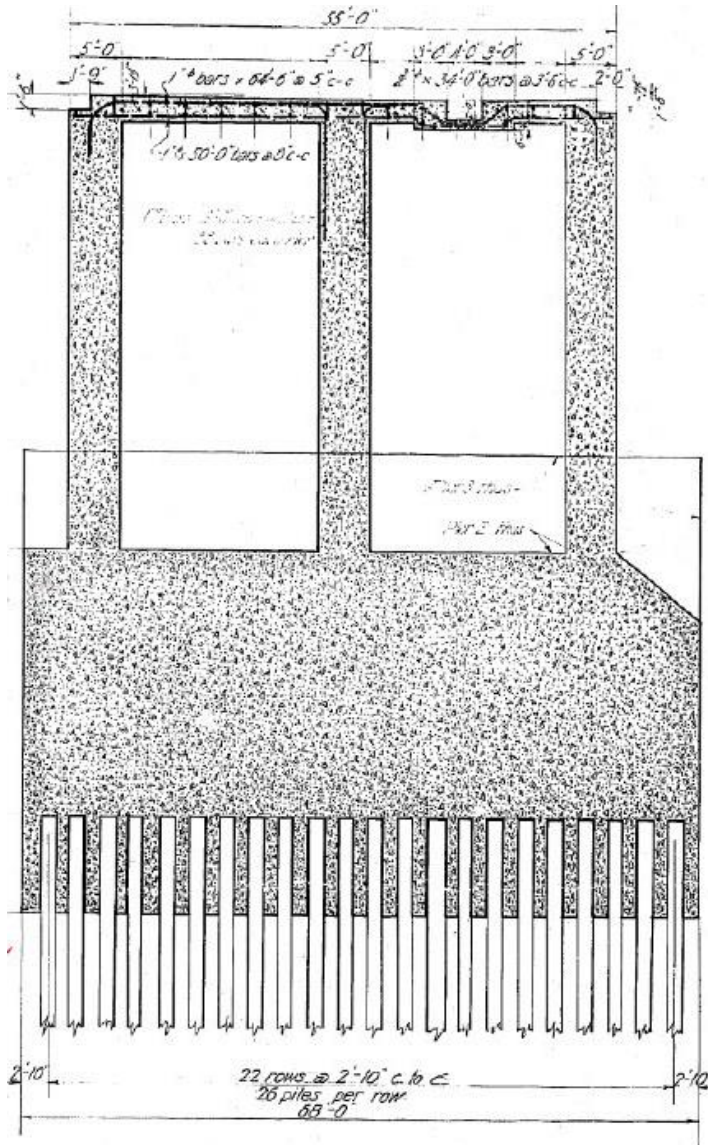


In examining the as-built plans (Multnomah County 1924), the lower parts of the piers below the pit floor are not reinforced (see Figure 5-28). The unreinforced and under-reinforced concrete pier wall is vulnerable under seismic loads and lateral movement. The pier wall reinforcing is not detailed as required per current seismic design standards. Major deficiencies include:

- Lack of lateral confinement reinforcing in the walls (see Figure 5-28). The pier back wall under the bearing of the steel trusses has lateral confinement reinforcing at 1-foot 6-inch vertical spacing, while current seismic design requires confinement reinforcing spacing of fewer than 6 inches. During a design-level earthquake, the vertical main reinforcement rebar will buckle due to lack of confinement, and the

concrete will crack and fall apart, thus causing the pier walls to lose vertical support capacities. The C/D ratio for this is 0.81.

Figure 5-28. Bascule Pier Lower Walls Unreinforced



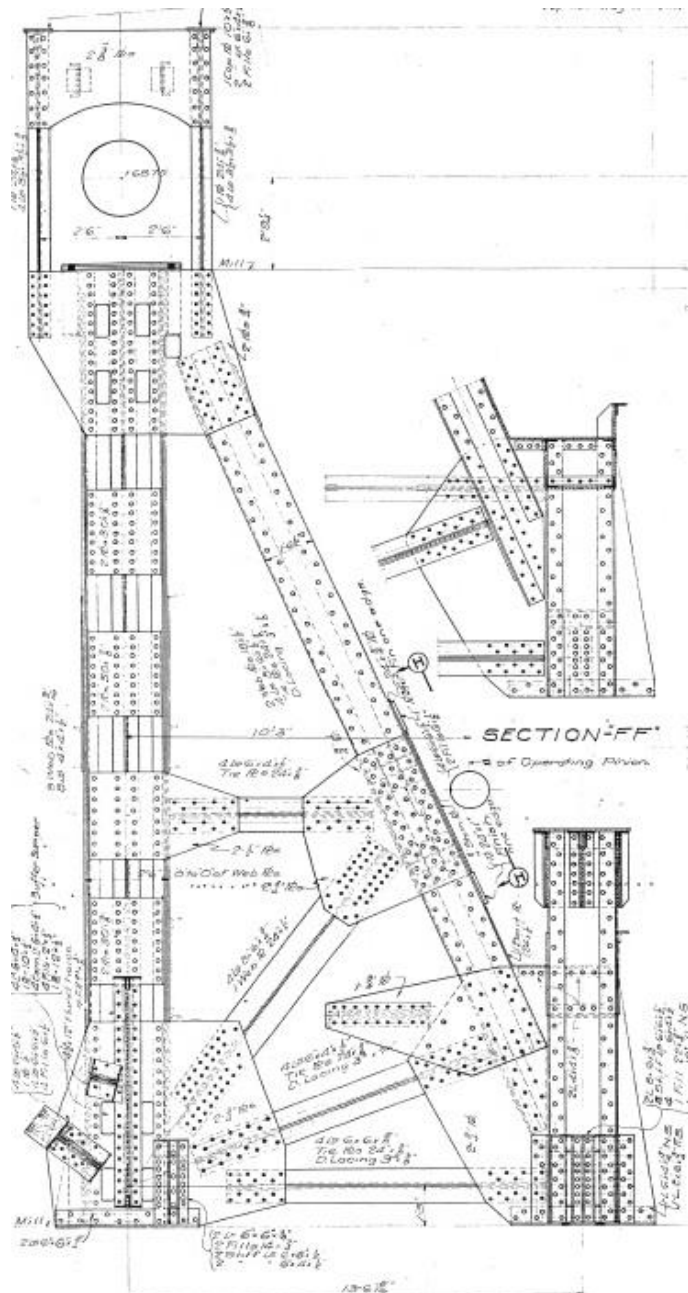
- No dimensions appear in the as-built plans for rebar embedment length, lapping splice length, and seismic hook details. Bridges built in the 1920s typically do not meet current seismic design requirements for the embedment length and splice length, etc. During an earthquake, the reinforcement would likely pull out and lose the load-carrying capacity.
- Unreinforced plain concrete was used in the lower portion of the piers and the walls. This unreinforced concrete will crack and fall apart during an earthquake, causing the piers to collapse. This concrete has a C/D ratio of 0.20 for this failure mode.

- Because of the above-identified deficiencies, bascule Pier 2 and Pier 3 will fail in both flexure and shear during a design-level earthquake event; therefore, seismic retrofit is required.

5.4.3 Trunnion Supports

The trunnion support frames were designed primarily for supporting the vertical loads of the bascule leaves and counterweights (see Figure 5-29).

Figure 5-29. Trunnion Tower Support Frame



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

5.4.3.1 Trunnion Support Frame Failure

The trunnion support frames are heavily loaded because all the loads from the bascule span leaf, including the counterweight, are transferred through the trunnion support frame to the piers. Without effective lateral restrainers or support, under design-level seismic motion and lateral forces, these trunnion support frames will fail in buckling.

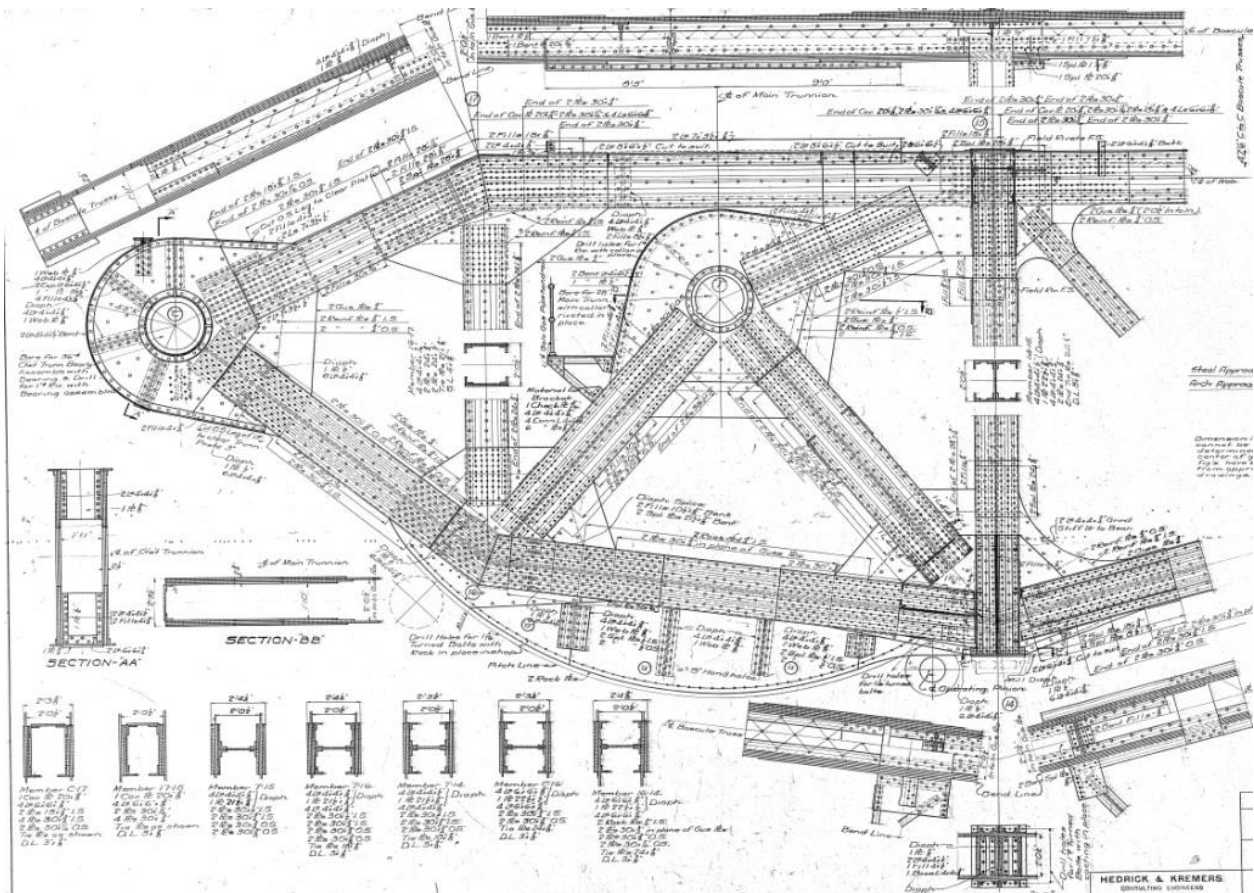
5.4.3.2 Anchor Failure

Existing anchor bolts under the trunnion tower support frames are insufficient to resist seismic and longitudinal forces. These anchor bolts will fail as a result of bolt shearing or concrete cracks. For combined tension and shear, the C/D ratio is 0.93. Note: For all anchor bolt calculations, grade A36 bolts were assumed due to lack of information.

5.4.4 Counterweight Supports

No lateral supports restrain the counterweight. This exposes the counterweight support frames to buckling (see Figure 5-30). Counterweight support member 16-C has a C/D ratio for shear of 0.21 and a value of 0.40 for the inverse of the compression-moment interaction (see Table 5-2). In addition, unrestrained lateral movement of the counterweight can impact the reinforced concrete walls supporting the sidewalks.

Figure 5-30. Counterweight Support Frame



5.4.4.1 Counterweight Link

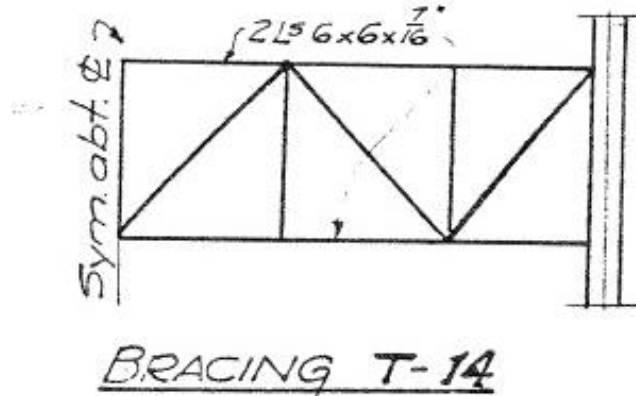
The existing counterweight link is exposed to large forces as it is the only member resisting the swinging of the counterweight in the longitudinal direction of the bridge. Failure of this member would cause unrestrained longitudinal motion of the counterweight, which could impact the wall of the pier supporting the fixed span.

5.4.5 Superstructure Connection to Trunnion

Bracing frame T-14 (see Figure 5-31) transmits lateral loads from the entire bascule span to the trunnion tower support frame. These members were not originally designed for transmitting the seismic motion-induced lateral forces, thus are vulnerable to buckling and yielding during a design-level seismic event. For shear, the C/D ratio for T-14 was 0.21. For the inverse of the tension-moment interaction, the resulting value was 0.62. In addition to T-14, member T-15 has a C/D ratio of less than 1.0 (see Table 5-2). Member

T-15 can be seen in Figure 5-30 as the top right member in the connection to the trunnion.

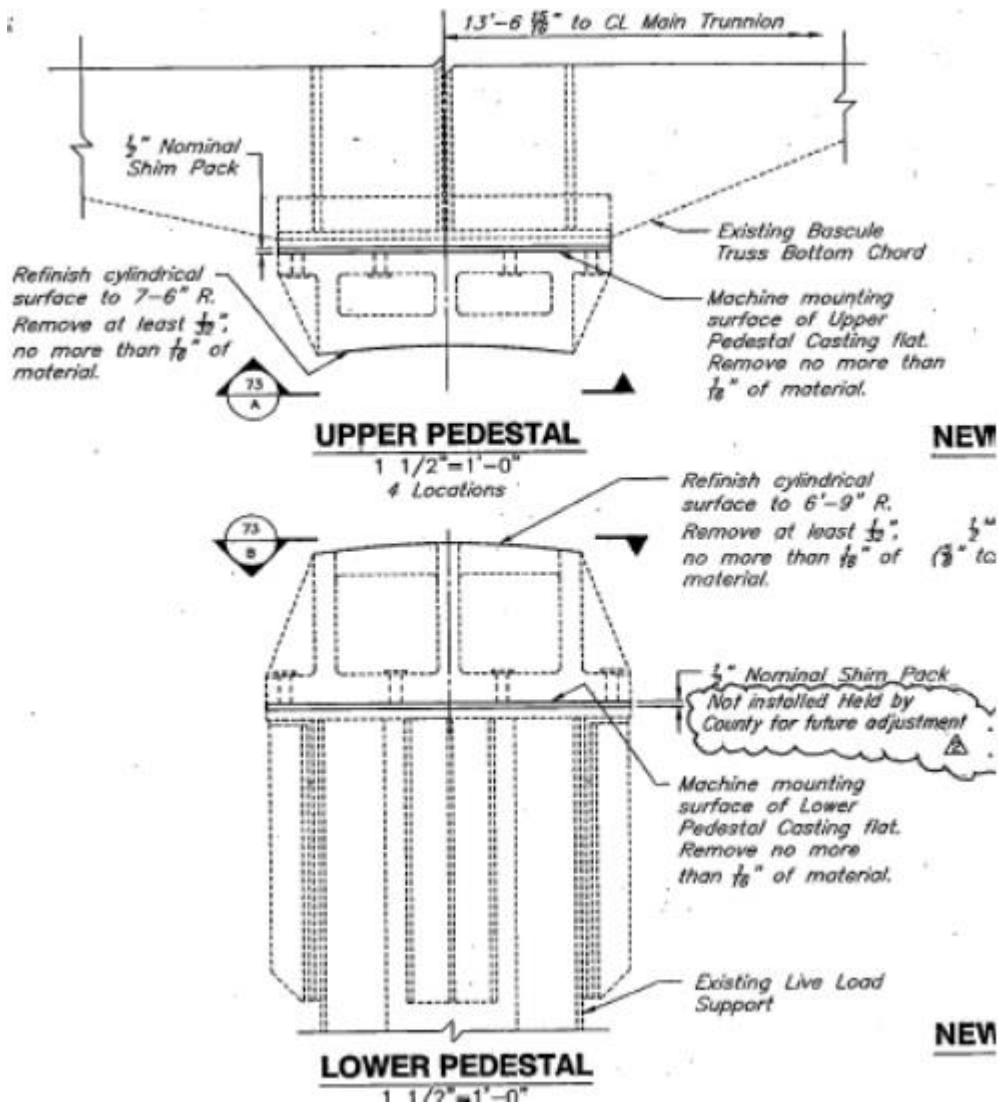
Figure 5-31. Bracing for Trunnion support Frame



5.4.6 Live Load Support Connections

The live load support shoes were designed to resist unbalanced vertical loads and live loads (see Figure 5-32). Because it is a simple bearing plate, the live load support cannot resist lateral or upward (tension) loads present in a seismic event. During a design-level seismic event, the bascule leaves will move horizontally and rotate around approximately the intersection of the centerline of the trunnion and the centerline of the bridge. Without the live load shoes' help in resisting vertical and horizontal rotational movement of the bascule leaf, the bascule trunnion support frames (member T-C3) will be exposed to large forces causing buckling or tension failures of the trunnion support frames.

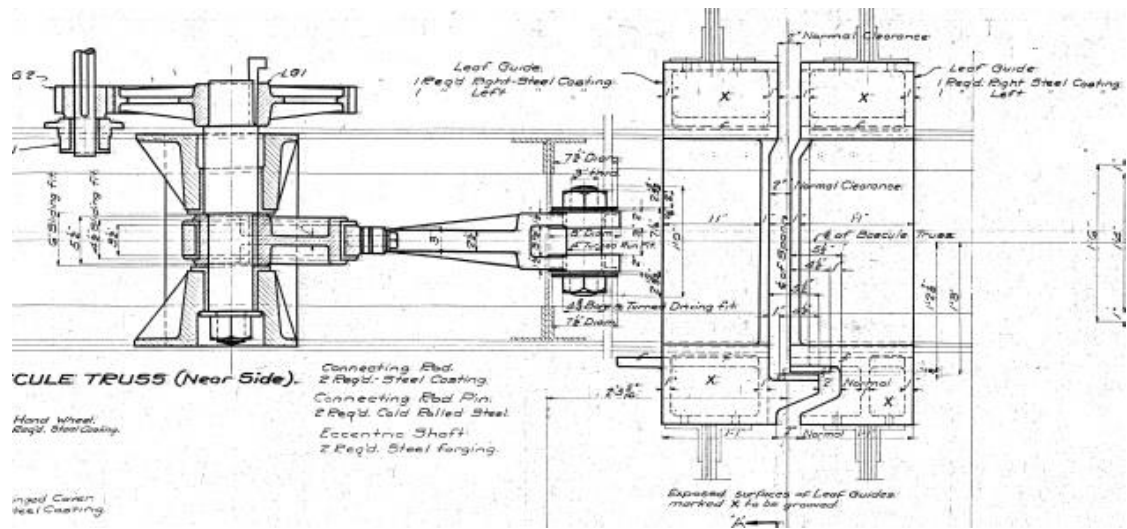
Figure 5-32. Live Load Shoe



5.4.7 Center Lock Shear

A typical bascule leaf center-span lock, such as the one on the Burnside Bridge, is not designed to transmit forces caused by the relative transverse displacement of the two bascule spans and can be severely damaged in a significant seismic event (see Figure 5-33). When the center lock is damaged during a seismic event, bascule leaves without the center lock can sway in different directions, causing large horizontal forces on the trunnion support frame.

Figure 5-33. Bascule Span Center Lock



5.5 Mechanical and Electrical Equipment

5.5.1 Mechanical Equipment

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

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

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

5.5.2 Electrical Equipment

Piers 2 and 3 each have an incoming service to provide power to each movable span leaf and other equipment. The incoming service is distributed to transformers and

panelboards for lighting and receptacles and to motor control centers to operate equipment ancillary to bridge operation, including the traffic warning gates and center-span lock.

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

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

5.6 Structures on the Bridge

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

The operator houses are integral parts of the seismic-deficient bascule piers that would be retrofitted during the pier retrofit.

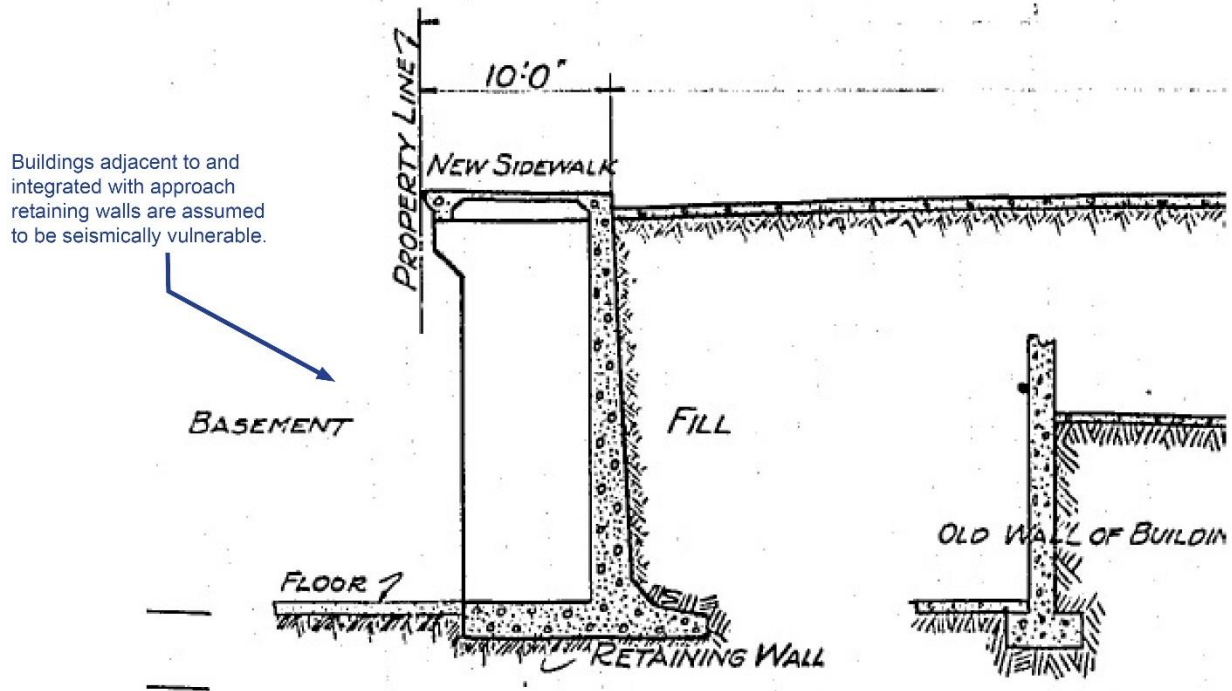
5.7 Bascule Leaves Opening Angle

According to Multnomah County, the existing bascule leaves can only open to an angle approximately 55 degrees from the horizontal. The reduced opening angle is caused by the enlarged bascule leaf counterweight size and the added stopper in the bascule pit. The added counterweight and the stopper were the results of several past rehabilitations, such as the deck repairs, etc. One of the requirements from the County is to restore the bascule leaf opening angle to 73 degrees and 30 minutes from the horizontal, as indicated in the 1924 as-built plans.

5.8 Approach Retaining Walls

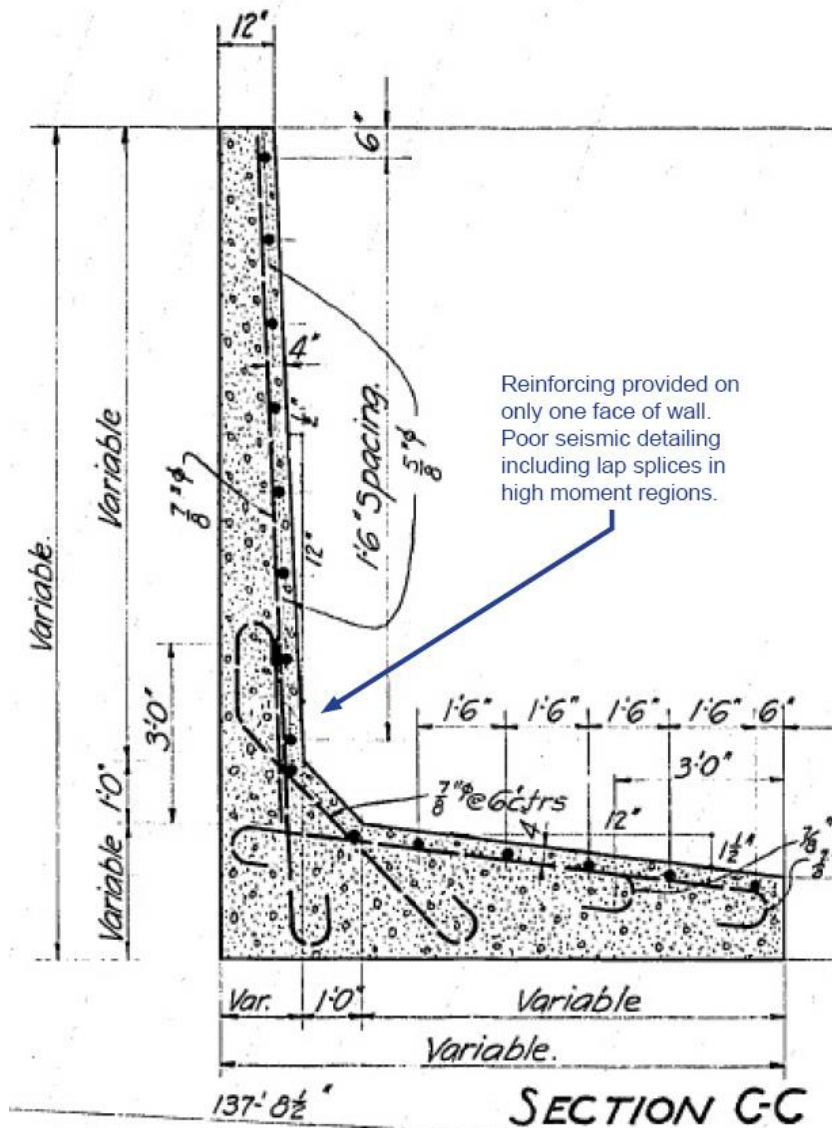
Approach retaining walls at both the west and east ends of the bridge consist of a mix of reinforced semi-gravity cantilever walls and counterfort walls. As discussed previously, in many cases, these retaining walls are integrated with the adjacent buildings (see Figure 5-34).

Figure 5-34. Retaining Walls at Approach



Vulnerabilities identified in these approach retaining walls include poor seismic detailing with lap splices in high moment regions, and buildings adjacent to and integrated with approach retaining walls (see Figure 5-35).

Figure 5-35. Retaining Wall Reinforcement



5.9 Existing Structure Material Properties

The original structure was built during 1924–1925, and the material properties were defined in Working Stresses in the as-built plans:

Concrete

Floor Slabs, Cross Girders, Cantilevers, Girders, etc.	650 lbs/sq in
Beams Continuous over Supports	815 lbs/sq in
Arch Rings Case 1: Not Including Temperature and Wind	600 lbs/sq in
Arch Rings Case 2: Including Temperature and Wind	800 lbs/sq in
Bond for Steel in Concrete	100 lbs/sq in
Flexural Stress for all Conditions not Including Wind	650 lbs/sq in
Flexural Stress for all Conditions Including Wind	800 lbs/sq in
Columns Direct Compression	450 lbs/sq in

Structural Steel

Tension, Net Section	16,000 lbs/sq in
Compression in Compression Members Fixed Ends	16,000 – 70L/r lbs/sq in

The main (river) spans were rehabilitated in 2005, and the deck was replaced. The various material properties of the replaced structural components are specified in the plans of *Burnside Bridge Main Span Rehabilitation (#00511)* General Notes, Drawing No. 70380, dated July 2005.

During the painting and rehabilitation project in 2017, some of the structural components were replaced or added. The material properties of those replaced or added structural components are specified in the plans of *Burnside St: Willamette River Bridge Painting and Rehabilitation Project*, General Notes, Drawing No. 98058, dated January 2017.

6 Structure Load Ratings

Load rating analyses were performed on the structures to identify load capacity deficiencies, according to the ODOT LRFR Manual (ODOT 2018). For those structural elements that do not meet the live load capacity requirements based on the load rating results, structural rehabilitation conceptual designs are included as part of the Enhanced Seismic Retrofit Alternative for the existing structures.

The following live loads were applied to the load ratings:

- AASHTO design load HL-93
- ODOT Legal Trucks
- Specialized Hauling Vehicles
- FAST Act Emergency Vehicles
- ODOT Continuous Trip Permit Trucks
- ODOT Single Trip Permit Trucks

Based on engineers' judgement by comparing the loads, future streetcar load is not a governing load case, therefore, the streetcar load is not load rated.

In addition to the requirements in ODOT LRFR Manual Section 1.5.2, Members to be Rated, the load rating analyses were also performed on the bridge deck to evaluate its capacity to meet current design criteria.

6.1 Approach Spans

The load rating results on the existing west and east approach spans indicated that there are multiple locations that have rating factors of less than one. Therefore, structural rehabilitation to increase the bridge live load-carrying capacity is included as part of the Enhanced Retrofit Alternative.

6.2 Steel Truss Fixed Spans

The load rating results on the existing steel truss fixed spans indicated that there are multiple locations that have rating factors of less than one, including locations such as the bridge deck, stringers, floor beams, bracings, etc. Therefore, structural rehabilitation to increase the bridge live load-carrying capacity is included as part of the Enhanced Retrofit Alternative.

The gusset plates and connection splices were not load rated. For this NEPA-phase conceptual design, it is assumed that 50 percent of the connections need to be rehabilitated.

6.3 Bascule River Span

The load rating results on the existing steel truss bascule span indicated that there are multiple locations that have rating factors of less than one, including locations such as the bridge deck, stringers, floor beams, bracings, etc. Therefore, structural rehabilitation to increase the bridge live load-carrying capacity is included as part of the Enhanced Retrofit Alternative.

The gusset plates and connection splices were not load rated. For this NEPA-phase conceptual design, it is assumed that 50 percent of the connections need to be strengthened.

7 Enhanced Seismic Retrofit – Approach Spans

7.1 Enhanced Retrofit Strategy

As specified in the *EQRB Bridge Design Criteria* (Multnomah County 2021) and *EQRB Seismic Design Criteria* (Multnomah County 2021) (see Appendix A for both reports), the Project performance requirements for the design events are Full Operation after a CSZ event and Limited Operation after a 1000-year event.

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

The Enhanced Retrofit Alternative requires that some of the structure members be strengthened or replaced to improve the load rating results.

The retrofit figures shown in this chapter and Chapter 8 are conceptual illustrations. More details and dimensions can be found in the Conceptual Plans (see Appendix C).

Post-retrofit seismic analyses were performed to verify that the C/D ratios for the major elements are greater than 1.0 after the enhanced retrofit.

The Enhanced Seismic Retrofit would not reduce the existing clearance envelopes to the roads and rail tracks under the existing approach structures.

There are existing buildings that are close to or even touch the west and east approach structures. Should the Enhanced Seismic Retrofit Alternative be selected for further development, discussion with the building owners about the seismic retrofit of these buildings would be necessary.

7.2 West Approach Seismic Retrofit

7.2.1 Bridge Deck and Girders

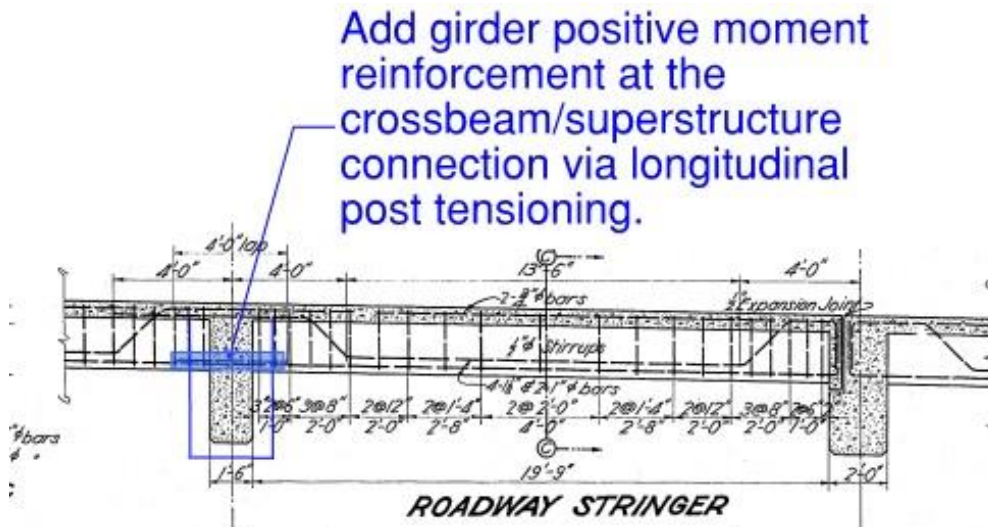
The existing bridge deck would be replaced due to the low load rating factors and the need to accommodate the proposed streetcar rail tracks.

New stringers would be added between the existing stringers where needed and would result in the following changes:

- Reduce the live load demand on the deck and stringers
- Increase the deck live load-carrying capacity
- Support the proposed streetcar tracks

At all expansion bents, replacing the existing seismic restrainers is proposed, as well as using post-tensioning and steel reinforcement to strengthen the existing positive moment stringer connections to the fixed bents (see Figure 7-1).

Figure 7-1. Girder Strengthening

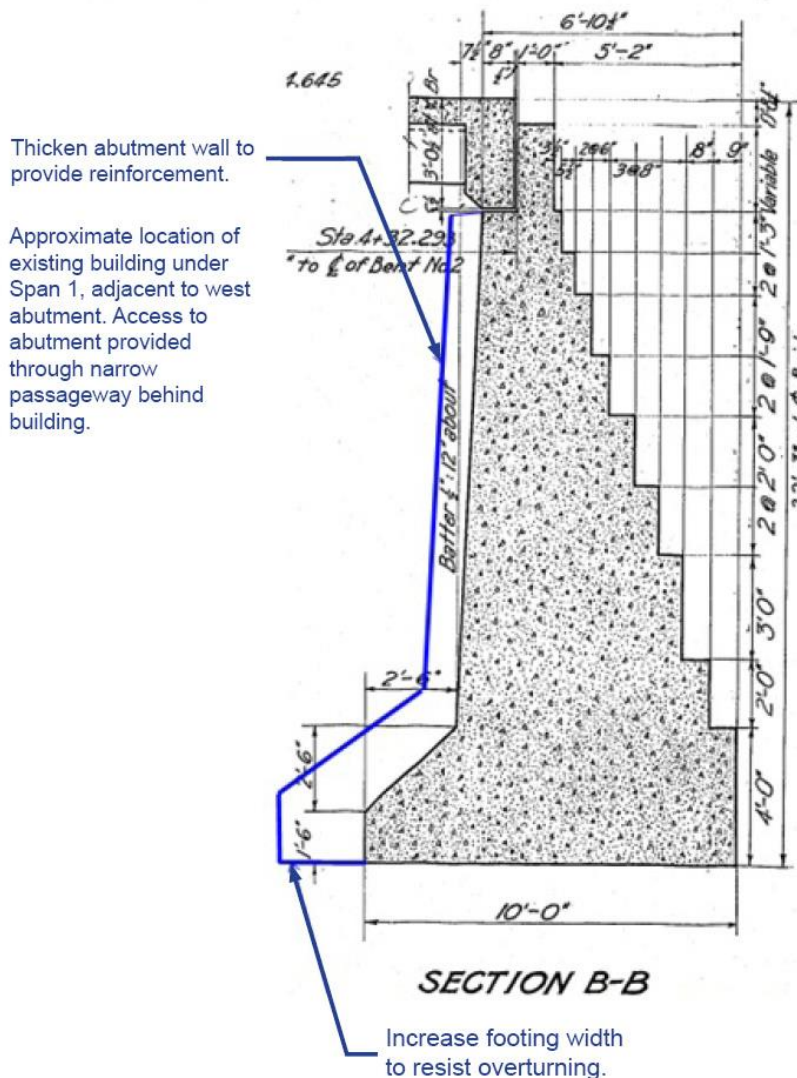


Fiber-reinforced polymer composite could be applied to the sides and bottom of the existing girders and stringers to increase their strength and durability.

7.2.2 Bent 1 (Abutment)

At Bent 1, a reinforced concrete thickening of the bent wall is proposed by drilling and doweling reinforcement into the existing wall. Increasing the footing width with a reinforced concrete section is also proposed (see Figure 7-2).

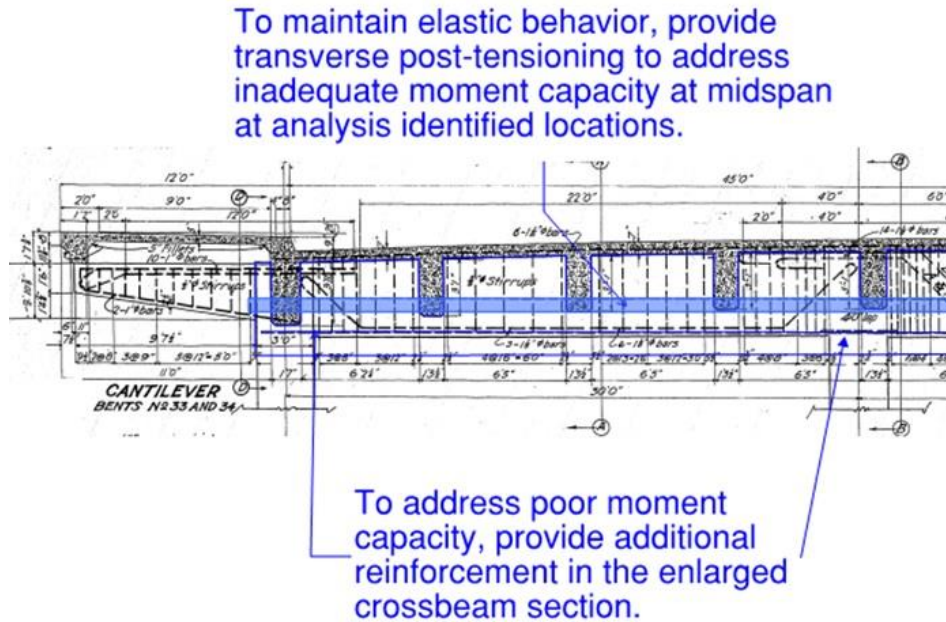
Figure 7-2. Abutment Retrofits



7.2.3 Floor Beams and Columns

At Bents 2 through 19, end floor-beam strengthening is proposed by enlarging the concrete section and adding post-tensioning. Applying steel column casing with reinforced concrete in the annulus of the casing is also proposed. The new longitudinal column reinforcement would be anchored into the floor-beam enlargement and enlarged spread footings or grade beams (see Figure 7-3).

Figure 7-3. Floor-Beam Strengthening

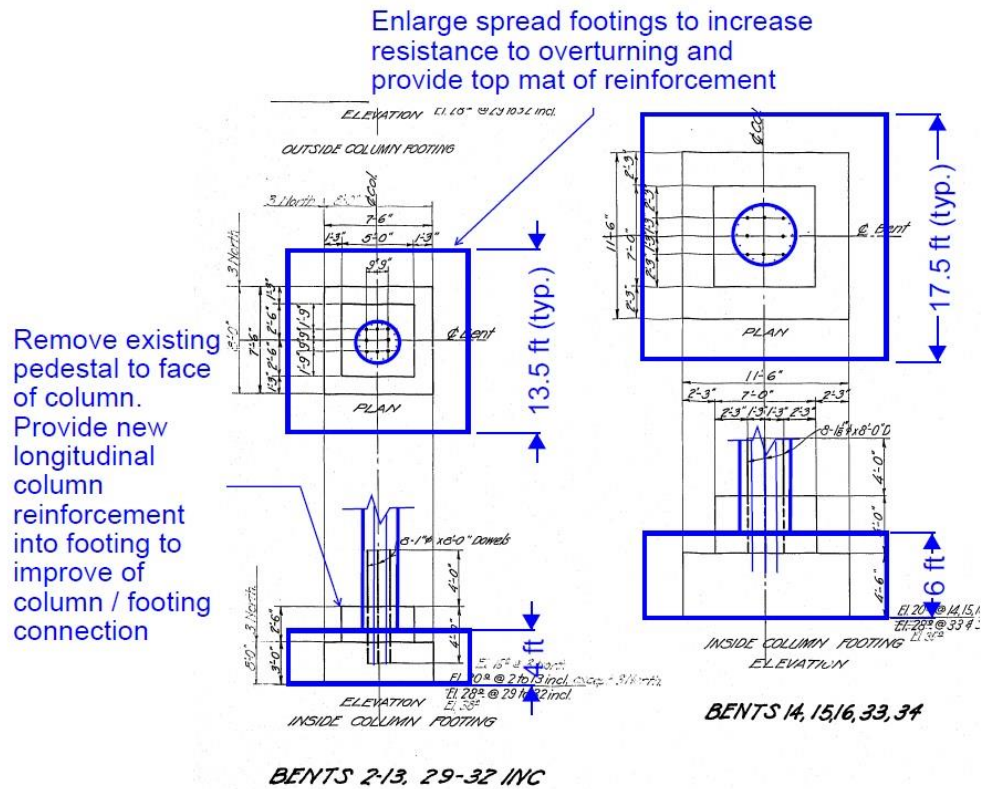


Fiber-reinforced polymer composite could be applied to the sides and bottom of the floor beams to increase their strength and durability.

7.2.4 Spread Footings

At Bents 2 through 16, enlargement of the spread footings with a reinforced concrete section is proposed by drilling and doweling reinforcement into the side of the footing. In addition, it is proposed that the footing be thickened to allow for a top mat of reinforcement and for anchorage of the new column reinforcement (see Figure 7-4).

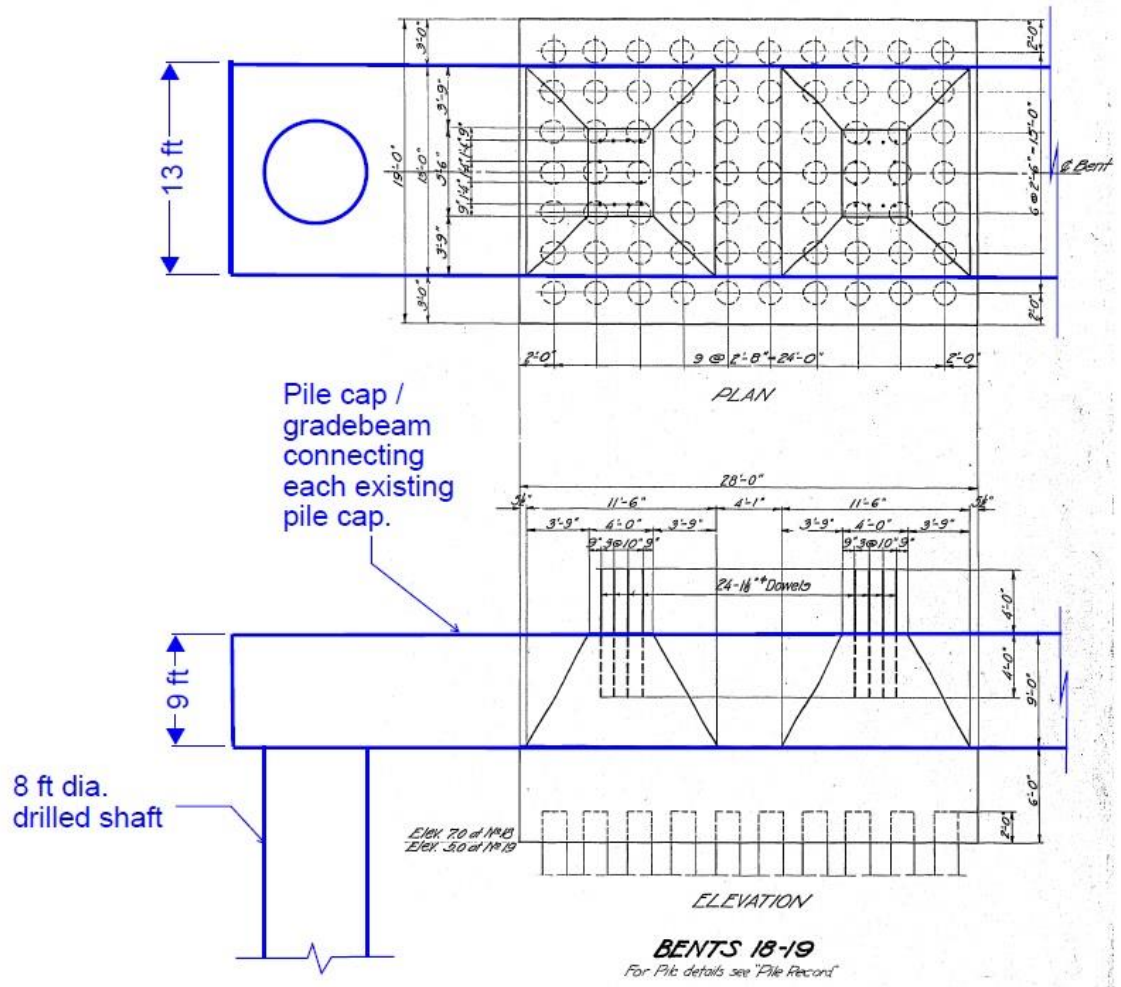
Figure 7-4. Spread Footing Enlargement



7.2.5 Pile Foundations

At Bents 17 to 19, a large post-tensioned grade beam is proposed that would carry the loads from the existing columns to new large-diameter drilled shafts constructed on each side of the existing bridge. The new shafts would be constructed outside of the existing bridge deck extents and extend through the liquefiable soil to suitable material for carrying the vertical loads (see Figure 7-5).

Figure 7-5. Pile Foundation Retrofit of Bents 18 and 19



Note: Bent 17 is similar.

7.2.6 Geotechnical Hazard Mitigation

At Bents 2 through 16, cellular soil-cement ground improvement is proposed under the footings. A zone of cellular soil-cement ground improvement is also proposed between Bent 19 and Pier 1. Geotechnical hazard mitigation is described further in the *EQRB Geotechnical Report* (Multnomah County 2021) (Appendix A).

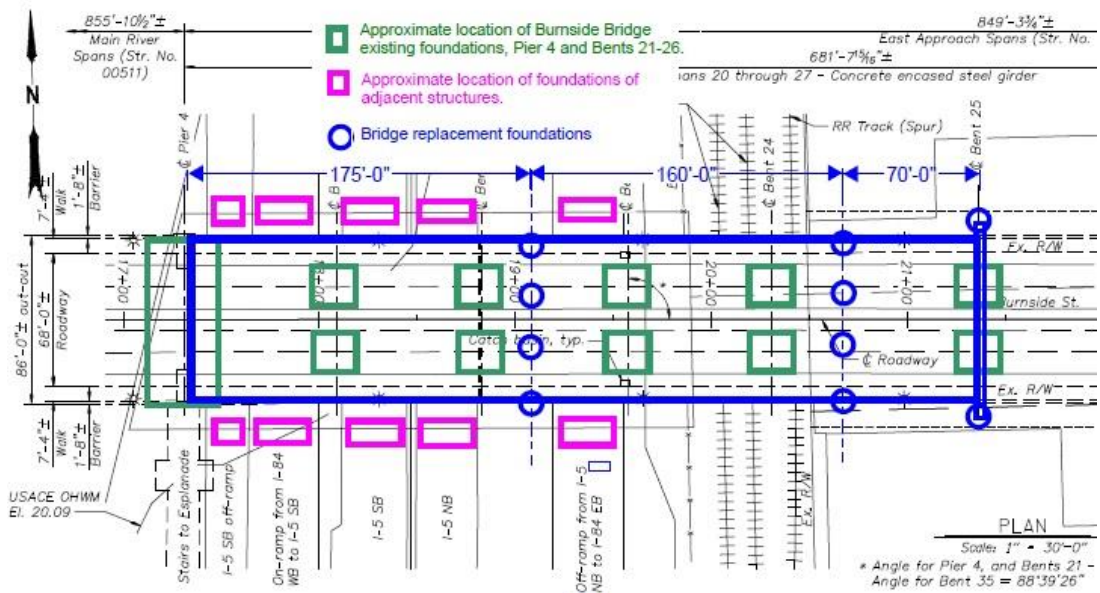
7.3 East Approach Seismic Retrofit

7.3.1 Spans 20 to 24 Replacement

Because of the anticipated challenges for shutting down I-5 for an extended period of time, as well for gaining construction access near the UPRR tracks, replacing Spans 20 to 24 of the east approach with a longer span structure could be more cost effective and practical than retrofitting the existing piers and foundations. Replacing the spans would reduce the length of time I-5 is closed, as well as shorten the period in which the trains would need to be rescheduled.

It is proposed that existing Spans 20 to 24 be replaced with a three-span steel-plate girder structure on modern reinforced concrete bents. The bents would be supported by large-diameter drilled shafts that extend through the liquefiable material to suitable material for carrying vertical loads (see Figure 7-6).

Figure 7-6. Existing Spans 20 to 24 Replacement



7.3.2 Bridge Deck and Girders

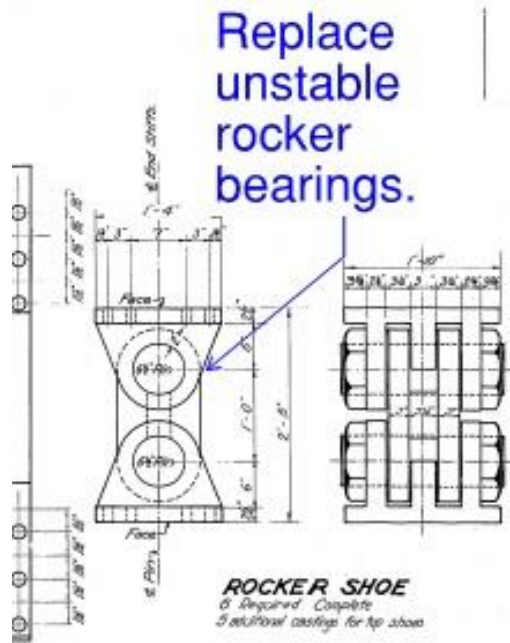
The existing bridge deck would be replaced due to the low load rating factors and the need to accommodate the proposed streetcar alignments.

New stringers would be added between the existing stringers where needed and would result in the following changes:

- Reduce the live load demand on the deck and stringers
- Increase the deck live load-carrying capacity
- Support the proposed streetcar tracks

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

Figure 7-7. Rocker Bearing Replacement



Fiber-reinforced polymer composite could be applied to the sides and bottom of the existing reinforced concrete deck girders and stringers to increase their strength and durability.

7.3.3 Floor Beams and Columns

At Bents 25 to 28, strengthening the concrete-encased steel end floor beams is proposed where the main girders tie into the end floor beams as well as at the column-to-floor beam connection (see Figure 7-8). Strengthening of the concrete-encased steel columns and cross bracing is proposed along with the adding a partial height-reinforced concrete infill wall to strengthen the bents (see Figure 7-9).

Figure 7-8. Column to Floor-Beam Strengthening

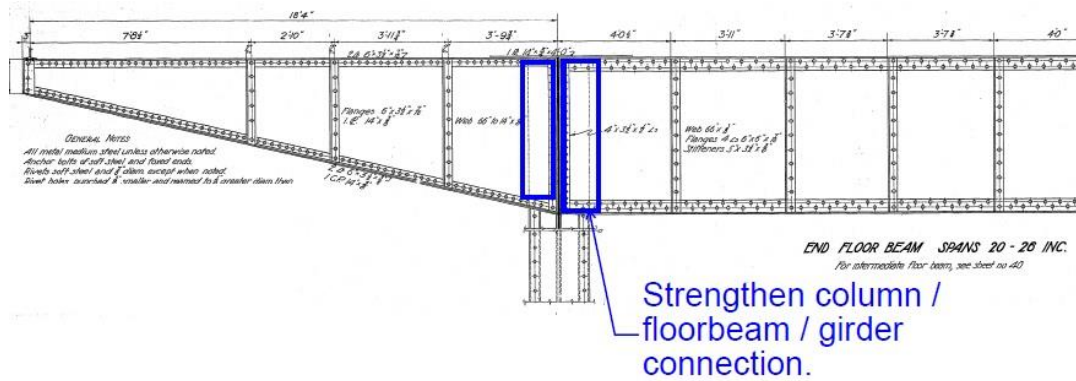
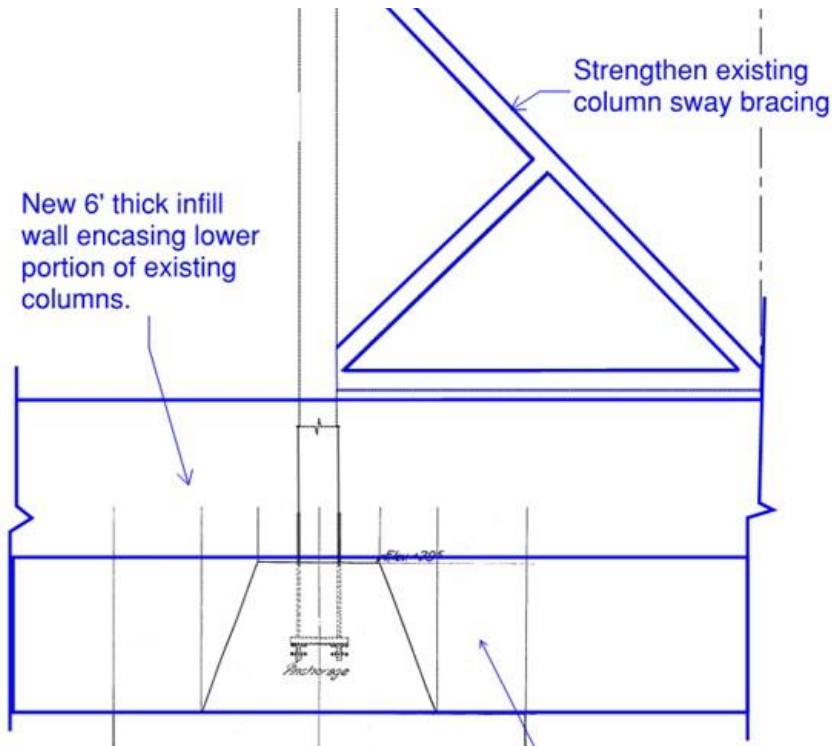


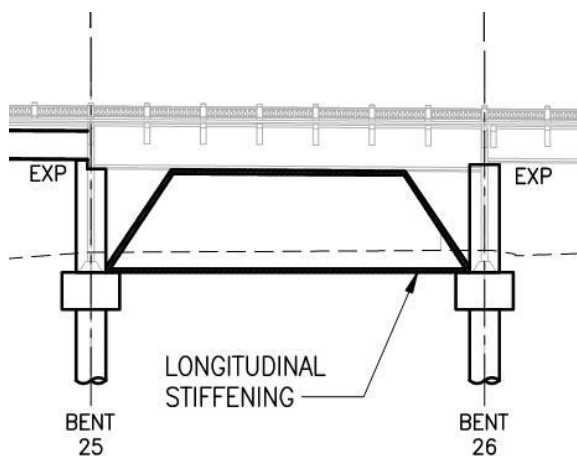
Figure 7-9. Infill Wall



At Bents 29 through 34, end floor-beam strengthening is proposed by enlarging the concrete section and adding post-tensioning (see Figure 7-3). Applying steel column casing with additional reinforced concrete in the annulus of the casing is also proposed. The new longitudinal column reinforcement would be anchored into the floor-beam enlargement and enlarged spread footings (see Figure 7-4). Fiber-reinforced polymer composite could be applied to the sides and bottom of the existing floor beams to increase their strength and durability.

Longitudinal bracing between Bents 25 and 26 is proposed to provide the additional longitudinal stiffness needed (for the bridge) that cannot be addressed by the adjacent bent and foundation retrofits (see Figure 7-10).

Figure 7-10. Longitudinal Bracing



7.3.4 Bent 35 (Abutment)

At Bent 35, a reinforced concrete thickening of the bent wall is proposed by drilling and doweling reinforcement into the existing wall. Increasing the footing width with a reinforced concrete section is also proposed (see Figure 7-2).

7.3.5 Spread Footings

At Bents 28 to 34, enlargement of the spread footings with a reinforced concrete section is proposed by drilling and doweling reinforcement into the side of the footing. In addition, it is proposed that the footing be thickened to allow for a top mat of reinforcement and to allow for anchorage of the new column reinforcement (see Figure 7-4).

7.3.6 Pile Foundations

At Bents 25 to 27, a large post-tensioned grade beam is proposed that would carry the loads from the existing columns to new large-diameter drilled shafts constructed on each side of the existing bridge. The new shafts would be constructed outside of the existing

bridge deck extents and extend through the liquefiable soil to suitable material for carrying the vertical loads (see Figure 7-5).

7.3.7 Geotechnical Hazard Mitigation

A zone of cellular soil-cement ground improvement is proposed between Bents 23 and 24. Geotechnical hazard mitigation is described further in the *EQRB Geotechnical Report* (Multnomah County 2021) (Appendix A).

7.4 Constructability

The constructability of the enhanced retrofits described above can be found in the *EQRB Construction Approach Technical Report* (Multnomah County 2021) (Appendix A).

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8 Enhanced Seismic Retrofit – Main River Spans

8.1 Steel Truss Fixed-Span Enhanced Retrofit

The analysis was conducted to verify the proposed retrofit/rehabilitation schemes and effectiveness. Figure 8-1 and Table 8-1 below show the member C/D ratios under a CSZ earthquake event after the enhanced retrofit.

Figure 8-1. Top Laterals and Floor System; Sway Bracing

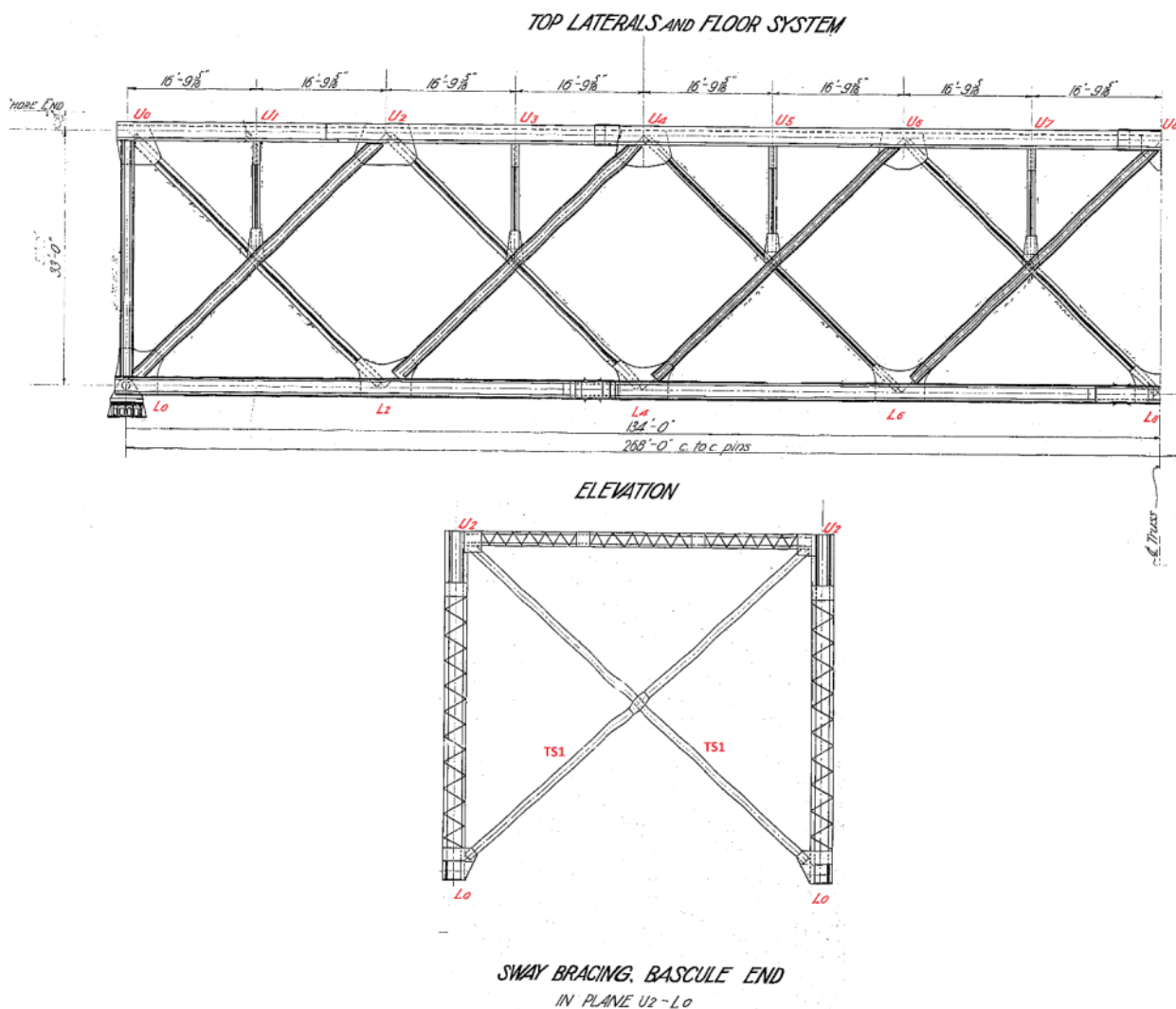


Table 8-1. C/D Ratio Post-Enhanced Retrofit of Fixed Spans

Drawing Member #	Member Location	Failure Mode	C/D Ratio
U8U9	Top Chord at Midspan	Compression	1.52
L8L9	Bottom Chord at Midspan	Tension Yield	1.10
L8L9	Bottom Chord at Midspan	Tension Fracture	1.70
U8L9	Diagonal Near Midspan	Compression	1.12
TS1	Out-of-Plane Bracing Near Bascule	Compression	1.14
TS1	Out-of-Plane Bracing Near Bascule	Tension Yield	3.27
TS1	Out-of-Plane Bracing Near Bascule	Tension Fracture	3.47
U16L16	End Post Near Bascule	Compression	1.94
	Fixed End Support Anchor Bolts	Shear	1.07

8.1.1 Pier 1 Retrofit

8.1.1.1 Pier 1 Foundations

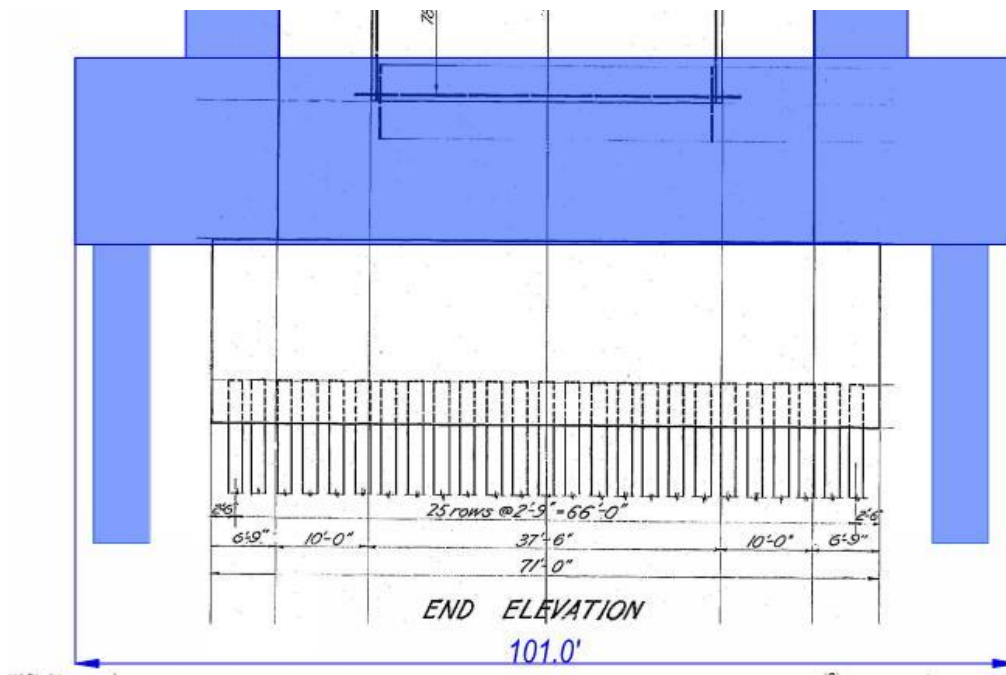
Because of the multiple potential failures identified in Chapter 5, the Pier 1 foundation would need to be retrofitted.

Drilled Shafts on Both Sides of the Existing Pile Cap

New drilled shafts are proposed on either side of the existing pile cap (see Figure 8-2) for the following purposes:

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

Figure 8-2. Pier 1 Foundation Enlargement



Note: Blue shading in figures indicates retrofit improvements.

Pile Cap Enlargement (Extension)

A new pile cap would be constructed on top of the existing concrete pile caps, and the new cap would be extended to cover the new drilled shafts on the sides of the existing foundations at Pier 1. The enlarged pile caps would be connected to the new drilled shafts and also connected to the existing pile cap and wall by using dowel bars and post-tensioning. More details can be seen on the Conceptual Plans (Appendix C).

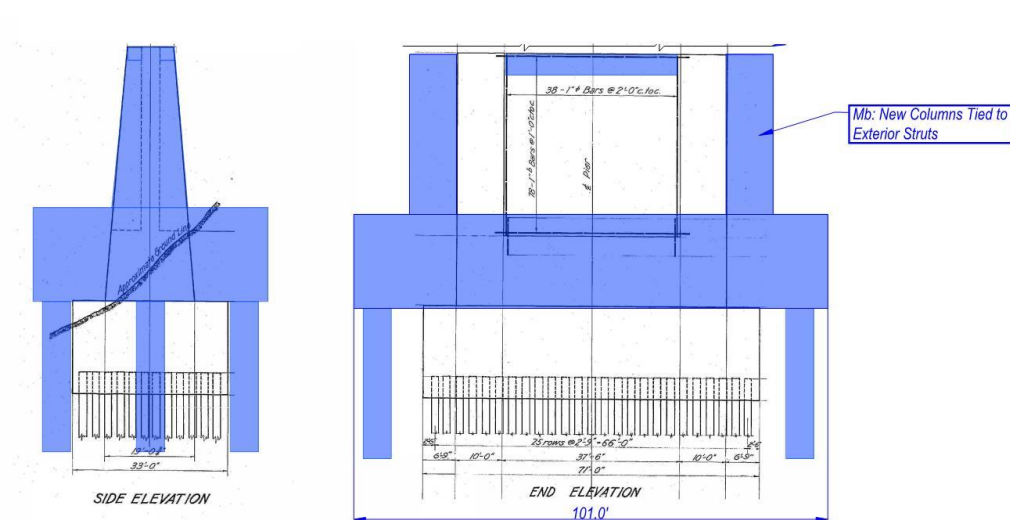
The Harbor Wall adjacent to Pier 1 is likely to be reconstructed because the enlarged pier footing and new shafts may conflict with the battered piles of the Harbor Wall, for construction access, and because the proposed soil mitigation may displace the timber piles of the Harbor Wall.

8.1.1.2 Pier 1 Column/Wall

The Pier 1 column/wall would be strengthened to conform to the seismic design requirements.

The pier columns would be enlarged. The enlarged portions of the columns would have adequate reinforcement to meet the seismic force demand and to provide needed ductility (see Figure 8-3). More details can be seen on the Conceptual Plans (Appendix C).

Figure 8-3. Pier 1 Strengthening



8.1.2 Pier 4 Replacement

8.1.2.1 Pier 4 Foundations

At Pier 4, an I-5 southbound off-ramp is immediately adjacent to the existing Pier 4 columns and is above the east portion of the existing foundation, which makes construction access extremely difficult. In addition, retrofitting the existing footing would require partially removing the elevated I-5 ramp structure. According to ODOT, that is not an allowed option.

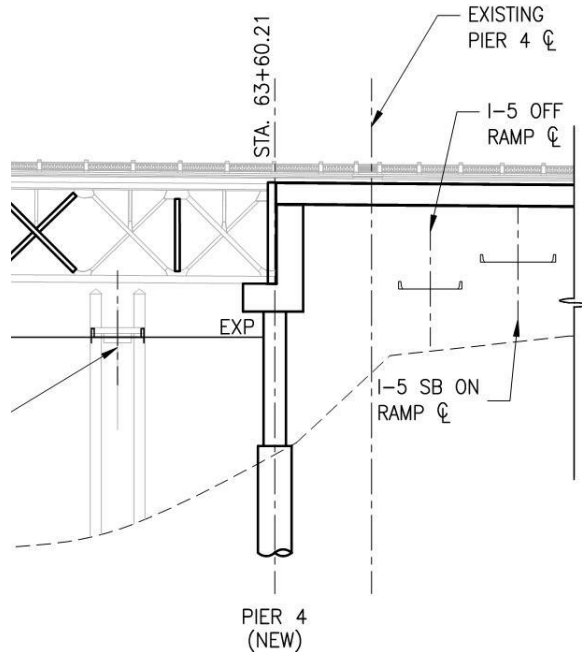
The enhanced retrofit would replace the existing Pier 4 with a new pier approximately 34 feet to the west to avoid the constructability restrictions.

The new Pier 4 would be supported on two 10-foot-diameter shafts.

8.1.2.2 Pier 4 Columns

The enhanced retrofit design would replace the existing Pier 4 with a new pier that consists of a super-bent crossbeam supported on two columns, see Figure 8-4. More details can be seen on the Conceptual Plans (Appendix C).

Figure 8-4. Pier 4 Replacement



8.1.3 Piers 2 and 3

Enhanced retrofit actions for Pier 2 and Pier 3 are described in the Bascule Span section, Section 8.2.

8.1.4 Truss Supports and Pier Connections

8.1.4.1 Replace Rocker-Type Expansion Bearings

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

8.1.4.2 Retrofit Fixed Bearings

Under the Enhanced Retrofit Alternative, the anchor bolts under the steel truss span fixed bearings at Pier 2 and Pier 3 would be replaced to meet the shear strength requirement for seismic loads. The pier concrete under the bearings would be widened with added reinforcing to prevent concrete splits and cracks. These retrofits would increase the C/D ratio to equal or greater than 1.0.

8.1.4.3 Seating Length Extension

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

8.1.4.4 Retrofit Transverse Restrainers

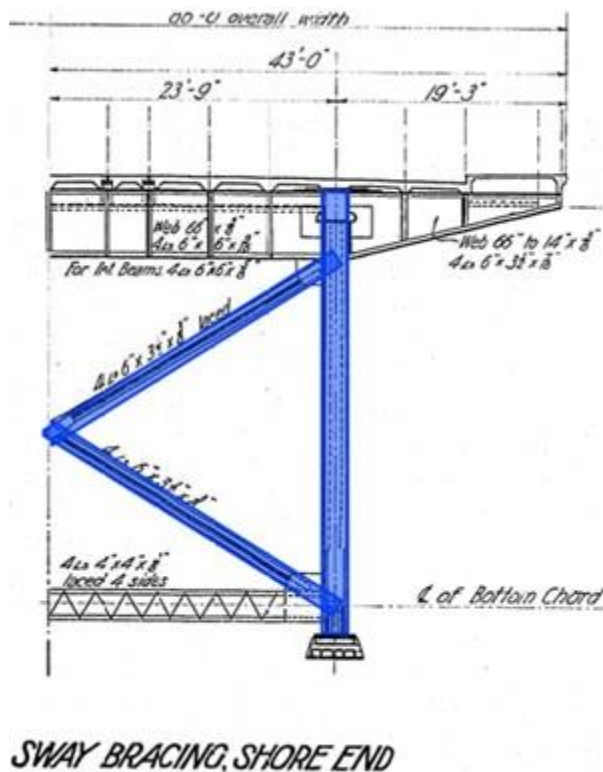
As part of the pier column/wall retrofit, the pier columns or walls at Piers 1, 2, 3, and 4 would need to be strengthened and reinforced. These retrofitted pier columns or walls would provide effective lateral resistance to the truss supports.

8.1.5 Steel Trusses

8.1.5.1 Strengthen Lateral Bracings

The steel trusses were not designed for seismic loads and movements, therefore adding or strengthening the lateral load-carrying members would be necessary. The members that would need to be added or strengthened include diagonal sway bracings and connected vertical members (see Figure 8-5), as well as bottom lateral bracings. At minimum, the bracing members at the two bays near each span support would be strengthened.

Figure 8-5. Sway Bracing Strengthening Required



8.1.5.2 East Steel Truss Span Modifications

Because the replacement Pier 4 would be approximately 34 feet west of the existing Pier 4, the existing steel truss would be shortened, and the members near the east end would be strengthened to support the truss on the bearings. More details can be seen on the Conceptual Plans (Appendix C).

8.1.5.3 Strengthen Deck to Floor-beam Connections

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

8.1.6 Bridge Deck

The existing bridge deck would be replaced due to the low load rating factors and the need to accommodate the proposed streetcar alignments.

New stringers would be added between the existing stringers where needed and would result in the following changes:

- Reduce the live load demand on the deck and stringers
- Increase the deck live load-carrying capacity
- Support the proposed streetcar tracks

8.2 Bascule Span Enhanced Retrofit

8.2.1 Enhanced Retrofit Strategy

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

Because a bascule bridge structure consists of many rigid elements and links and has limited ductility, a seismic retrofit strategy using base isolation technology has been discussed in previous Project reports and is evaluated in more detail in Section 8.2.5.

In order to meet the operational performance requirements described in the *EQRB Seismic Design Criteria* (Multnomah County 2021) (Appendix A), the bridge seismic retrofit would not only improve the C/D ratios to meet the strength requirements, but would also limit the displacements. This requires the analysis to be conducted step by step to identify the required retrofit. For example, eliminating the displacement range to meet the operational performance requirements can result in increased force demands on members and reduce the member C/D ratios.

Unlike a conventional fixed-span bridge that relies on structural ductility to allow movement during a seismic event thus reducing the seismic demand forces, a bascule

bridge has very strict displacement restrictions. In order for the bascule span be operational after a design-level earthquake, the bascule leaf mechanical drive gears and pinions have to be functional. Although the exact displacement upper limits are difficult to quantify at this conceptual level of study, the following are estimated displacement limits between the gear racks and the pinions:

- Transverse 0.25 inch
- Along the centerline of the rack and the pinion < 1/100 inch

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

Figure 8-6 and Table 8-2 indicate the CSZ-level displacements at key locations after the enhanced retrofit improvements. In Table 8-2, X corresponds to the longitudinal direction, Y corresponds to the transverse direction, and Z corresponds to the vertical direction.

Table 8-2. Displacements at Key Points Post-Enhanced Retrofit

Model Node #	Node Location	Direction	Full Operation EQ		Limited Operation EQ	
			Displacement (in)	Relative Displacement to Top of Pedestal (in)	Displacement (in)	Relative Displacement to Top of Pedestal (in)
20611	Operating Pinion connection To Trunnion Support	X	6.1	0.1	7.8	0.1
		Y	5.0	0.5	6.4	0.6
		Z	-4.0	0.0	-4.0	0.0
20545	Top of Trunnion Support at Trunnion Pin	X	6.2	0.2	7.9	0.1
		Y	5.7	1.1	7.3	1.5
		Z	-4.1	-0.1	-4.2	-0.2
20451	Counterweight Mass	X	6.2	0.2	8.0	0.2
		Y	6.6	2.1	8.6	2.8
		Z	-4.5	-0.5	-4.5	-0.5
20716	Live Load Shoe	X	6.1	0.1	7.8	0.1
		Y	4.8	0.3	6.2	0.4
		Z	-4.0	-0.1	-4.1	-0.1
20914	Bascule Tip	X	6.3	0.3	8.1	0.4
		Y	7.7	3.2	9.9	4.1
		Z	-9.3	-5.3	-9.5	-5.5
134	Bottom of Pile Cap	X	5.4	-0.6	7.0	-0.8
		Y	4.1	-0.4	5.3	-0.5
		Z	-3.9	0.1	-3.9	0.1
20503	Top of Pedestal	X	6.0	N/A	7.8	N/A
		Y	4.5	N/A	5.8	N/A
		Z	-4.0	N/A	-4.0	N/A

Table 8-3 indicates that member C/D ratios are improved after the enhanced retrofit improvements.

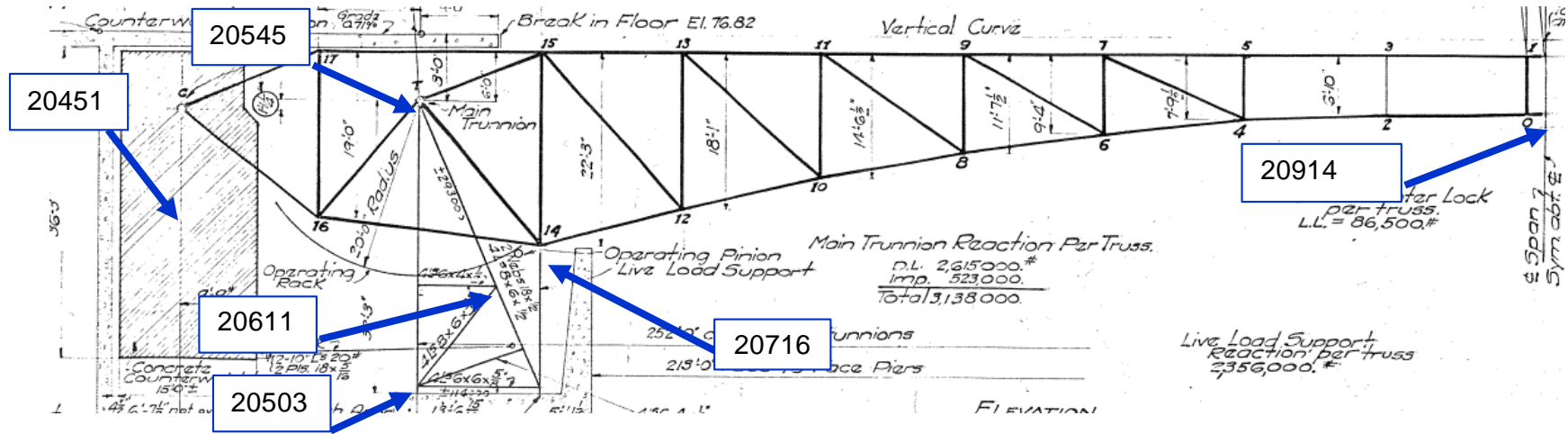
Table 8-3. C/D Ratio (CSZ) Summary After Enhanced Retrofit of Bascule Span

Drawing Member #	Member Location	Failure Mode	C/D Ratio
16-C	Counterweight Compression Support	Comp.-Moment Interaction*	1.04
16-C	Counterweight Compression Support	Shear	1.11
Counterweight Link	Counterweight Link	Compression	1.32
Counterweight Link	Counterweight Link	Tension Yield	2.78
Counterweight Link	Counterweight Link	Tension Fracture	3.04
C1	Trunnion Post	Compression	1.66
-	Trunnion Support Anchor Bolts	Combined Tension and Shear	1.12
14-T	Trunnion Brace	Comp.-Moment Interaction*	1.04
14-T	Trunnion Brace	Shear	2.95
14-T	Trunnion Brace	Ten.-Moment Interaction*	1.06
15-T	Trunnion Link	Comp.-Moment Interaction*	1.46
15-T	Trunnion Link	Shear	1.10
15-T	Trunnion Link	Ten.-Moment Interaction*	1.01
14-15	Bottom Chord Bascule Truss	Compression	1.18
12-15	Diagonal Bascule Truss	Tension Yield	1.84
12-15	Diagonal Bascule Truss	Tension Net Fracture	2.47
13-15	Top Chord Bascule Truss	Tension Yield	1.15
13-15	Top Chord Bascule Truss	Tension Net Fracture	1.62

* For interaction failure, the C/D ratio shown represents 1/(interaction result)

The member designation in the table, such as 16-C, denotes this member connects node 16 to node C, see Figure 8-6.

Figure 8-6. Nodes Where Displacement was Checked



8.2.2 Bascule Pier Foundations

8.2.2.1 Bascule Pier Foundation Enlargements

Because of the multiple potential failures identified in Chapter 5, the bascule pier foundations would need to be retrofitted to meet the operational performance requirements in the *EQRB Seismic Design Criteria* (Multnomah County 2021) (Appendix A).

Replacing the existing unreinforced pile caps wholesale and placing additional piles under the existing structures while minimizing the impacts to the bridge superstructure would be expensive and have a significant impact on traffic. Therefore, instead of replacing the existing foundations, the existing foundations would be enlarged and strengthened.

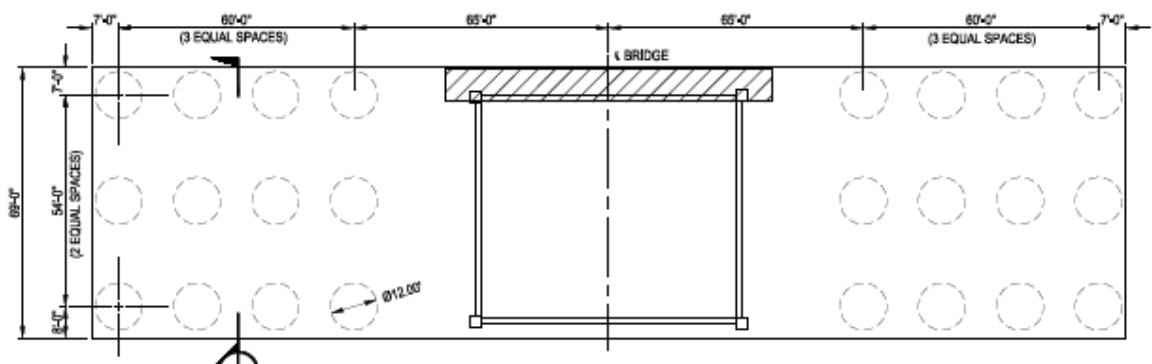
Because of the strict requirement of not reducing the existing navigational channel width, the foundation enlargement could only be widened transversely to the bridge.

Drilled Shafts on Both Sides of the Existing Pile Caps

New drilled shafts are proposed on both sides of the existing pile caps (see Figure 8-7) for the following purposes:

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

Figure 8-7. Bascule Pier 2 and 3 Footing Enlargement



Pile Cap Enlargement (Extension)

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

8.2.2.2 Maintain Navigation Channel Clearance

Because Pier 2 and Pier 3 are adjacent to the river navigational channel, the foundation pile cap enlargement on the navigation channel side would be restricted to match with the existing pile caps.

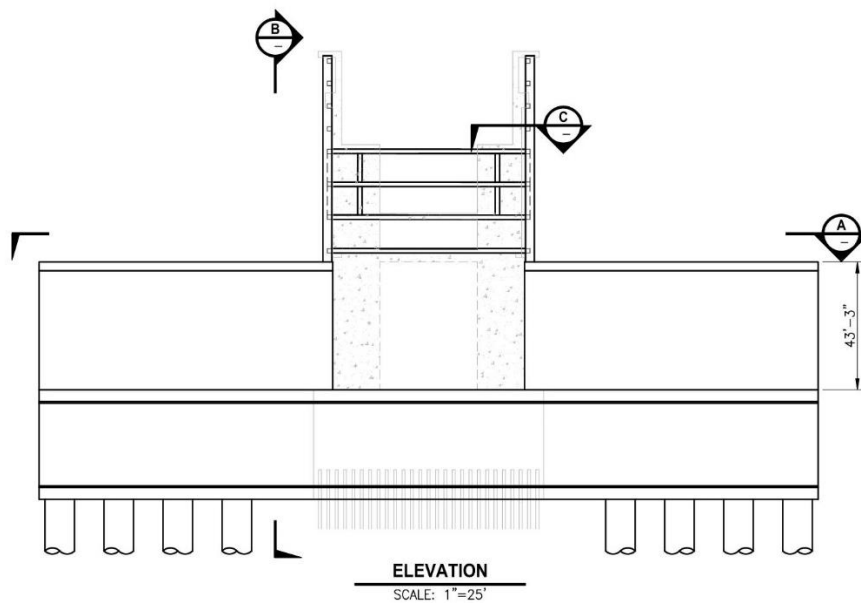
8.2.3 Bascule Pier Walls

The pier walls at bascule Pier 2 and Pier 3 would be strengthened to conform to the seismic design requirements. The proposed strengthening is described below.

8.2.3.1 New Columns

A new column would be constructed at each pier corner to act as lateral load-carrying members that transfer the seismically induced lateral loads from the bridge deck and trunnion support structures to the new extended foundations (see Figure 8-8). These corner pier columns would be integral with the existing pier walls and have an adequate amount of reinforcement to meet the seismic force demand and to limit the displacement. The bottom of each column would have dowel bars embedded in the pile caps with sufficient embedment length to resist potential uplifting forces during a design-level seismic event.

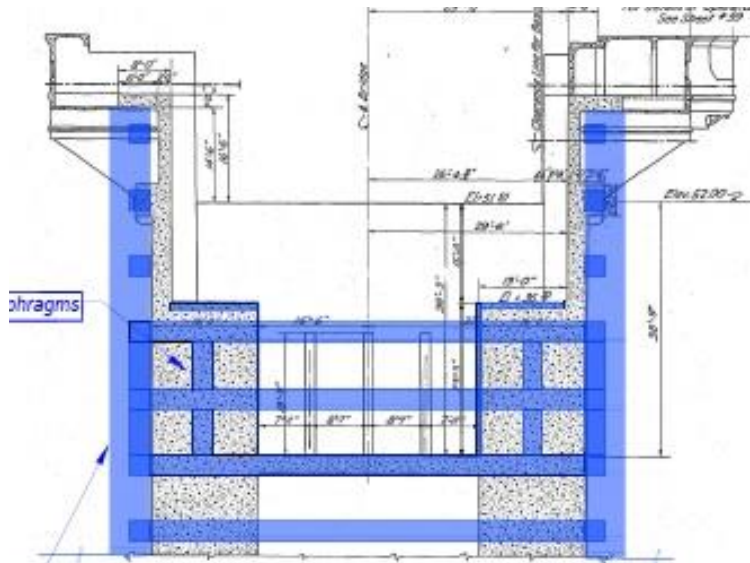
Figure 8-8. Bascule Pier Retrofit with Columns and Horizontal Struts



8.2.3.2 Horizontal Struts

Horizontal struts would be constructed to connect the new corner columns, providing confinement while strengthening the pier walls (see Figure 8-9).

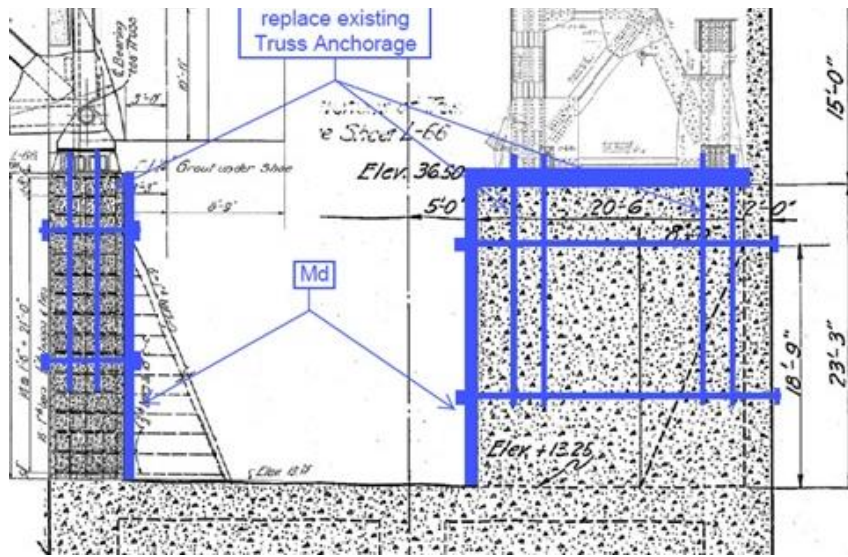
Figure 8-9. Bascule Pier Retrofit with Horizontal Struts



8.2.3.3 Confinements around the Concrete Pedestals and Supports

This Alternative would provide confinement around the concrete pedestals under the trunnion support frames and fixed truss span supports to prevent the concrete from cracking, because the trunnion support frames and the steel fixed truss will exert a huge amount of vertical and horizontal force onto these pedestals and supports during a design-level seismic event (see Figure 8-10).

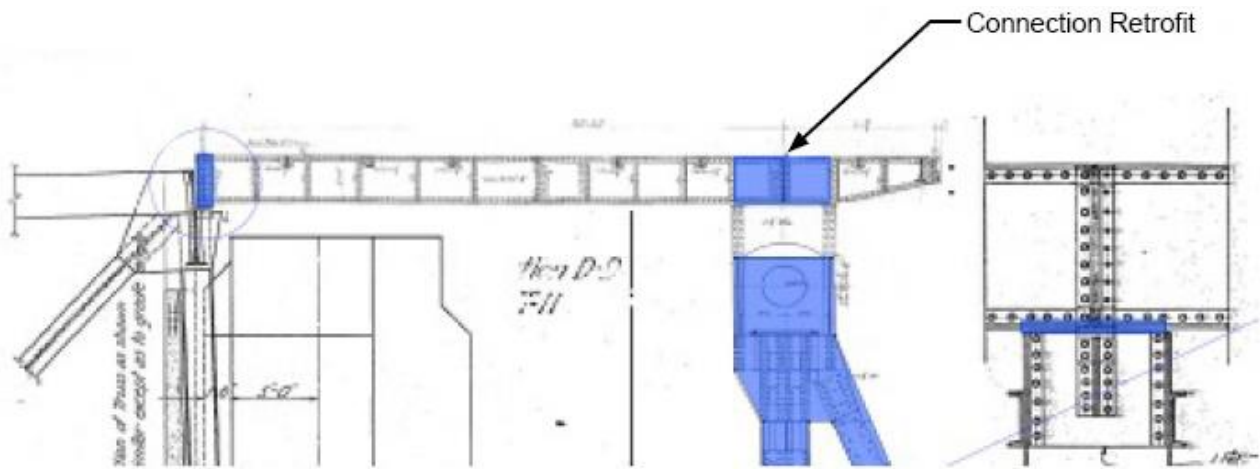
Figure 8-10. Confinement to Concrete



8.2.3.4 Pit Deck Girder Connections

Project improvements would strengthen the connections between the pier pit deck girders and the pier back walls, as well as the connections between the deck girders and the trunnion support frames (see Figure 8-11).

Figure 8-11. Connection Strengthening at Pit Deck Girder Supports



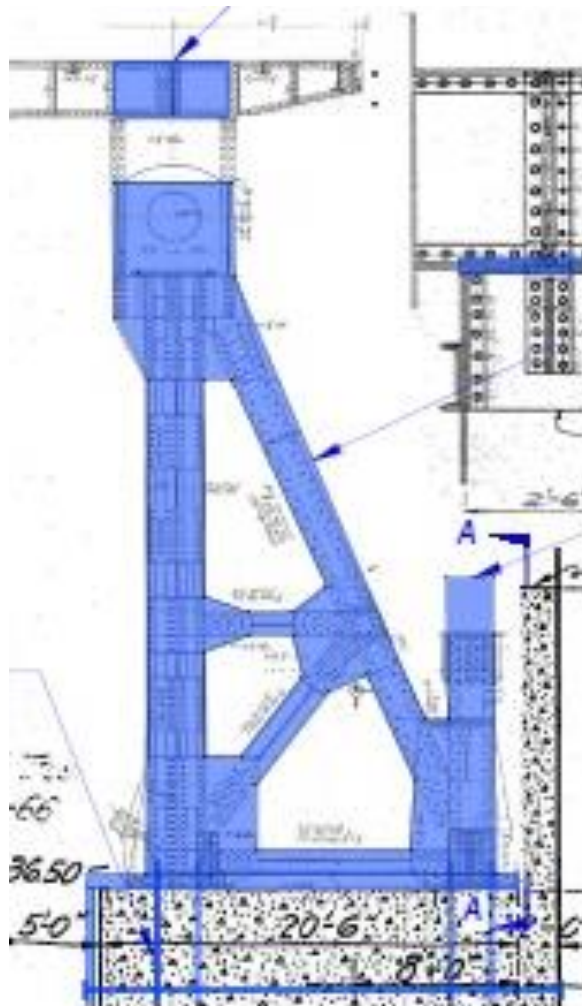
This strengthening would not only prevent the pit deck from falling into the bascule pier pit, but would also provide a horizontal load path that transfers the horizontal load from the top of the trunnion support frames to the pier back wall and corner columns, then down to the foundation level.

8.2.4 Bascule Leaf Trunnion Supports

8.2.4.1 Trunnion Tower Support Frames

In addition to the foundation retrofit and installation of lateral restrainers to the pier walls, the trunnion tower support frames would be strengthened (see Table 8-3) to prevent buckling and to provide adequate support to the bascule leaves and the counterweights during a design-level seismic event (see Figure 8-12).

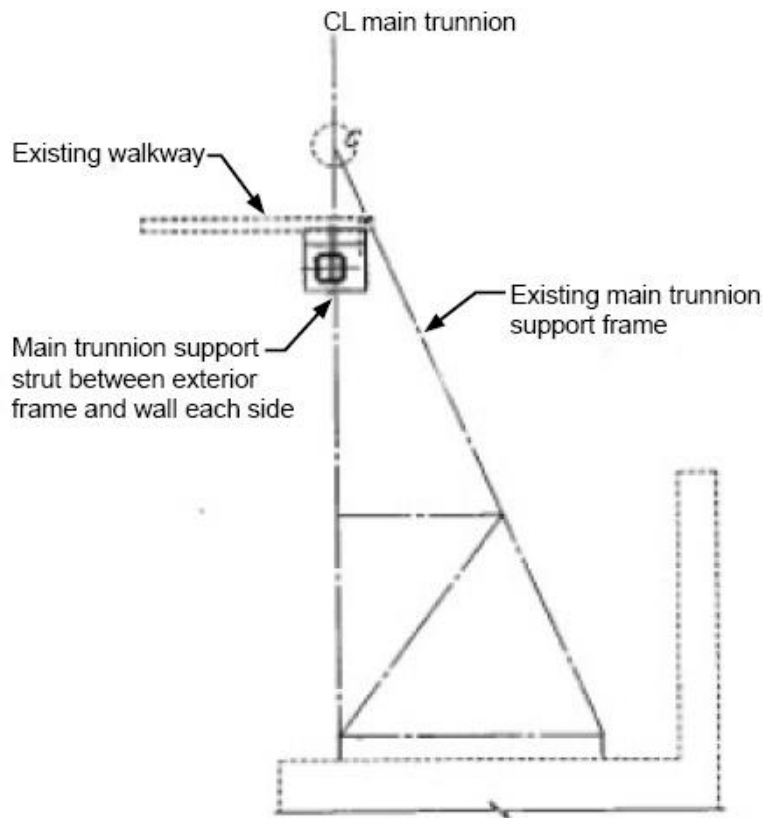
Figure 8-12. Strengthening the Trunnion Tower Support Frame



8.2.4.2 Lateral Restrainers

The lateral restrainers installed in 2005 would be replaced as part of the pier wall strengthening. The replaced restrainers would be connected to the retrofitted walls with strut reinforcements that are capable of transferring the lateral load down to the foundations via the corner columns. These retrofitted restrainers would provide longitudinal and transverse restraint to the trunnion support frames to prevent them from buckling or tipping over (see Figure 8-13).

Figure 8-13. Install New Lateral Restrainers



8.2.4.3 Anchor Bolts

Existing anchor bolts would be replaced with larger anchor bolts to resist the design-level seismic forces. The embedment depth into the strengthened concrete pedestal below should also be deeper than with the existing condition (see Figure 8-14). Comparing the C/D ratios in Table 5-2 and Table 8-3, the installation of lateral restrainers reduces the load on the anchor bolts, and the C/D ratio for the combined tension and shear is improved. Replacing the existing anchor bolts is also required to improve the C/D ratios of the anchor bolts.

Figure 8-14. Replace Existing Anchor Bolts



8.2.4.4 Trunnion Frame Connections to Pit Deck

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

8.2.5 Feasibility and Application of Base Isolation

8.2.5.1 Feasibility

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

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

Unfortunately, the existing bascule span structure lacks both of these conditions, respectively, as described below.

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

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

system together with the trunnion support system would lead to redesigning, rearranging, and replacing the entire machinery system.

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

8.2.5.2 Base Isolation Evaluation

Enhanced Retrofit-Only Alternative

Base isolation would not be applied to the bascule structure due to the abovementioned concerns.

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

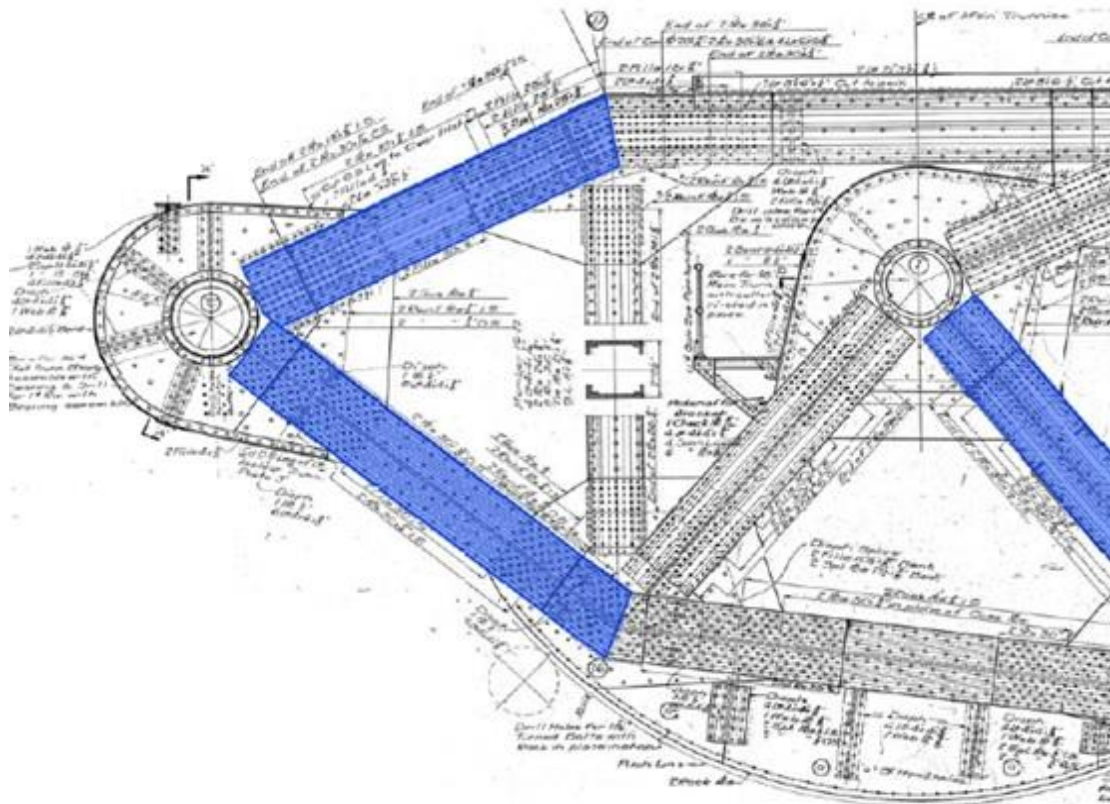
The retrofit strategies described in this report do not incorporate base isolation technology. The possibility of applying base isolation is more suitable for the study of replacement alternatives.

8.2.6 Counterweight Supports

8.2.6.1 Counterweight Support Frames

The counterweight support frames would be strengthened to prevent buckling and to provide adequate support to the counterweight during a design-level seismic event (see Figure 8-15). Strengthening the counterweight support frames is an integral part of the bascule structure enhanced retrofit.

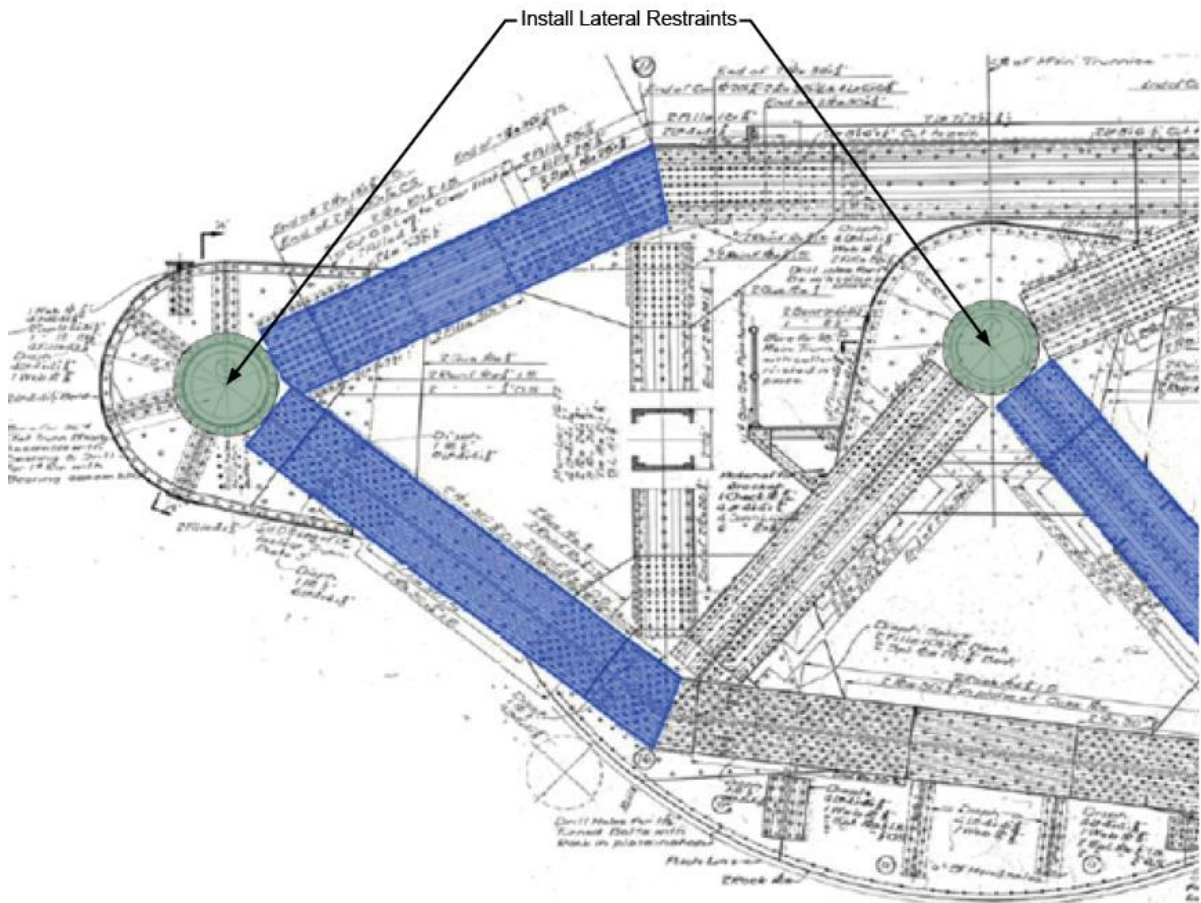
Figure 8-15. Counterweight Support Frame



8.2.6.2 Lateral Restrainers

Lateral restrainers would be installed on counterweight frames and the pier walls to prevent the counterweight from unrestrained sway. Two sets of restrainers are required: one at a position when the bascule span is closed, and the other at a position when the bascule span is fully open (see Figure 8-16).

Figure 8-16. Seismic Lateral Restrainer Locations



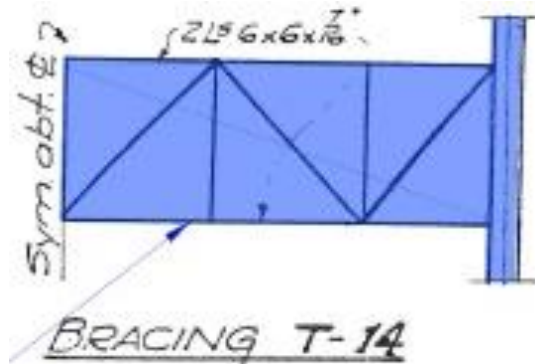
8.2.6.3 Counterweight Link

The counterweight link resisting motion of the counterweight along the longitudinal direction of the bridge will be strengthened or replaced. With the additional retrofits, the counterweight link increases its C/D ratio to 1.32.

8.2.7 Superstructure Connection to Trunnion

Since the bracing frame T-14 (see Figure 8-17) transmits lateral loads from the entire bascule span to the trunnion support frame, these members will be strengthened to prevent them from buckling and yielding. The members of T-14 and T-15 shall be reinforced to bring the C/D ratio to above 1. Post-retrofit analysis indicated the C/D ratios after the seismic retrofit increased to 1.04 or higher

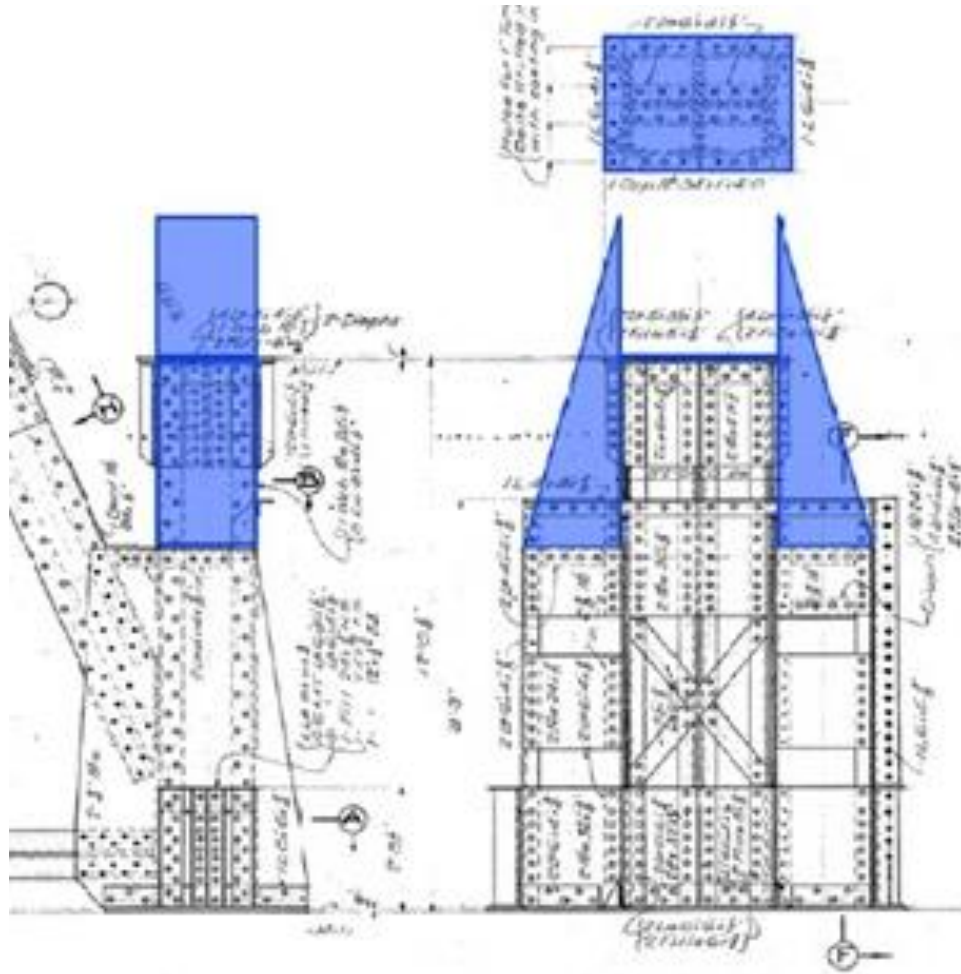
Figure 8-17. Lateral Bracing at Trunnion Support Frame



8.2.8 Live Load Support Connections

The live load support shoes would be retrofitted to provide lateral restraint to the bascule leaves. By providing three lateral restraining points to the bascule leaf—at the trunnion, at the counterweight, and at the live load shoes (see Figure 8-18)—it could more effectively reduce the horizontal sway of the bascule leaf.

Figure 8-18. Retrofit of Live Load Shoes



8.2.9 Pit Deck Supports

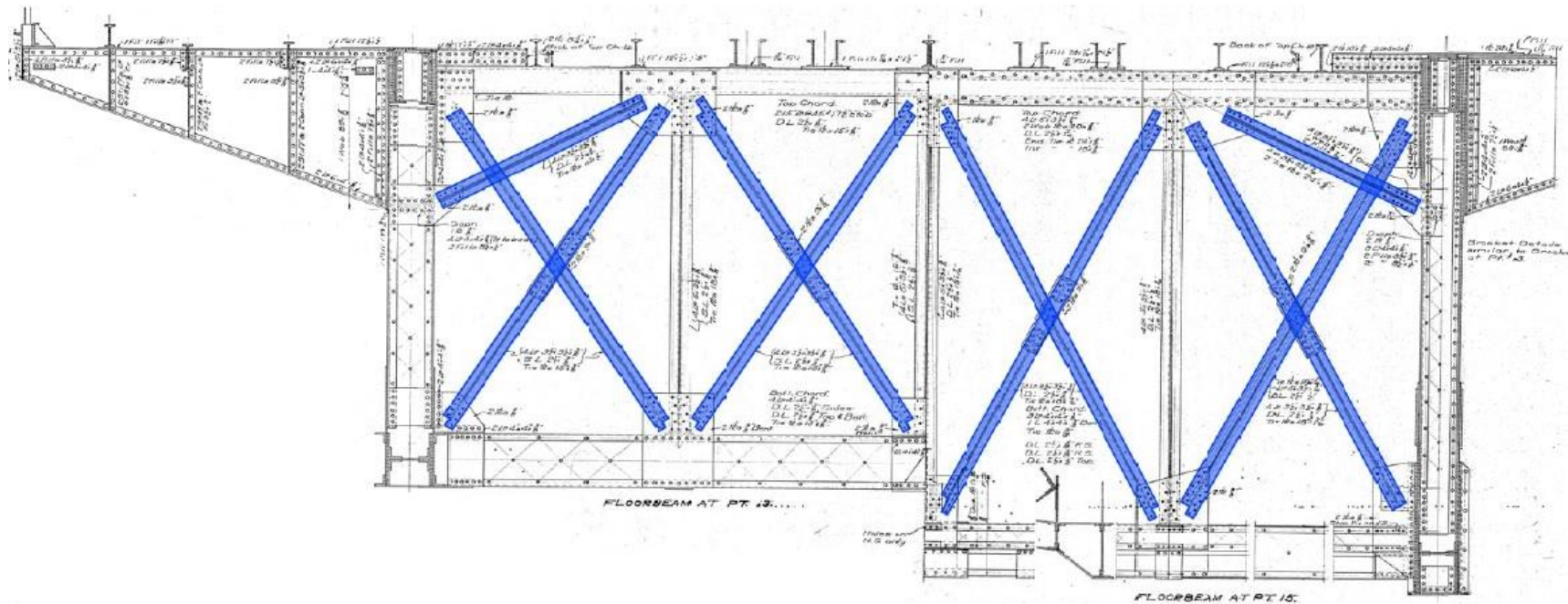
As part of the overall pier wall retrofit, the seating length on top of the pier walls under the pit deck stringers would be increased to prevent unseating of the pit deck over the tops of Piers 2 and 3 (see Figure 8-11).

8.2.10 Strengthening the Bascule Leaves

8.2.10.1 Add and Strengthen Lateral Bracings

The bascule leaf trusses were not designed for seismic loads and movement; adding or strengthening the lateral load-carrying members is required. These members include diagonal sway bracings and connected vertical members, as well as bottom lateral bracings (see Figure 8-19). The bracing members at truss member connections 13, 14, and 15 would have the highest priority for strengthening.

Figure 8-19. Strengthening of Bascule Leaf Lateral Bracings



8.2.10.2 Strengthen Deck to Floor-Beam Connections

Shear capacity of the deck to steel floor-beam connection would need to be verified and strengthened as needed.

8.2.11 Center Lock Shear

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

8.2.12 Bridge Deck

The existing bridge deck shall be replaced due to the low load rating factors and the need to accommodate the proposed streetcar alignments.

New stringers would be added between the existing stringers where needed and would result in the following changes:

- Reduce the live load demand on the existing stringers
- Increase the deck live load-carrying capacity
- Support the proposed streetcar tracks

8.2.13 Structures on the Bridge

Other structures attached on the bridge, such as the overhead sign structure and light poles, would be checked and strengthened to prevent them from collapsing onto the bridge during a design-level seismic event.

8.2.14 Mechanical and Electrical Equipment Replacement

8.2.14.1 Mechanical Equipment Replacement

The mechanical rehabilitation would include a full replacement of the entire movable span operating machinery up to, but not including, the racks mounted on the bascule girders. The same basic machinery layout would be maintained, but all open gearing sets would be replaced with enclosed gearing. The system would maintain two drive motors with motor brakes. These would drive a single differential gearbox. The differential gearbox would drive two output shafts with machinery brakes mounted along their lengths. These cross shafts would each drive a second gearbox at the north and south ends of the span. This gearbox would be coupled to the final new rack pinion at each of the two existing racks. The machinery system would be mounted on a support platform that is integral with the trunnion support frame, in order to minimize the differential movement between the pinions and the racks.

The center span locks and the machinery would be replaced with new span locks that are able to provide restraints to the bascule leaf tip relative movements.

8.2.14.2 Electrical Equipment Replacement

The electrical rehabilitation would include replacement of the old and relatively outdated components. Incoming power distribution and span operation motor and drive system infrastructure would be replaced, which includes the manual transfer switches, generator receptacles, disconnect switches, circuit breakers, span operation motors, and span operation motor drives. The potential increased amperage capacity of this equipment would require installing larger wiring and conduits with these pieces of equipment.

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

The communication cables and conduit could also be replaced if the age or condition of the cables is suspect.

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

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

8.2.14.3 Emergency Winch System

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

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

8.2.15 Restore Bascule Leaves' Opening Angle

To restore the bascule leaves' opening angle to reach 73 degrees and 30 seconds from the horizontal, as indicated in the 1924 as-built plans, the previously added stopper in the bascule pier pit would be removed, and the previously added counterweight at the bottom of the original counterweight block would be removed. To keep the weight balance of the bascule leaf, part of the existing concrete counterweight block would be removed and replaced with heavier material such as steel.

9 Eliminating Legal and Permit Load Rating Deficiencies

According to the load rating results described in Chapter 6, there are elements of the Burnside Bridge that have load rating factors less than 1.0 for legal, special haul, permit, and emergency vehicles. These elements include reinforced concrete deck, stringers, crossbeams, members of steel trusses, etc. As part of the Enhanced Seismic Retrofit Alternative, these deficient elements would be strengthened to achieve a rating factor greater than 1.0 for the legal, special haul, permit, and emergency vehicles.

Strengthening the structure to achieve a rating factor greater than 1.0 for all legal, special haul, permit, and emergency vehicles would allow heavy equipment for debris management to be transported over the bridge on multiple-axle trucks with low-boy trailers in the event of a catastrophic seismic event. The concepts of strengthening the structures to meet the current design code requirements for live load-carrying capacity are integral parts of the Enhanced Retrofit Alternative, as shown in the Conceptual Plans (Appendix C).

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10 Pier Fender Replacements

The existing bascule pier fender systems would be removed during the construction of the drilled shafts and the enlarged pile caps. Therefore, the fenders would be replaced as part of the Enhanced Retrofit Alternative.

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

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11 Geotechnical Hazard Mitigation Approach

Because of the risk of seismically induced soil liquefaction hazards at the Project site, soil mitigation measures have been proposed and are described in the *EQRB Geotechnical Report* (Multnomah County 2021) (Appendix A). Figures 9 to 11 in the *EQRB Geotechnical Report* (Multnomah County 2021) provide an overview of the proposed soil mitigation.

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12 Constructability, Access and Impacts

The Burnside Bridge is located in a highly congested downtown area and crosses over a major waterway, several highways, and railroad tracks. Several major constructability and impact issues are described below. More in-depth discussions on constructability and impacts are in the *EQRB Construction Approach Technical Report* (Multnomah County 2021) (Appendix A).

12.1 Vertical Construction Clearances

Some of the drilled shafts could be constructed under the existing bridge deck, more likely under the sidewalks, in order to minimize the foundation sizes. This would increase construction costs and extend the construction time. Potential construction methods include the following:

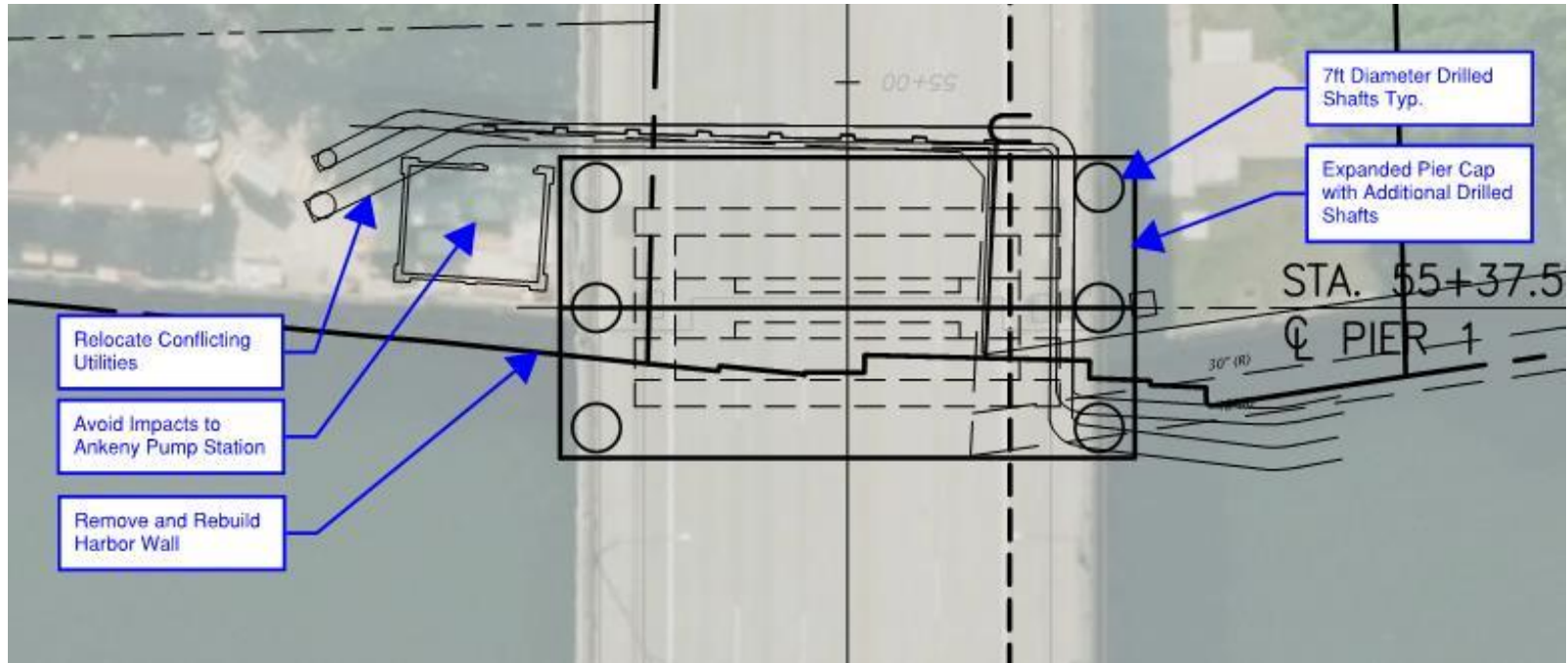
- Construct the drilled shafts in phases by partially closing the traffic on the bridge and removing part of the bridge deck. The shaft steel casings and reinforcing cages could be dropped from the removed deck spaces.
- Construct under the bridge deck where minimum vertical clearance is allowed. The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

12.2 Site Restraints and Construction Access

At Pier 1, there are major underground utility lines, a pump station, and a seawall adjacent to the pier foundation. The foundation retrofit would need to be coordinated with potential utility line relocations and reconstruction of the seawall, and would need to avoid impacting the pump station (see Figure 12-1). These requirements would increase construction costs.

Given that the Ankeny Pump Station is adjacent to Pier 1, transverse post-tensioning on the enlarged new shaft cap would be limited for jacking operation from the north side only. One end jacking would not be an issue because the relatively short tendon length.

Figure 12-1. Site Restraints at Pier 1



The construction of the new replacement Pier 4 would require removing a section of the Eastbank Esplanade to provide construction access. The equipment could, therefore, be shipped in on a barge.

Temporary construction trestles would be used for the in-water foundation construction.

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

More detailed discussions on construction access can be found in the *EQRB Bridge Replacement Technical Report* (Multnomah County 2021) and the *EQRB Construction Approach Technical Report* (Multnomah County 2021).

12.3 Navigational Channel

USCG requires that the Willamette River navigational channel remain open during construction; only short-term channel width reduction would be allowed. See the *EQRB Preliminary Navigation Study* (Multnomah County 2021) (Appendix A) for details.

To maintain navigational channel clearance during the construction, the bascule leaves would be temporarily removed for retrofit in an off-site construction location. The retrofitted and enhanced leaves would be shipped back and reinstalled on to the bascule piers.

12.4 Construction Staging

Two methods for construction and traffic staging are being investigated.

- Divert multimodal traffic to an onsite temporary bridge.
- Close the Burnside Bridge river crossing for the duration of construction, and reroute all traffic to adjacent river crossings.

12.4.1 Enhanced Retrofit with Temporary Bridge

This approach would divert multimodal traffic around the existing bridge through use of a temporary bridge located immediately adjacent to the south of the existing bridge alignment. The temporary bridge would be located sufficiently south to allow for construction access for retrofit activities.

The temporary bridge could consist of fixed spans along the east and west approach, and a movable lift span within the river navigation channel. This would allow for closing of the majority of the existing bridge spans during the construction of the enhanced retrofit.

Two types of the temporary bridge are investigated: a multi-model temporary bridge and a bike/ped-only temporary bridge. More detailed discussions can be found in the *EQRB Bridge Replacement Technical Report* (Multnomah County 2021).

12.4.2 Enhanced Retrofit without Temporary Bridge

This approach would close the Burnside Bridge crossing (from E Martin Luther King Jr. Boulevard to W 3rd Avenue) to all modes of transportation for the duration of construction. Detour routes would be established to route multimodal traffic to adjacent river crossings. This approach would allow the contractor to close the existing bridge during the construction without concerns for staging traffic. All other facilities crossed by Burnside Street (e.g., I-5, various city streets, and TriMet MAX lines) would have to be maintained and protected, except for short-term closures for construction activities such as girder erection and deck placement.

13 Summary of Enhanced Seismic Retrofit

13.1 Major Work Items

Enhanced retrofit-related major structural, mechanical, and electrical work elements are listed below.

Table 13-1. Summary of Major Work Elements for Enhanced Retrofit

Location	Retrofit
Structural	
Bent 1 & Span 1	Abut. Strengthening
Bent 2 & Span 2 to Bent 16 & Span 16	Girder Restrainers and Strengthening
	Floor-Beam Strengthening
	Column Jacketing
	Footing Enlargement
Bent 17 & Span 17 to Bent 19 & Span 19	Girder Restrainers and Strengthening
	Floor-Beam Strengthening
	Column Jacketing
	Footing Enlargement
	Drilled Shafts
Pier 1	Relocation of Force Mains
	7-foot Dia Shafts
	Pile Cap Enlargement
	Harbor Wall Reconstruction
	Pier Column Strengthening
	Bearing Replacement
West Truss Span	Lateral Load Member Strengthening
	Connection Retrofit

Table 13-1. Summary of Major Work Elements for Enhanced Retrofit (continued)

Location	Retrofit
Pier 2	12-foot Dia Drilled Shafts
	Pile Cap Enlargement
	Adding Corner Columns
	Pier Wall and House Strengthening
	Support Pedestal Strengthening
	Pit Deck Bearing Retrofit
	Trunnion Frame Strengthening
	Trunnion Frame Anchorage Strengthening
	Counterweight Frame Strengthening
	Install Lateral Restraints
	Rocker Bearing Replacement
	Live Load Shoe Retrofit
Pier 3	12-foot Dia Drilled Shafts
	Pile Cap Enlargement
	Adding Corner Columns
	Pier Wall & House Strengthening
	Support Pedestal Strengthening
	Pit Deck Bearing Retrofit
	Trunnion Frame Strengthening
	Trunnion Frame Anchorage Strengthening
	Counterweight Frame Strengthening
	Install Lateral Restraints
	Rocker Bearing Replacement
	Live Load Shoe Retrofit
Bascule Leaves	Lateral Load Member Strengthening
	Connection Retrofit
	Center Lock Retrofit
East Truss Span	Lateral Load Member Strengthening
	Shorten Existing Truss
Pier 4	10-foot Dia Shafts
	New Columns
	New Pier Cap Super-bent
	New seismic restrainers Bearing Replacement

Table 13-1. Summary of Major Work Elements for Enhanced Retrofit (continued)

Location	Retrofit
Spans 20-24	Replace with three New Spans
Bent 25 & Span 25 to Bent 27 & Span 27	10-foot Dia Drilled Shafts
	Pile Cap and Grade Beam Extension
	Partial Infill Wall
	Column Strengthening
	Floor-Beam Strengthening
	Bearing Replacement
	Steel Girder Strengthening
Bent 28 & Span 28 to Bent 34 & Span 34	Girder Restrainers and Strengthening
	Floor-Beam Strengthening
	Column Jacketing
	Footing Enlargement
Bent 35	Abut. Strengthening
<i>Mechanical and Electrical</i>	
Bascule Span	Operating Machinery Replacement
	Rehabilitation of Trunnions and links
	Span Balance Work
	Replace incoming electrical service
	Center-span lock power feed
	Replace motors and drives
	Relocate and update PLCs
	Replace navigation lighting
	Replace traffic warning gates
Relocating electrical equipment	
<i>Bridge Deck Replacement</i>	
Approach Structures	Deck Replacement
River Spans	Deck Replacement

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

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

13.2 Conceptual Plans for Enhanced Seismic Retrofit

The Conceptual Plans for Burnside Bridge enhanced seismic retrofits are attached in Appendix C.

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- 2021d. EQRB Enhanced Seismic Retrofit Technical Report. <https://multco.us/earthquake-ready-burnside-bridge/project-library>.
- 2021e. EQRB Existing Roadway Deficiency Memo. <https://multco.us/earthquake-ready-burnside-bridge/project-library>.
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Appendix A. Supporting Reports

Supporting Reports

EQRB Bridge Design Criteria

EQRB Seismic Design Criteria

EQRB Existing Roadway Deficiency Memo

EQRB Facility Standards List

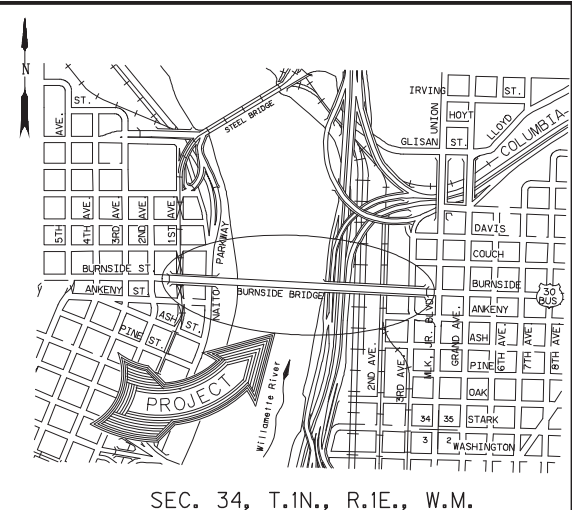
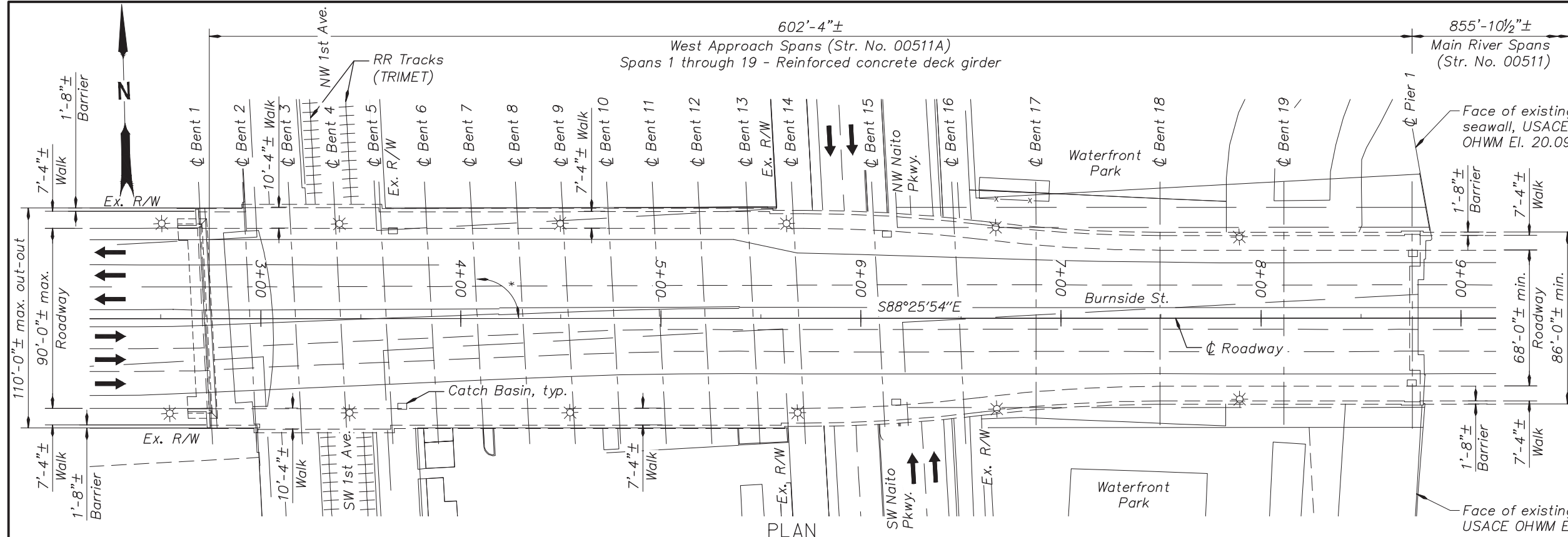
EQRB Geotechnical Report

EQRB Preliminary Navigation Study

EQRB Construction Approach Technical Report

Supporting documents were developed to support the NEPA Environmental Impact Statement (EIS) are available in the project library (<https://multco.us/earthquake-ready-burnside-bridge/project-library>).

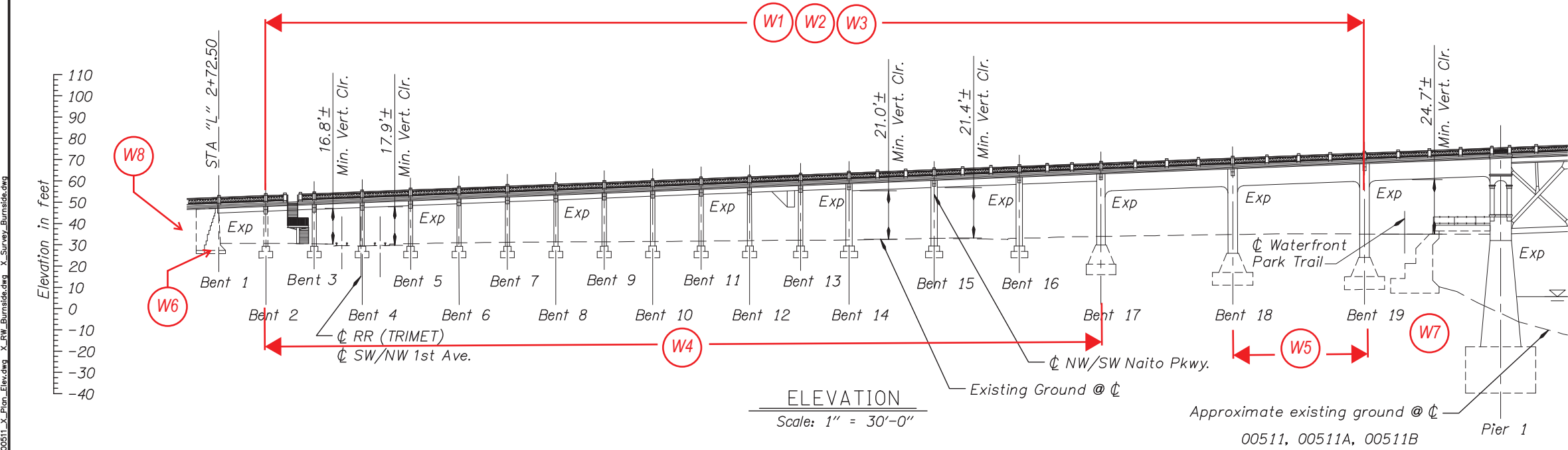
Appendix B. Seismic Deficiency Plans



POTENTIAL SEISMIC DEFICIENCY

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

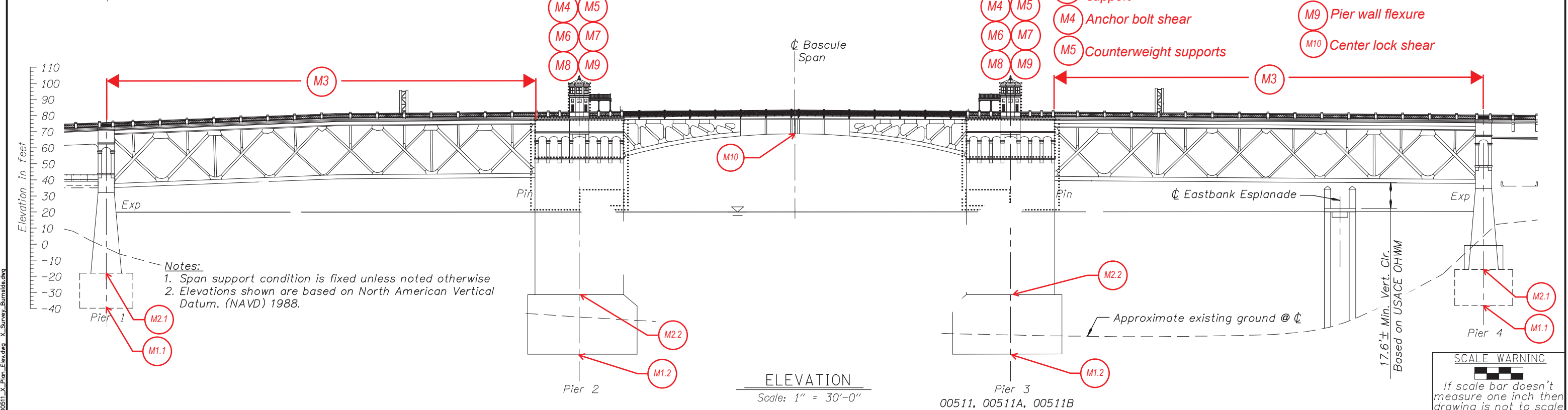
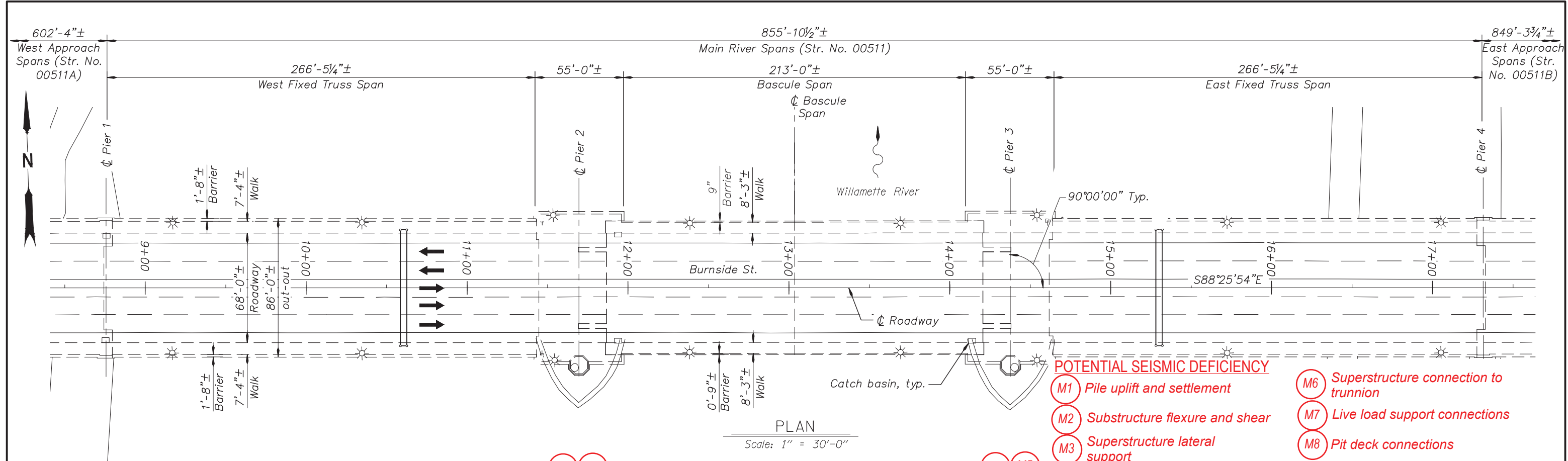
Notes:
1. Span support conditions otherwise.
2. Elevations shown are based on North American Vertical Datum, (NAVD) 1988.



SCALE WARNING

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

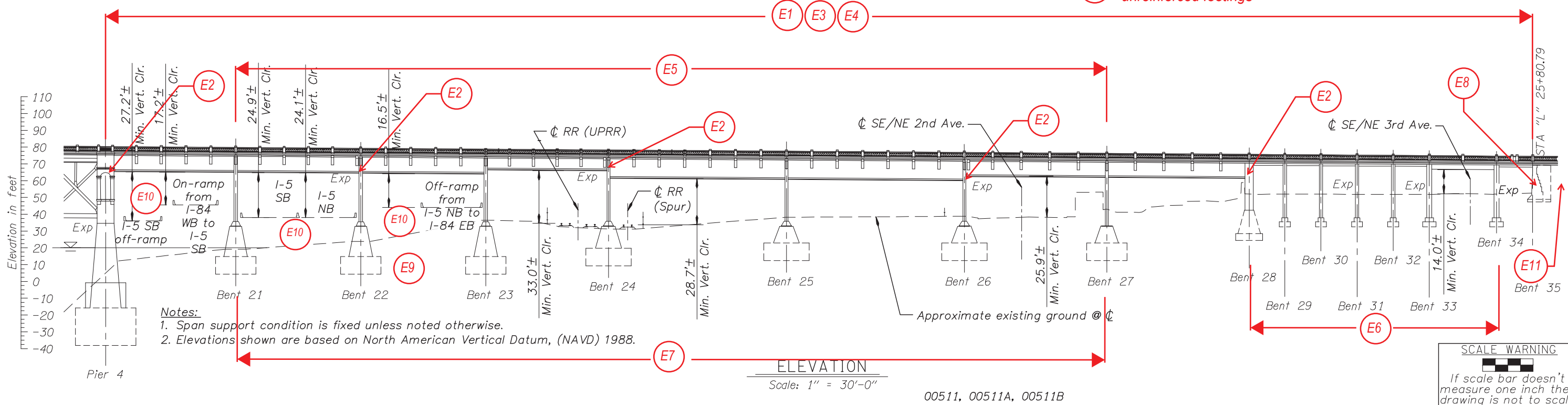
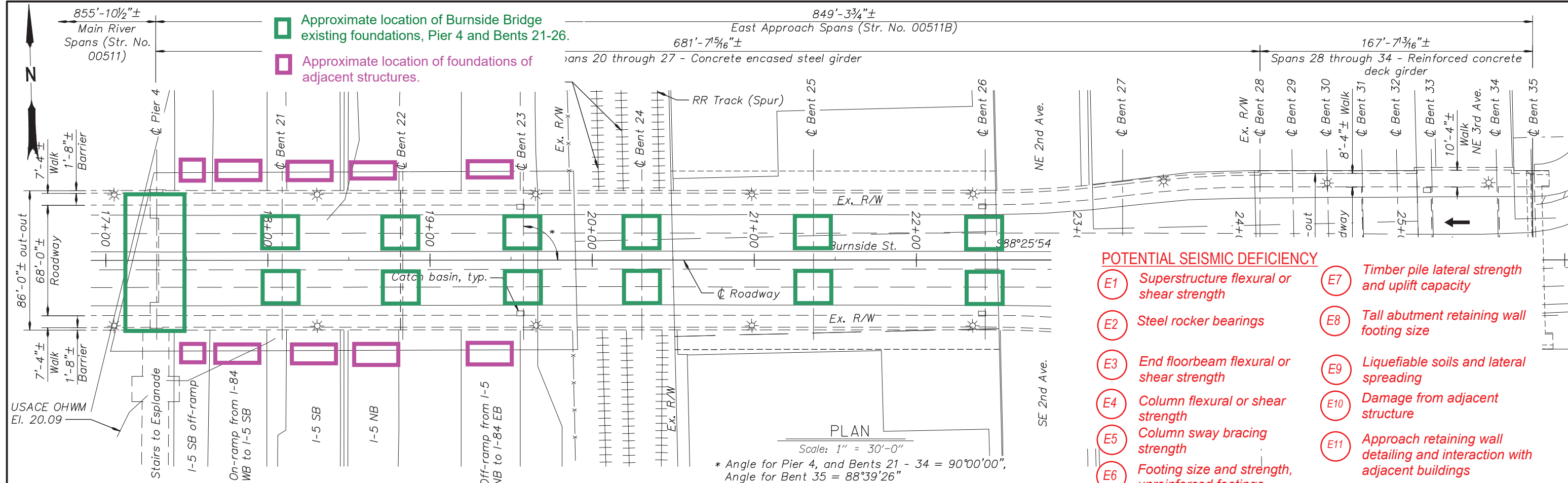
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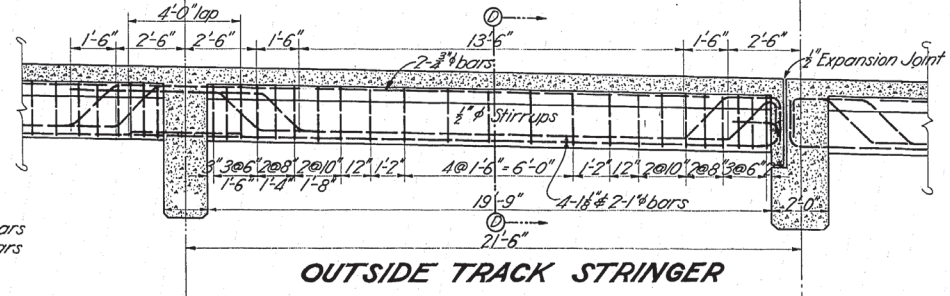
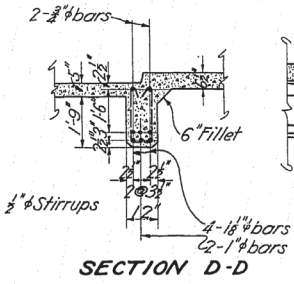
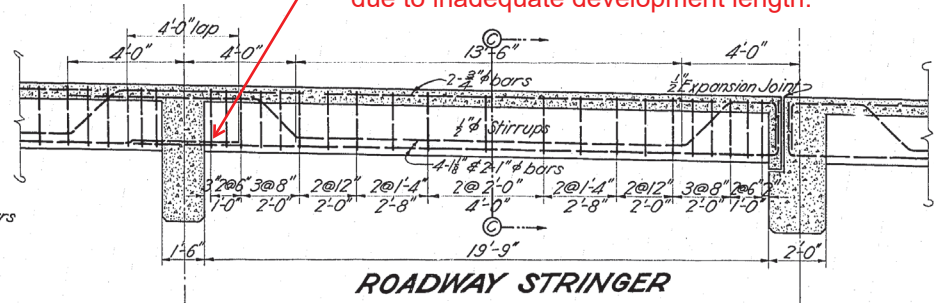
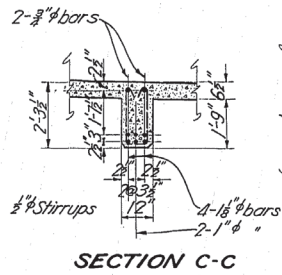
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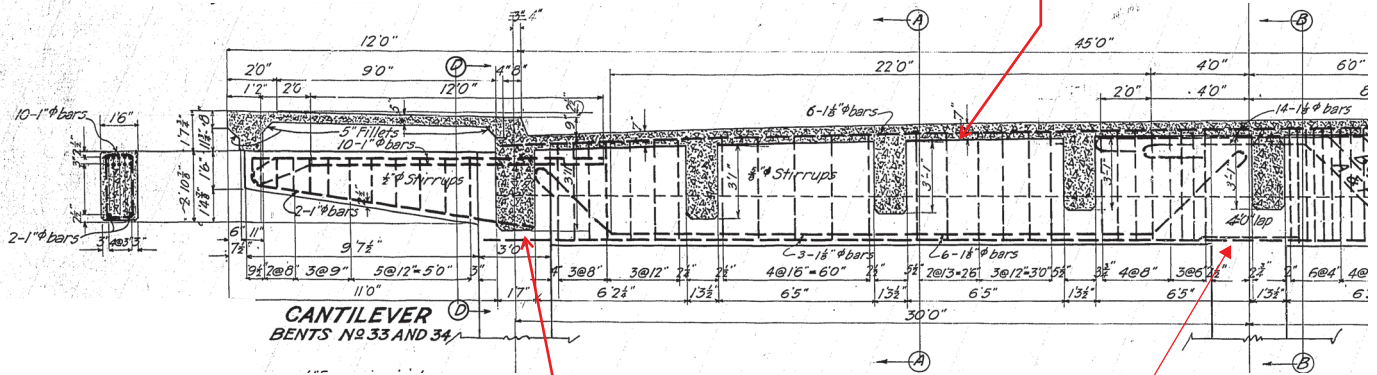
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									DATE		DRAWING NO.
ACCOMPANIED BY DWGS.									CALC. BOOK	PLAN AND ELEVATION - 3	

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



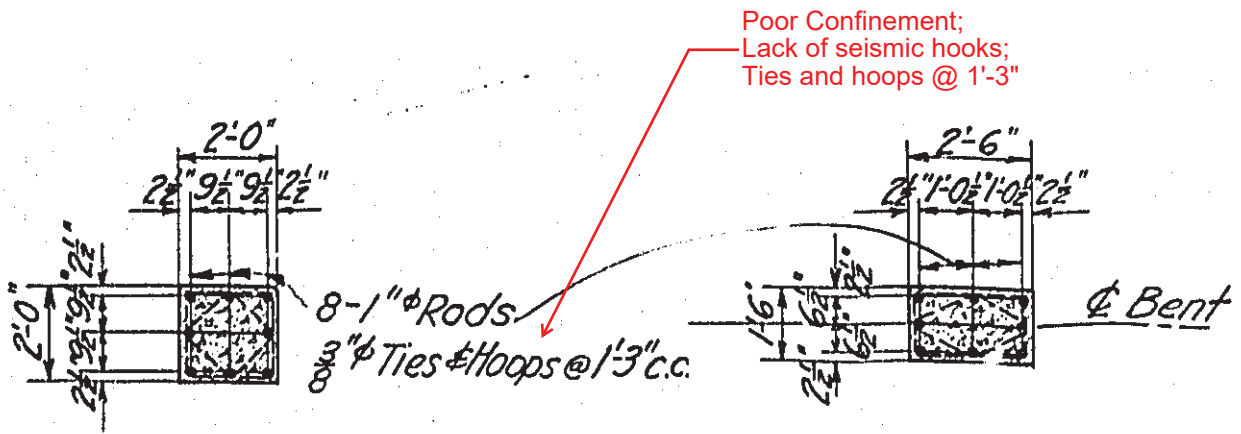
W1

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

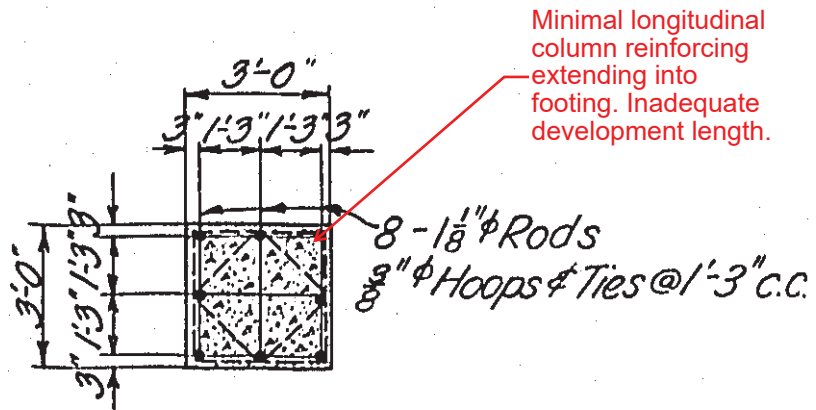


Poor seismic detail. Floorbeam positive moment reinforcement spliced or has limited embedment at column connection thus limiting moment capacity due to inadequate development length.

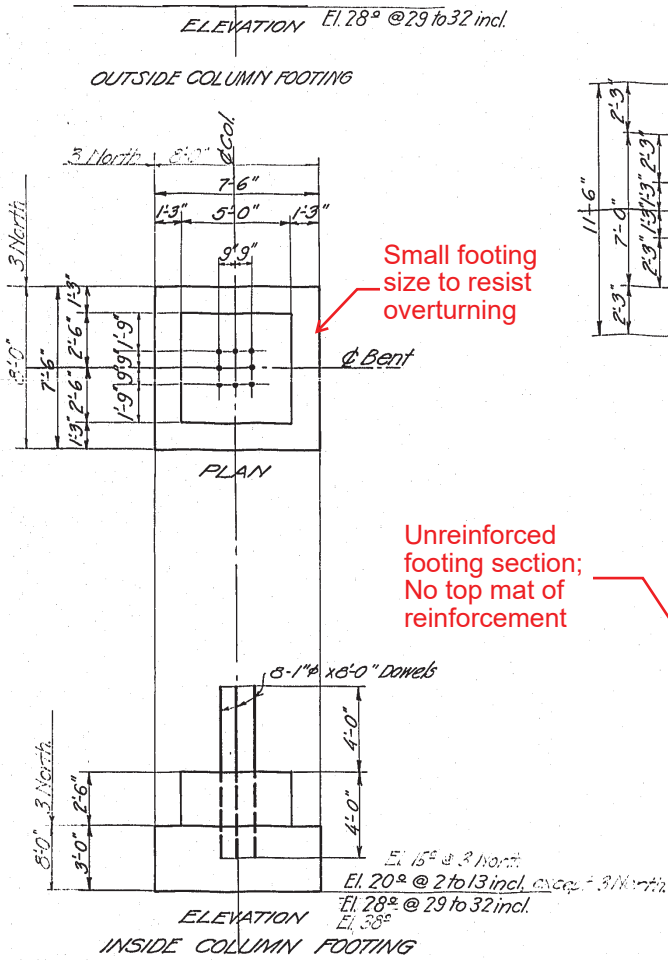
W2



COLUMN CROSS SECTIONS
BENTS 2, 5 TO 13, & 29-32 INC BENTS 3-4 ONLY

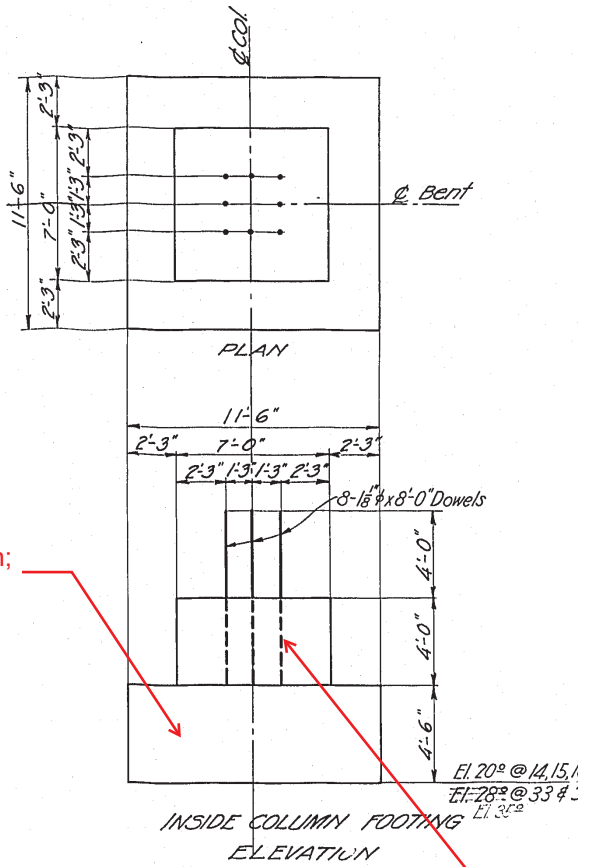


COLUMN CROSS SECTION



Small footing size to resist overturning

Unreinforced footing section; No top mat of reinforcement



BENTS 14, 15, 16, 33, 34

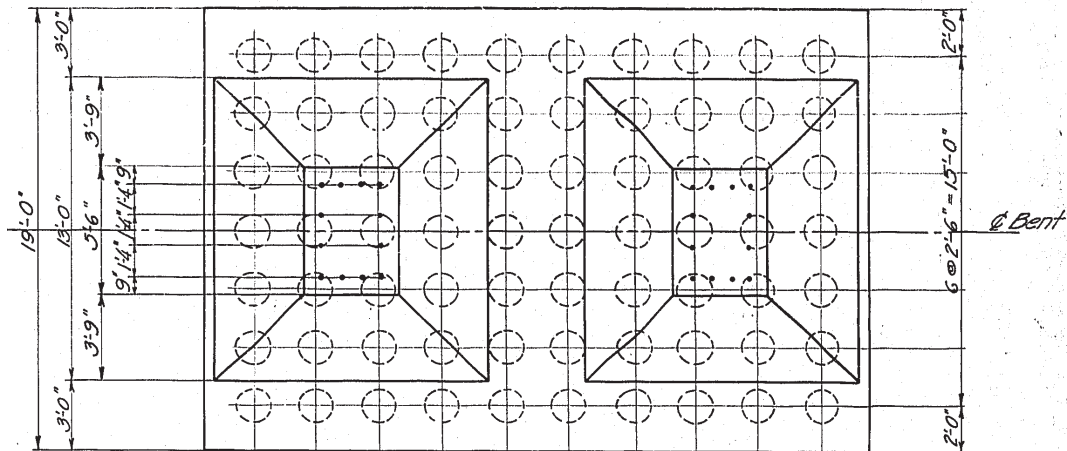
Poor connection detail; Column reinforcement does not extend into footing with adequate embedment.

BENTS 2-13, 29-32 INC

NOTE -- For arrangement of dowels, see column cross section.

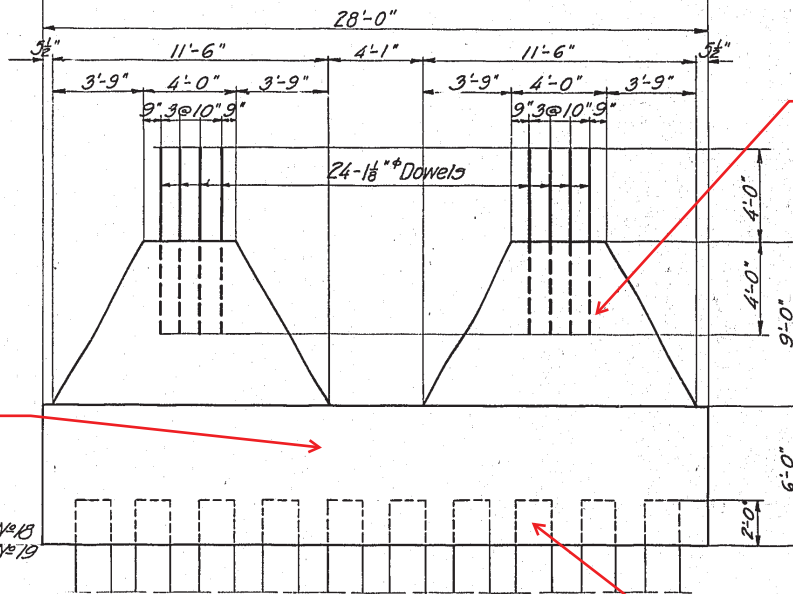
Soil Structure Interaction:
 Spread footing inadequate to limit settlement from liquefaction to a desirable level.





2'-0" 9 @ 2'-8" = 24'-0" 2'-0"

PLAN



ELEVATION

BENTS 18-19

For Pile details see "Pile Record"

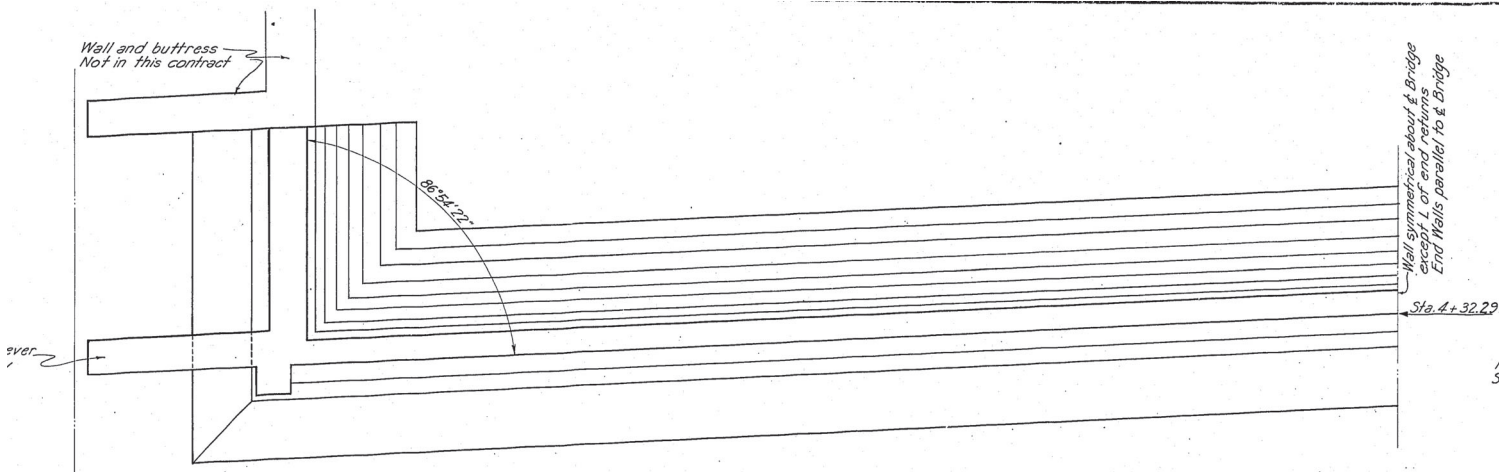
Unreinforced footing section; No top mat of reinforcement

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

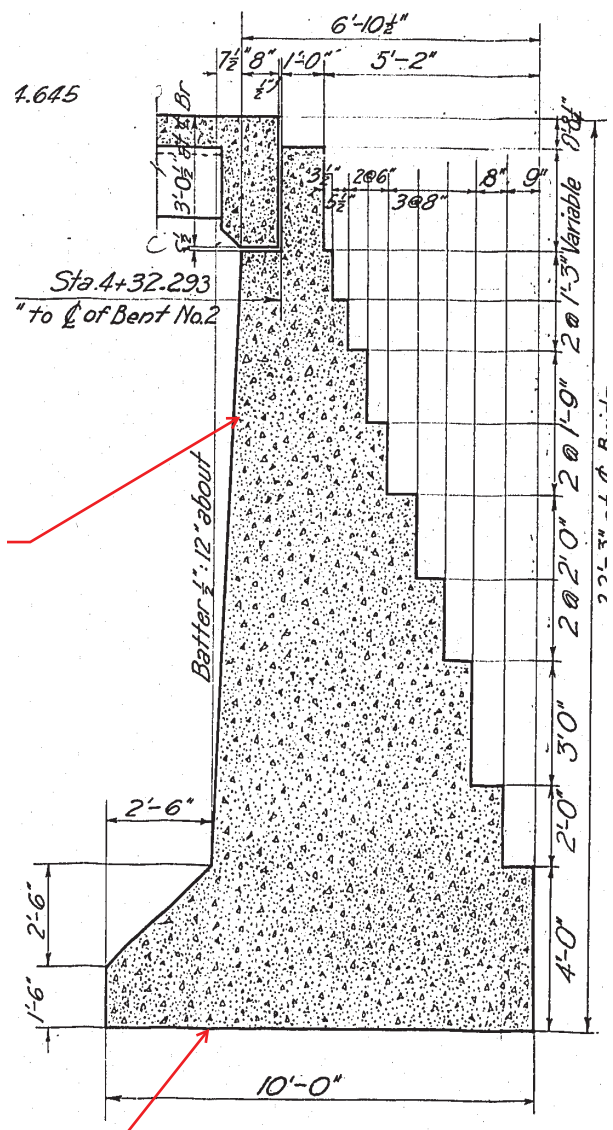
Embedment of piles into pile caps inadequate to resist seismic uplift forces.

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





PLAN



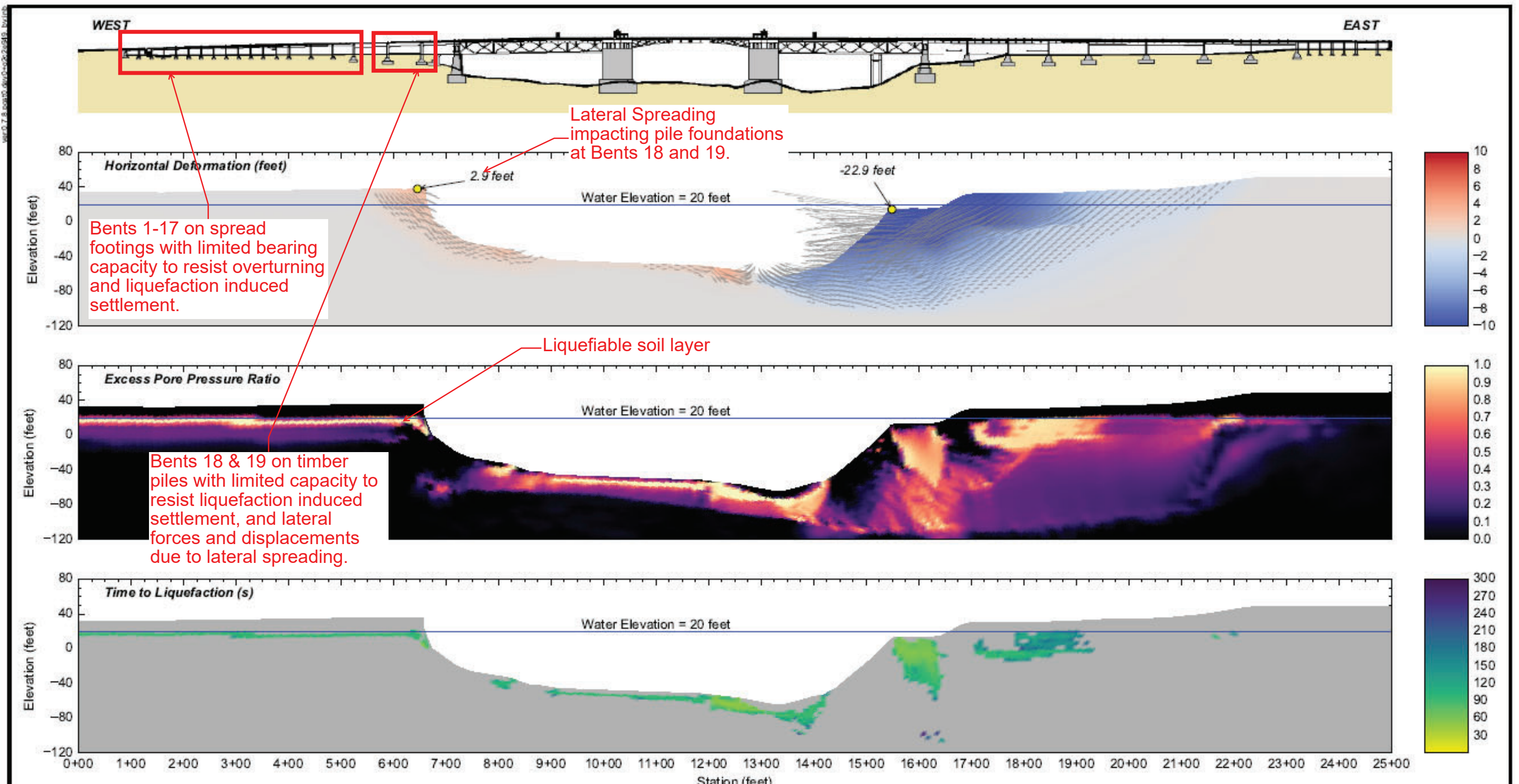
SECTION B-B

Unreinforced abutment wall.

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

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

W6



Bents 1-17 on spread footings with limited bearing capacity to resist overturning and liquefaction induced settlement.

Lateral Spreading impacting pile foundations at Bents 18 and 19.

Liquefiable soil layer

Bents 18 & 19 on timber piles with limited capacity to resist liquefaction induced settlement, and lateral forces and displacements due to lateral spreading.

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

Burnside Bridge
Portland, Oregon

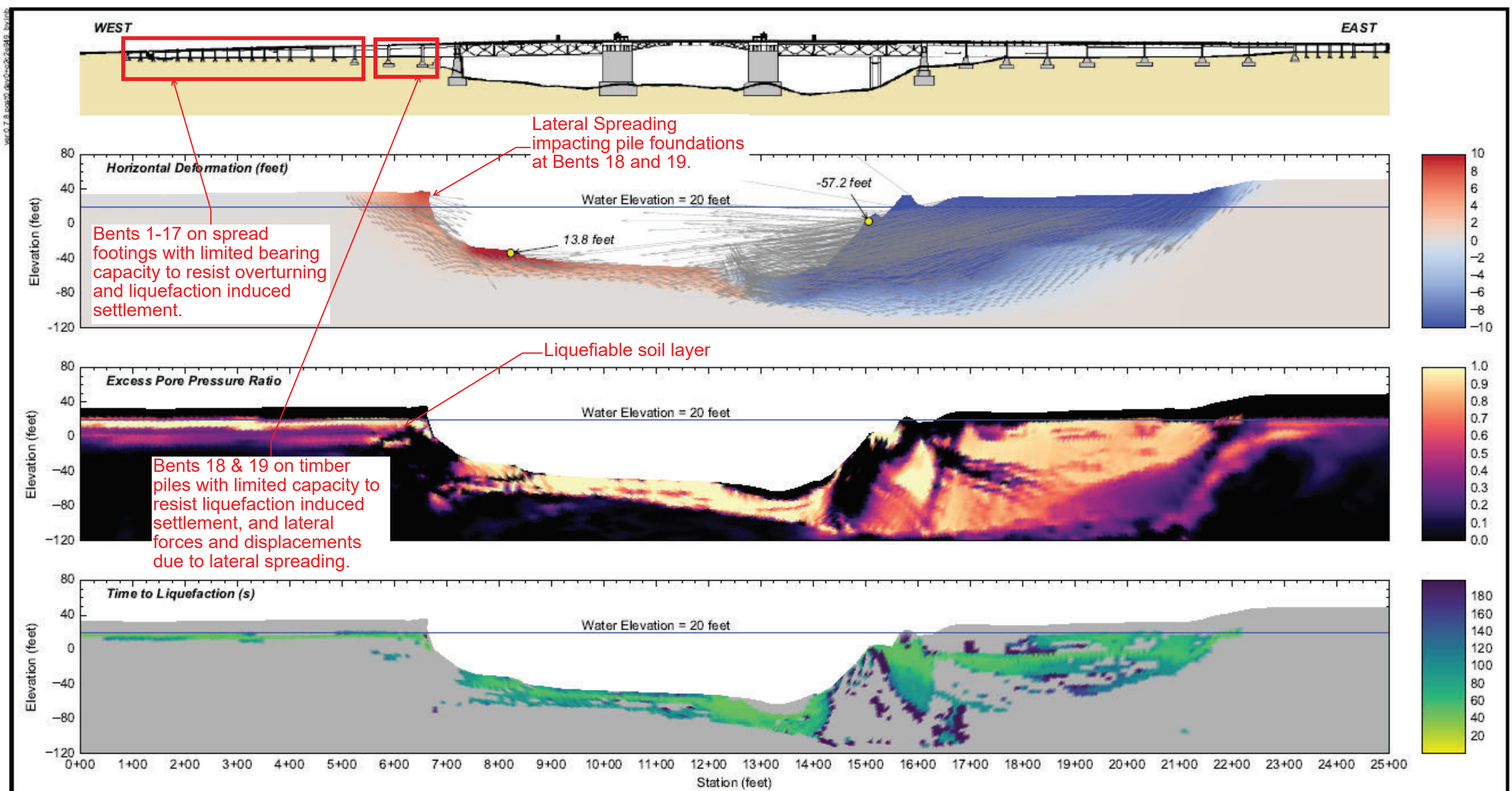
**MODEL RESPONSE
TIME HISTORY T-CSZ**

January 2017 24-1-04065-003

SHANNON & WILSON, INC. Geotechnical and Environmental Consultants

FIG. 6

W7
CSZ

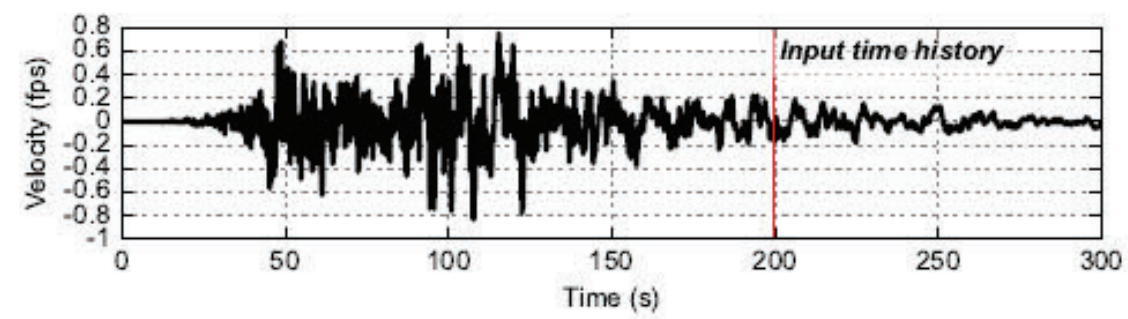


Bents 1-17 on spread footings with limited bearing capacity to resist overturning and liquefaction induced settlement.

Lateral Spreading impacting pile foundations at Bents 18 and 19.

Bents 18 & 19 on timber piles with limited capacity to resist liquefaction induced settlement, and lateral forces and displacements due to lateral spreading.

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



Burnside Bridge
 Portland, Oregon

**MODEL RESPONSE
 TIME HISTORY T-1000**

January 2017 24-1-04065-003

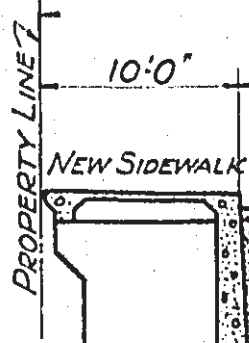
SHANNON & WILSON, INC.
 Geotechnical and Environmental Consultants

FIG. 2

W7
 1000-yr

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

BASEMENT



FILL

OLD WALL OF BUILDING

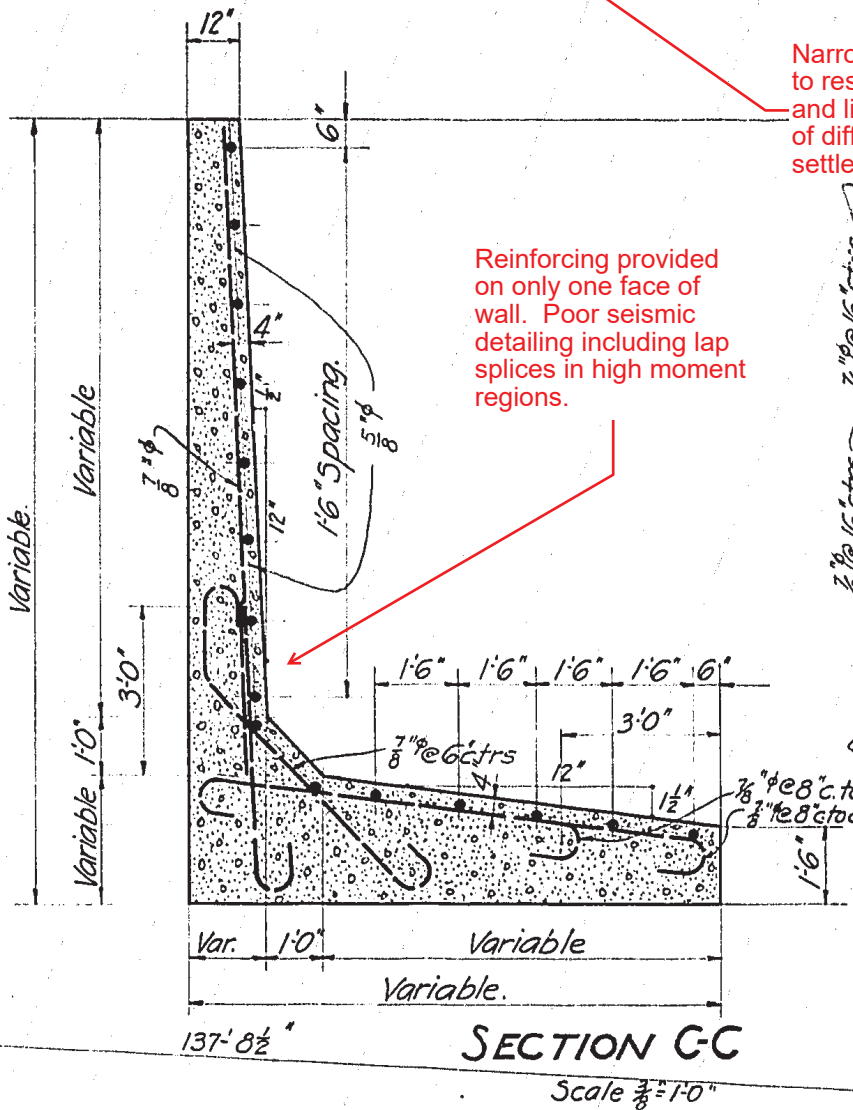
FLOOR 1

RETAINING WALL

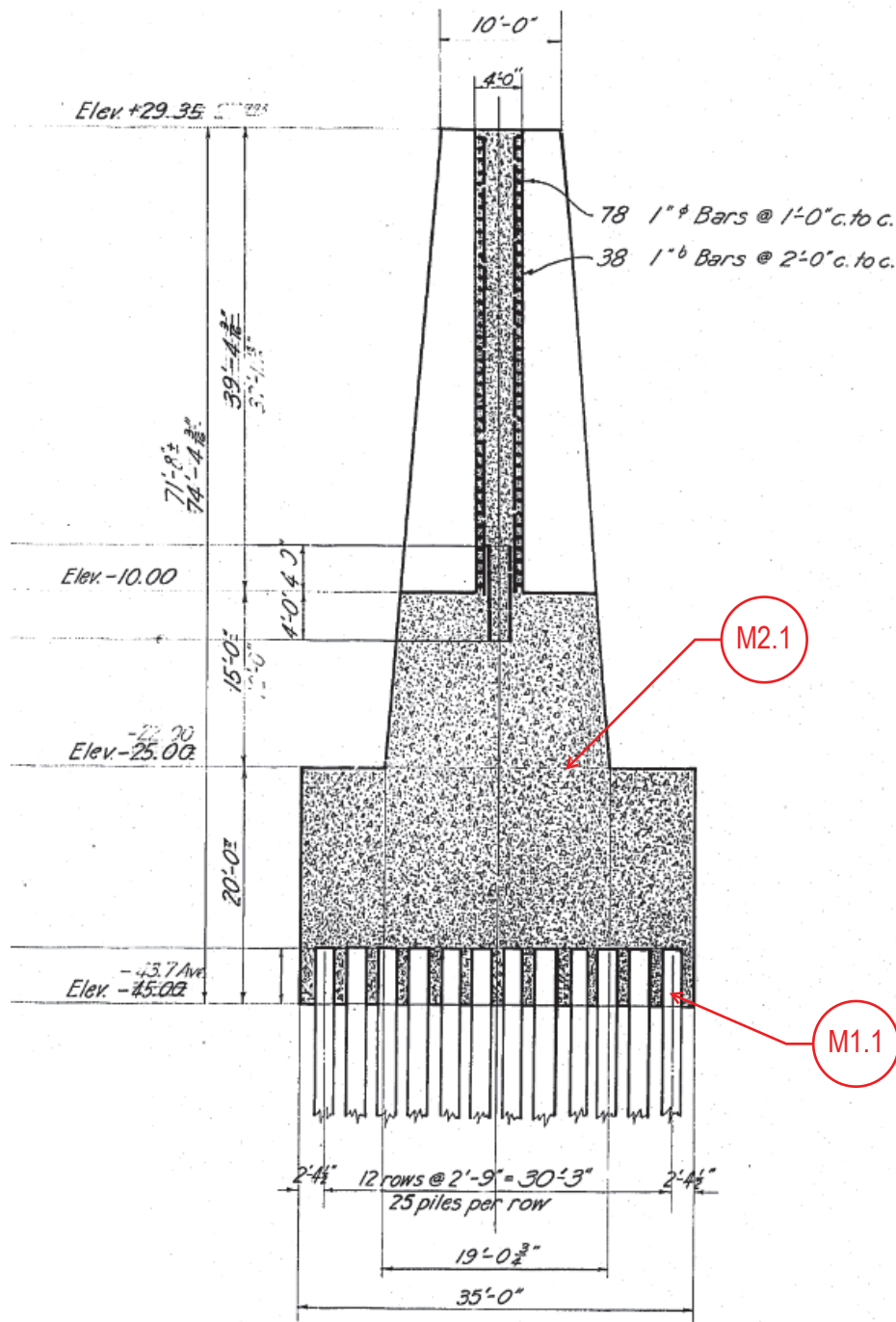
Narrow footing size to resist overturning and limited effects of differential settlement.

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

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



W8



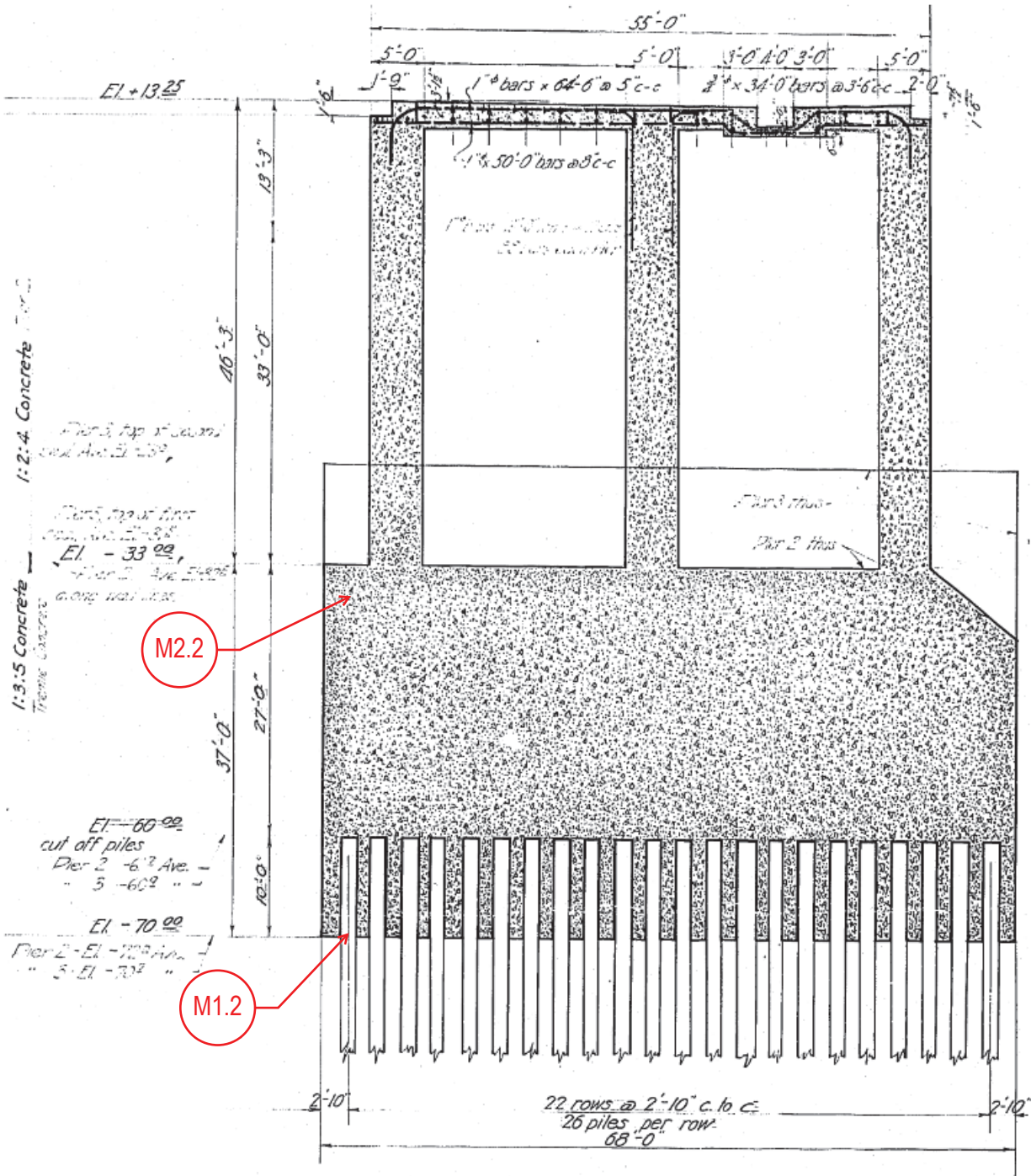
SECTION THROUGH CENTER

M1.1

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

M2.1

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

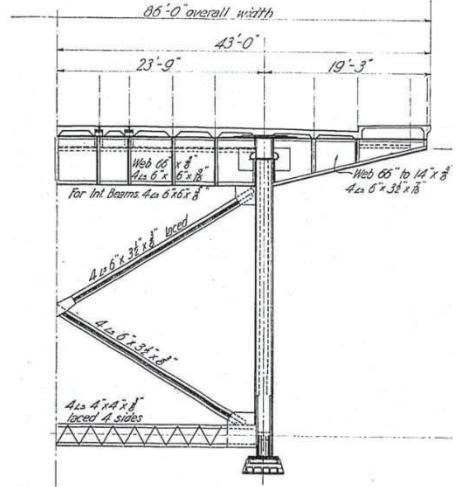
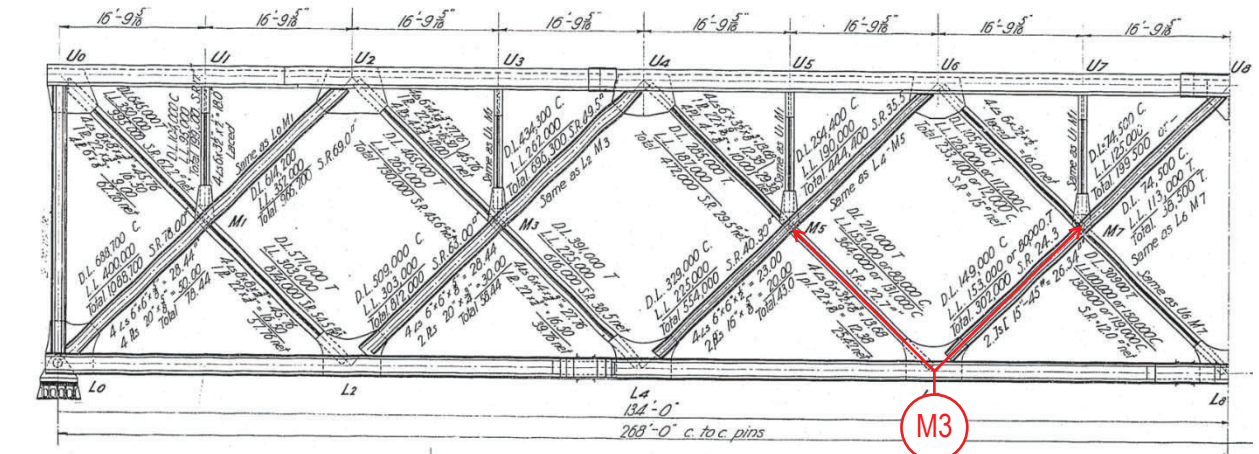


M1.2

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

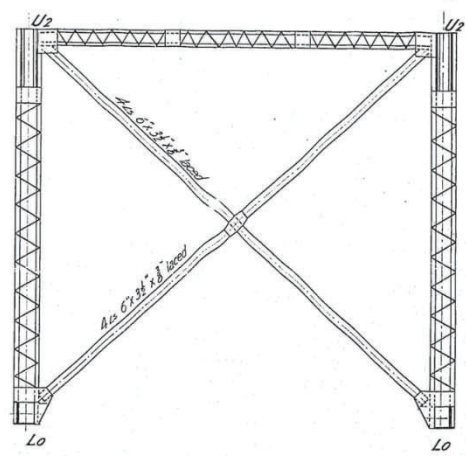
M2.2

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



SWAY BRACING, SHORE END

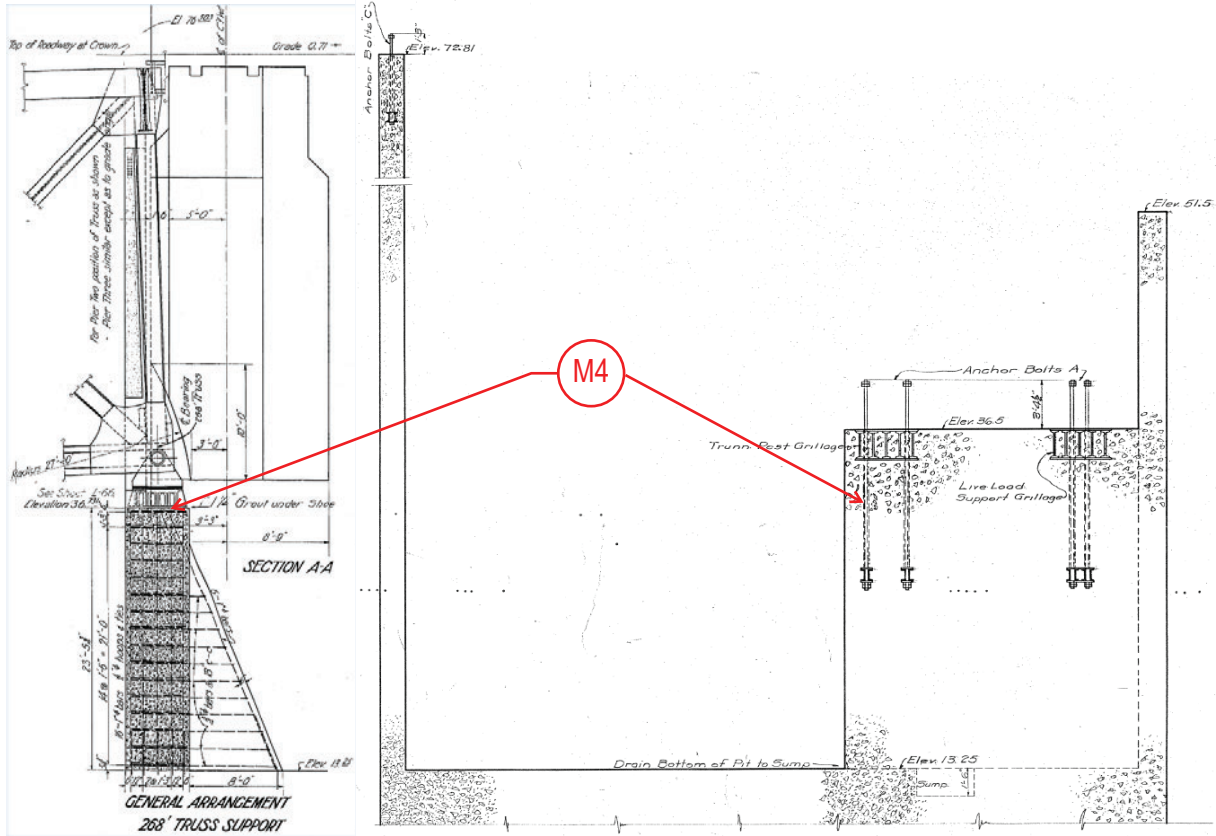
ELEVATION



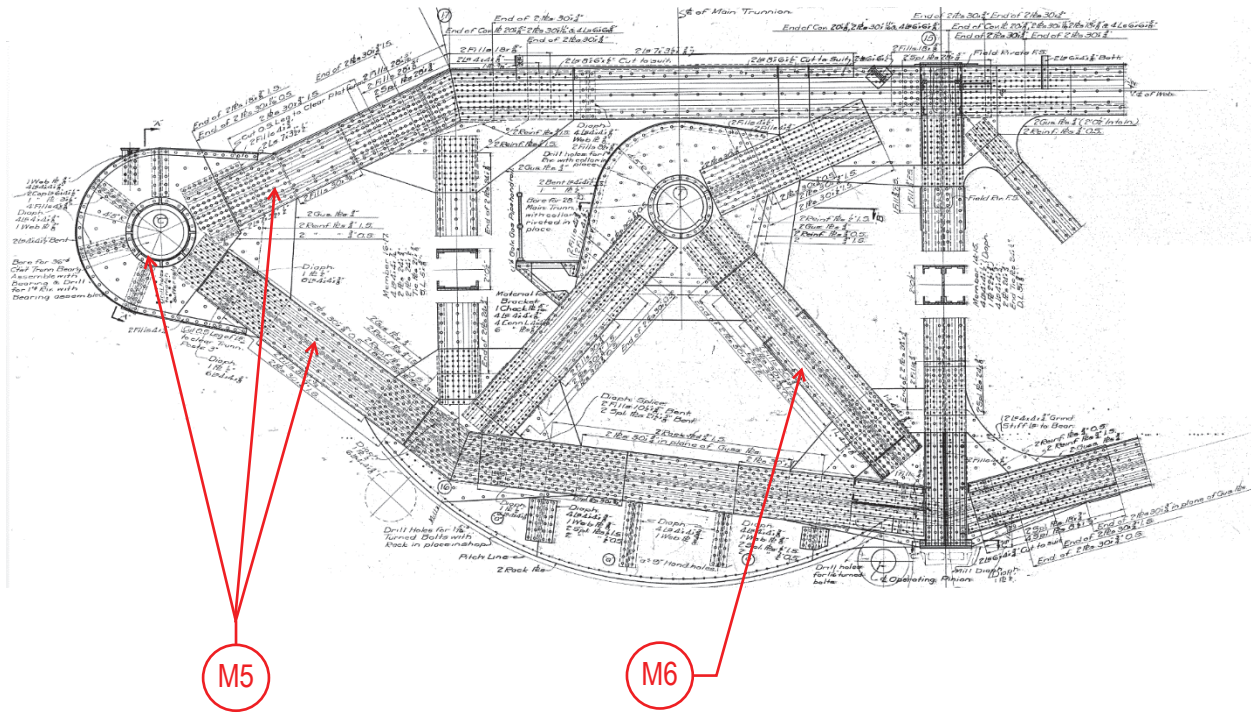
SWAY BRACING, BASCULE END
IN PLANE U2-L0

M3

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



M4 Existing anchor bolts are insufficient to resist seismic forces.

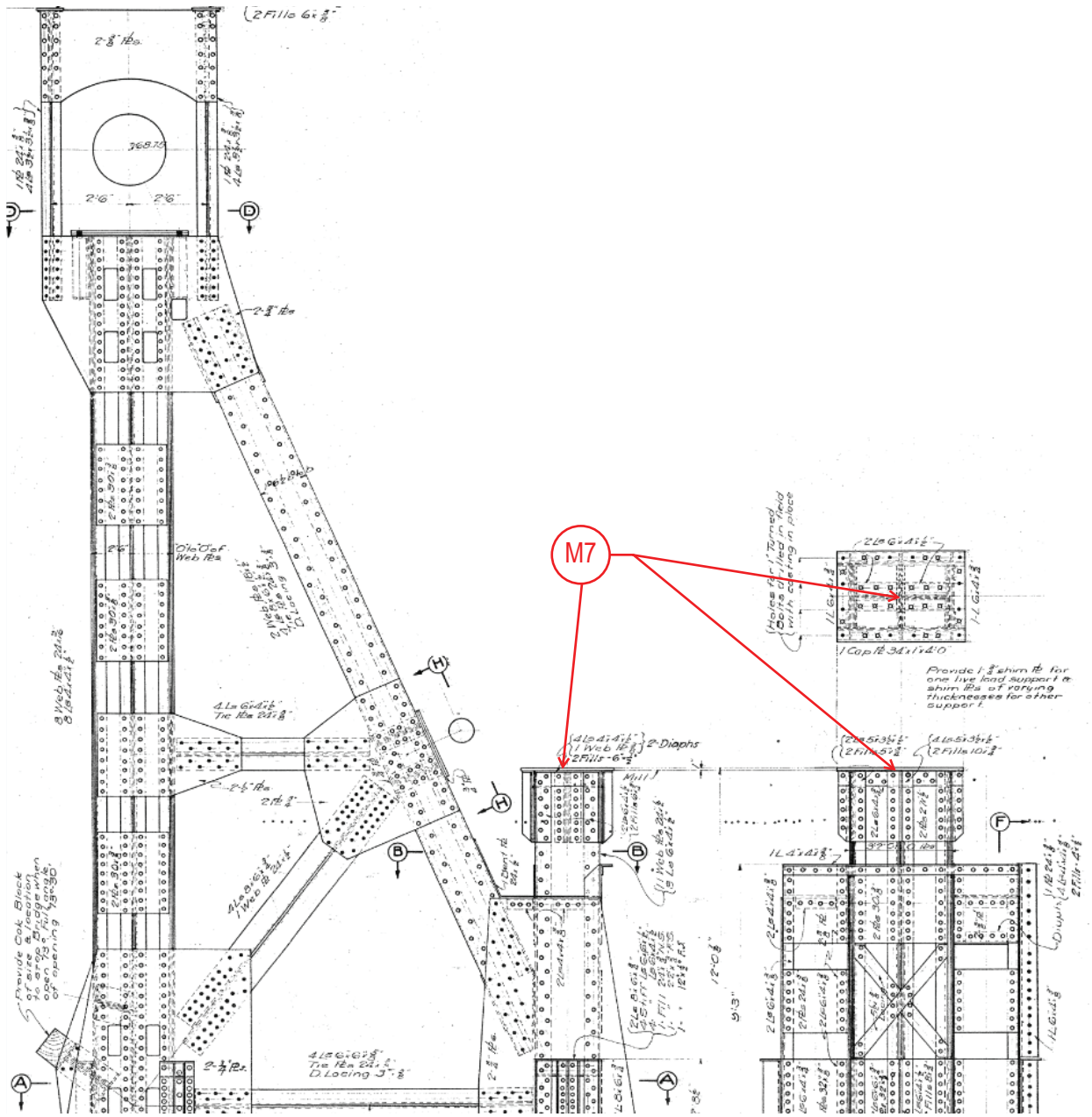


M5

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

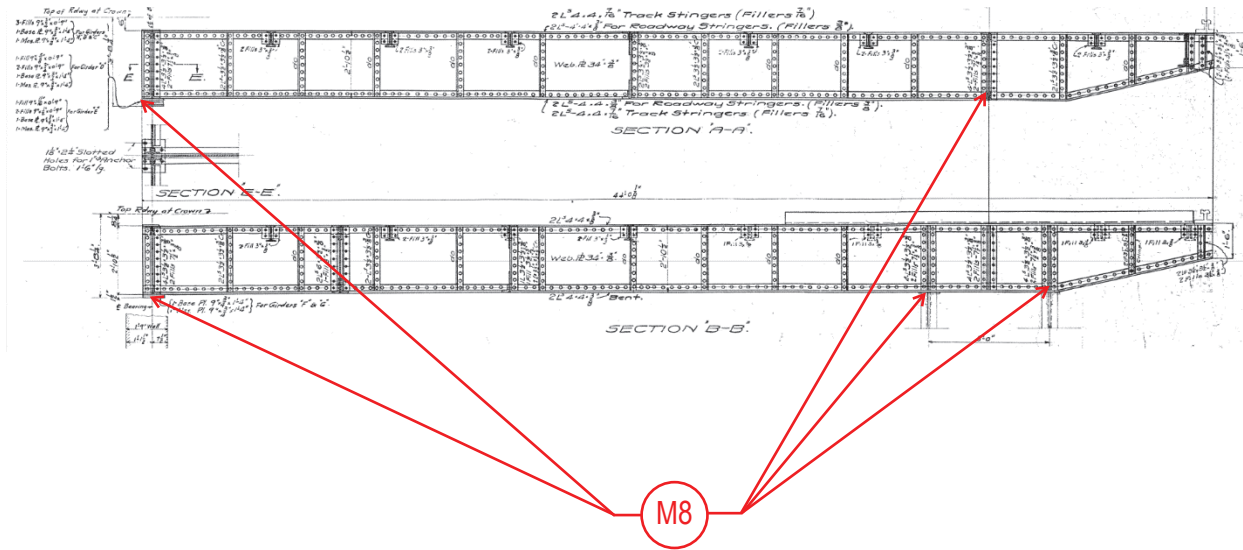
M6

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



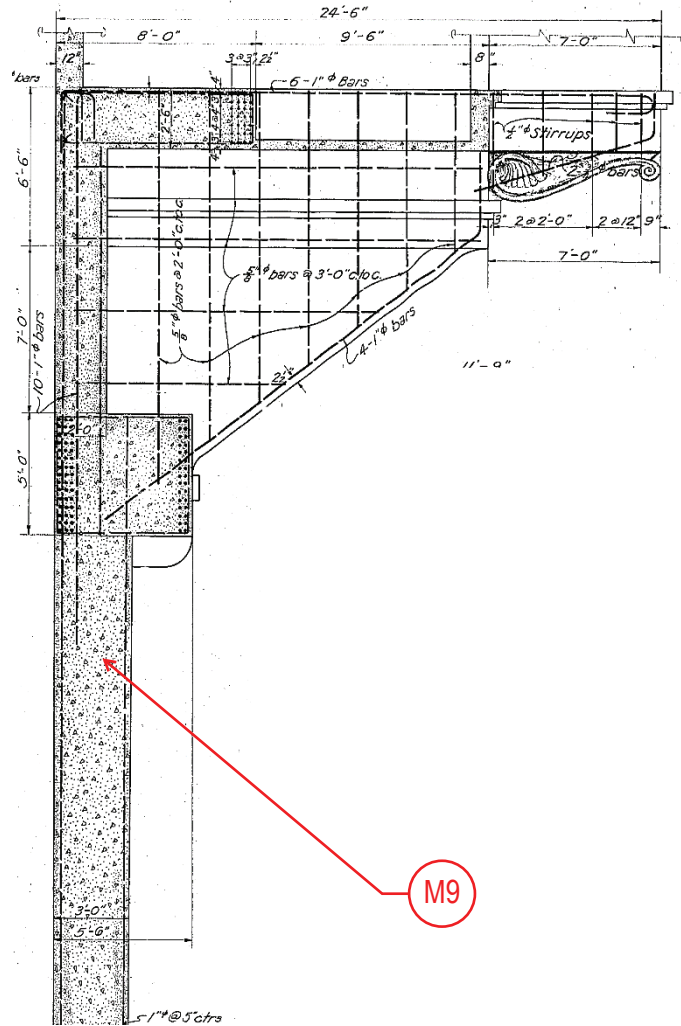
M7

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



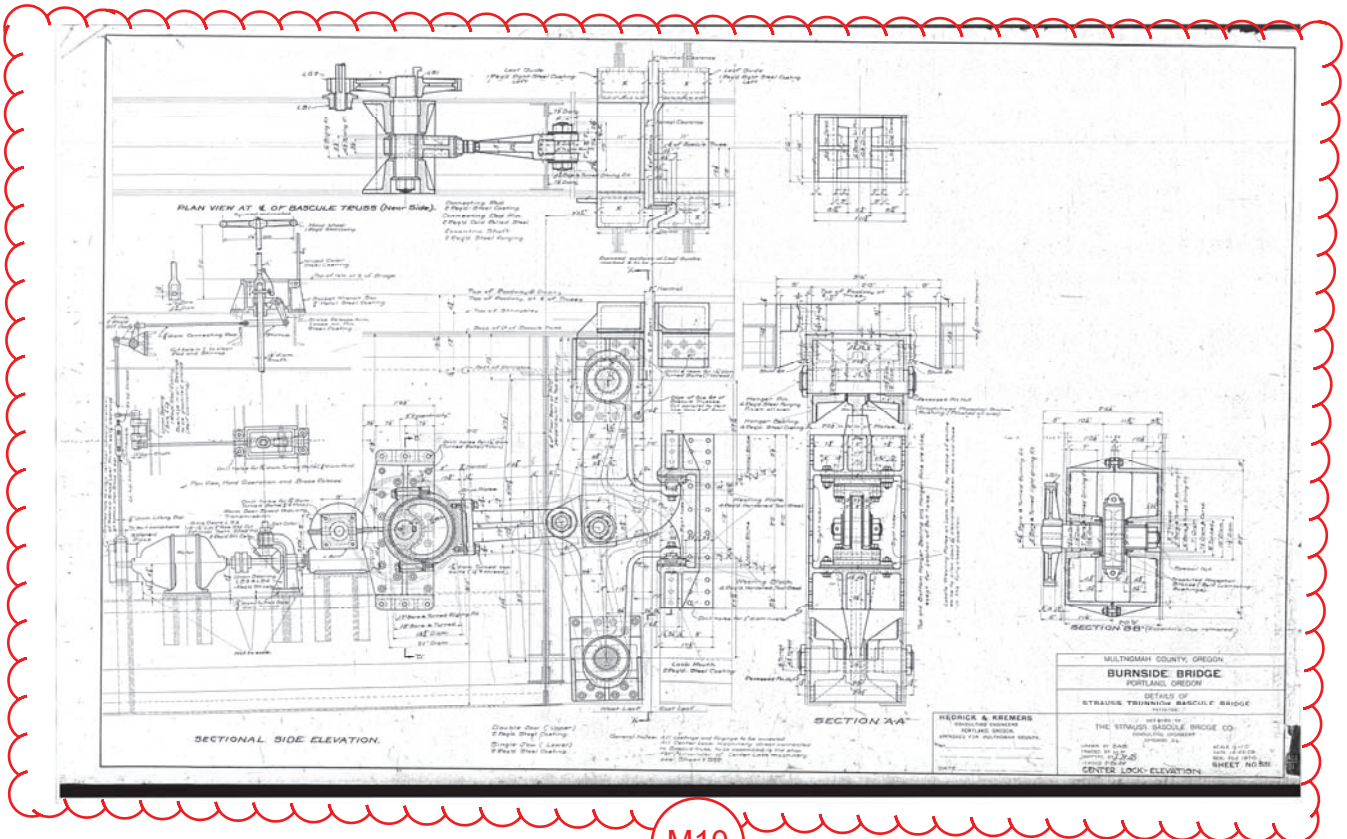
M8

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



M9

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



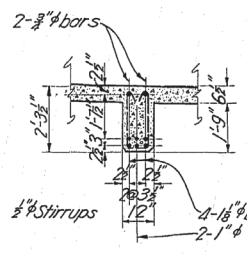
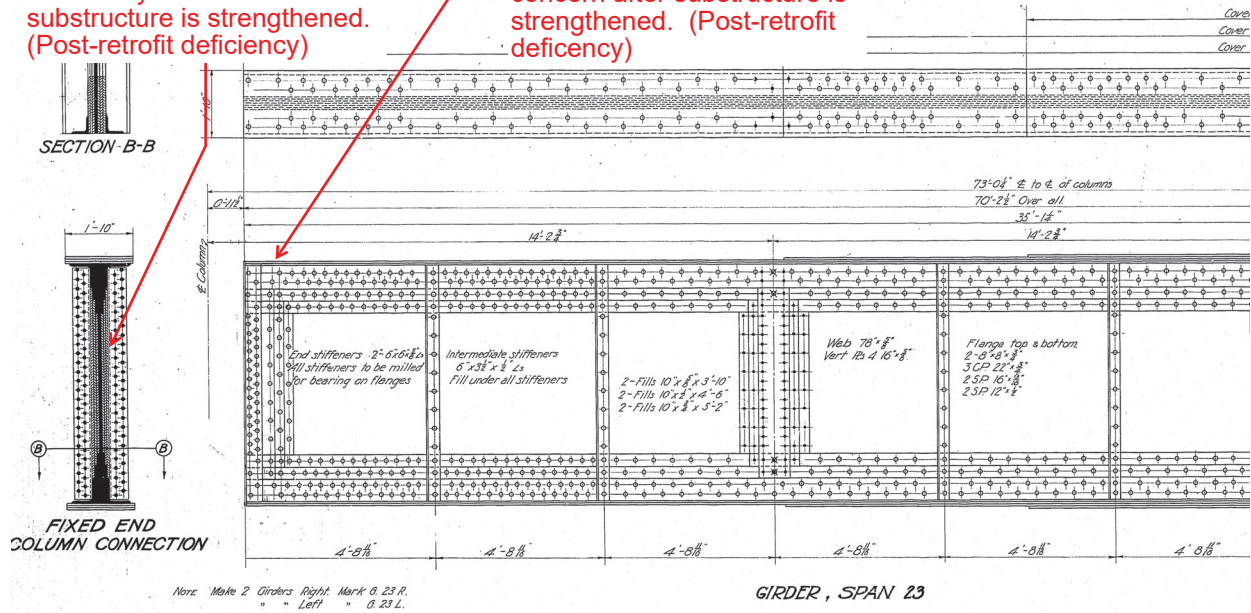
M10

M10

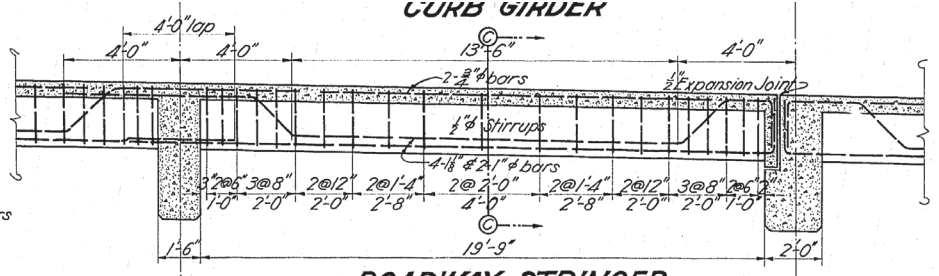
The center lock was not designed to transmit forces caused by the relative displacement of the two bascule spans, and may be severely damaged in a significant seismic event.

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

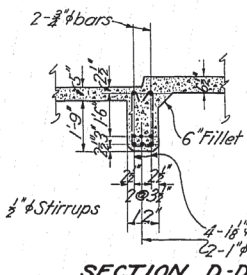
Reduced flange section at fixed end impacts moment capacity of girder / floorbeam connection. Potentially a concern after substructure is strengthened. (Post-retrofit deficiency)



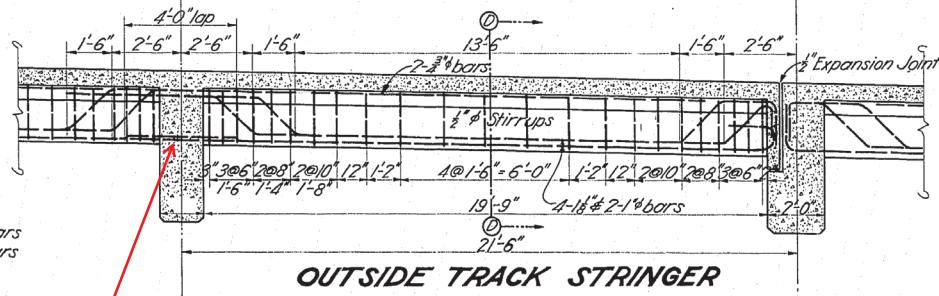
SECTION C-C



ROADWAY STRINGER



SECTION D-D

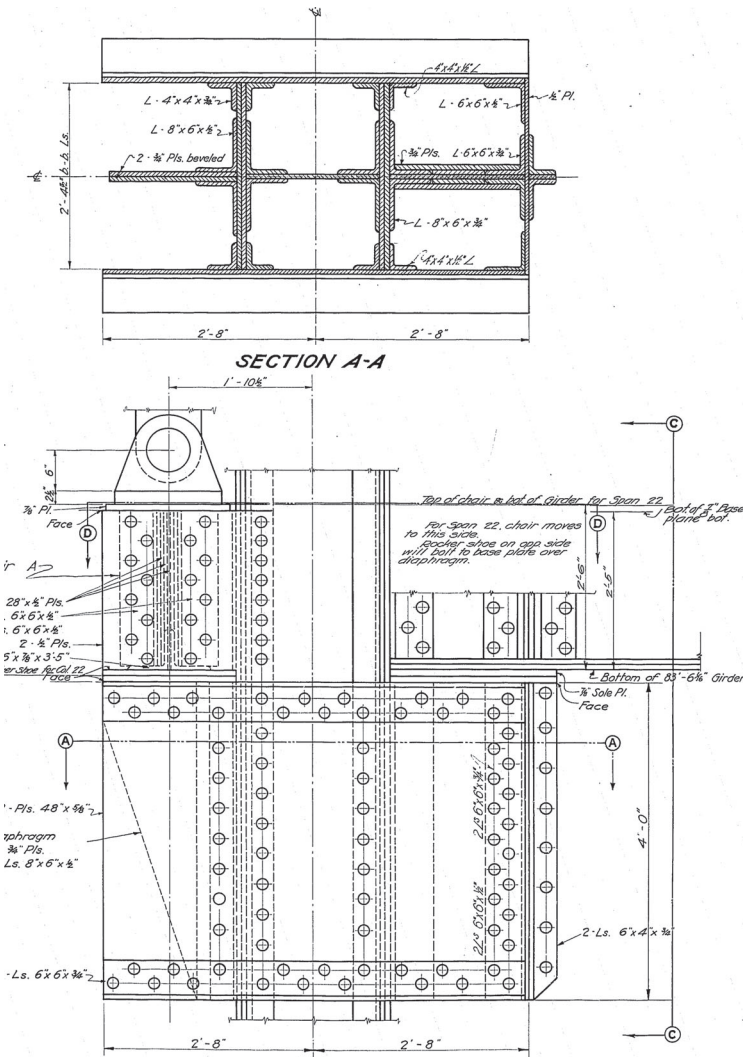


OUTSIDE TRACK STRINGER

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

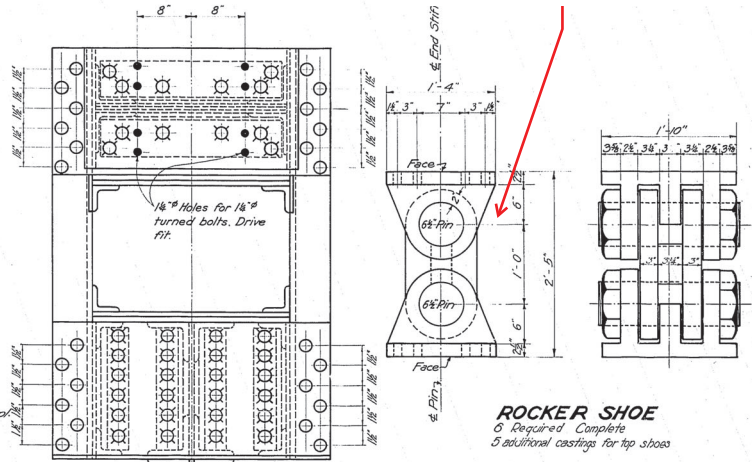


Rocker bearings are not stable for large displacements and likely to tip over. Also the rocker was not designed to restrict transverse movement.

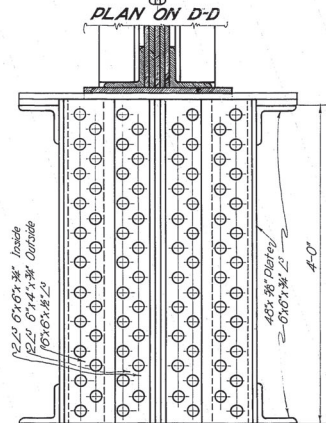


ELEVATIONS COL. 24

For Columns Bent-22 use same detail with chair 1/4" on east side and expansion rocker on west side of columns.



ROCKER SHOE
6 Required Complete
3 additional castings for top shoes



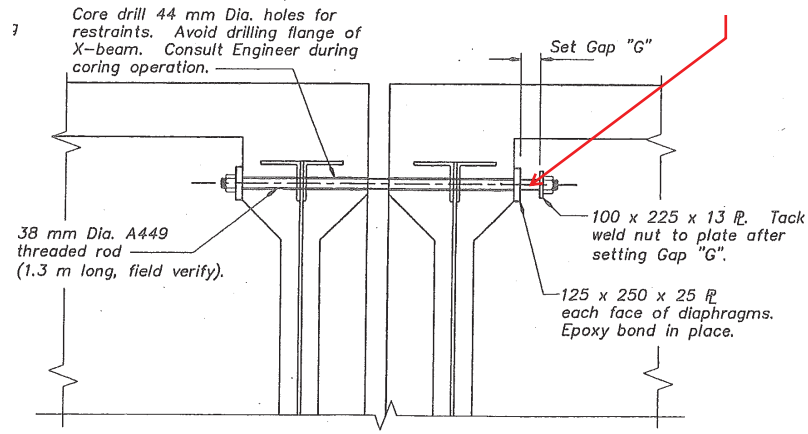
SECTION C-C

Note: All rivets 1" Diam in Details Shown Here.

See Shop Plans, American Bridge Co. Order 15882.

MULTNOMAH COUNTY, OREGON	
BURNSIDE BRIDGE	
PORTLAND, OREGON	
DETAILS	
EXPANSION JOINT, BENT 22-24	
MADE BY <i>RJH</i>	HEDRICK & KRÉMERS
CHECKED BY <i>DK</i>	CONSULTING ENGINEERS
TRACED BY <i>WJS</i>	PORTLAND, OREGON
CHECKED BY <i>RJH</i>	DATE FEB. 20, 1924

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



LONGITUDINAL RESTRAINT DETAIL

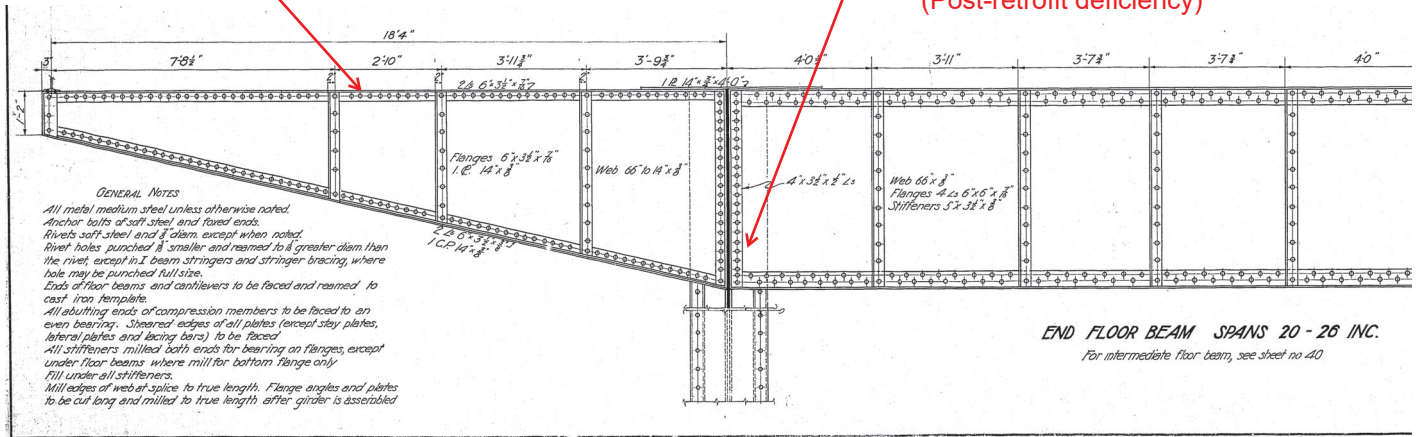
Scale: 1:10



Re No. 0511, 0511A, & 0511

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

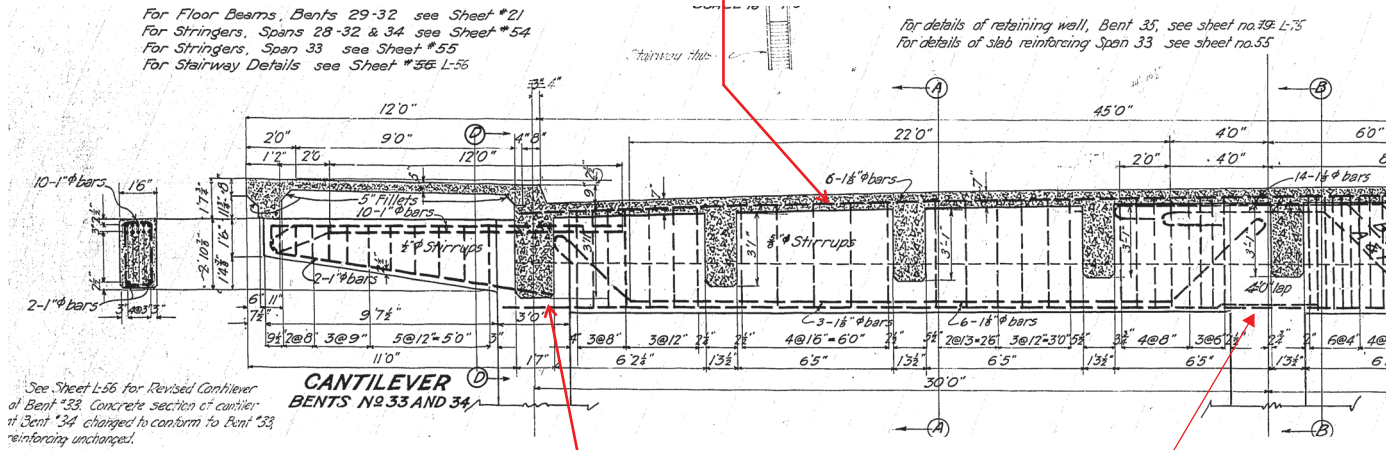
Riveted connection not originally designed to resist additional moment from seismic loading. Potentially a concern after substructure is strengthened. (Post-retrofit deficiency)



NOTICE: IF THIS DOCUMENT IS LESS LEGIBLE THAN THIS NOTICE IT IS DUE TO THE QUALITY OF THE ORIGINAL.



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

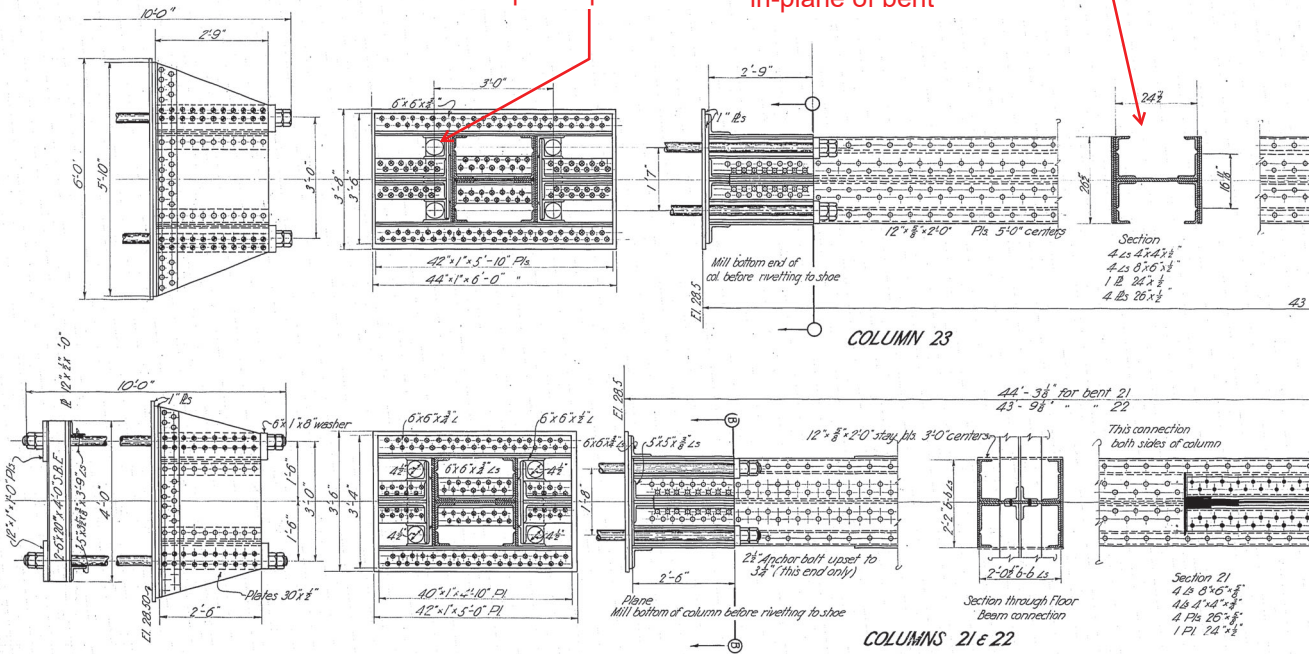


Poor seismic detail. Floorbeam positive moment reinforcement spliced or has limited embedment at column connection thus limiting moment capacity due to inadequate development length.



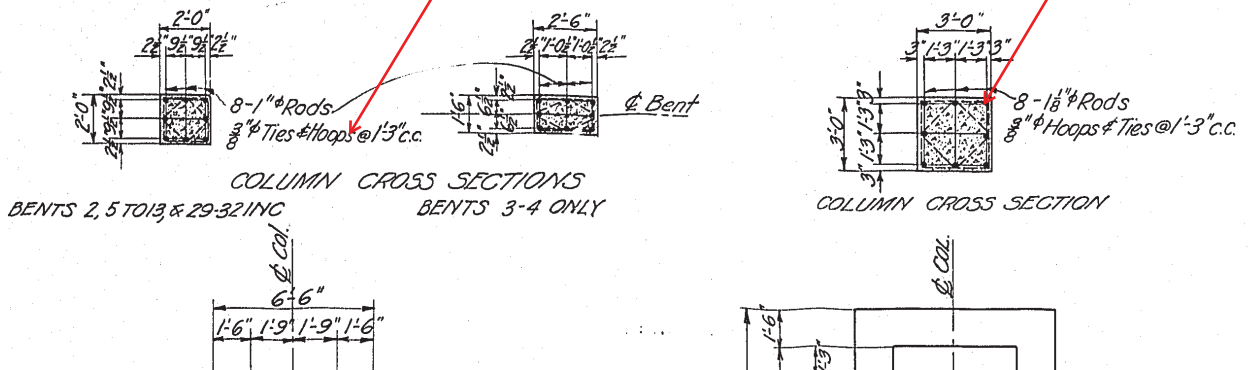
Column poorly anchored to footing pedestal. Anchors do not extend into the pile cap.

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

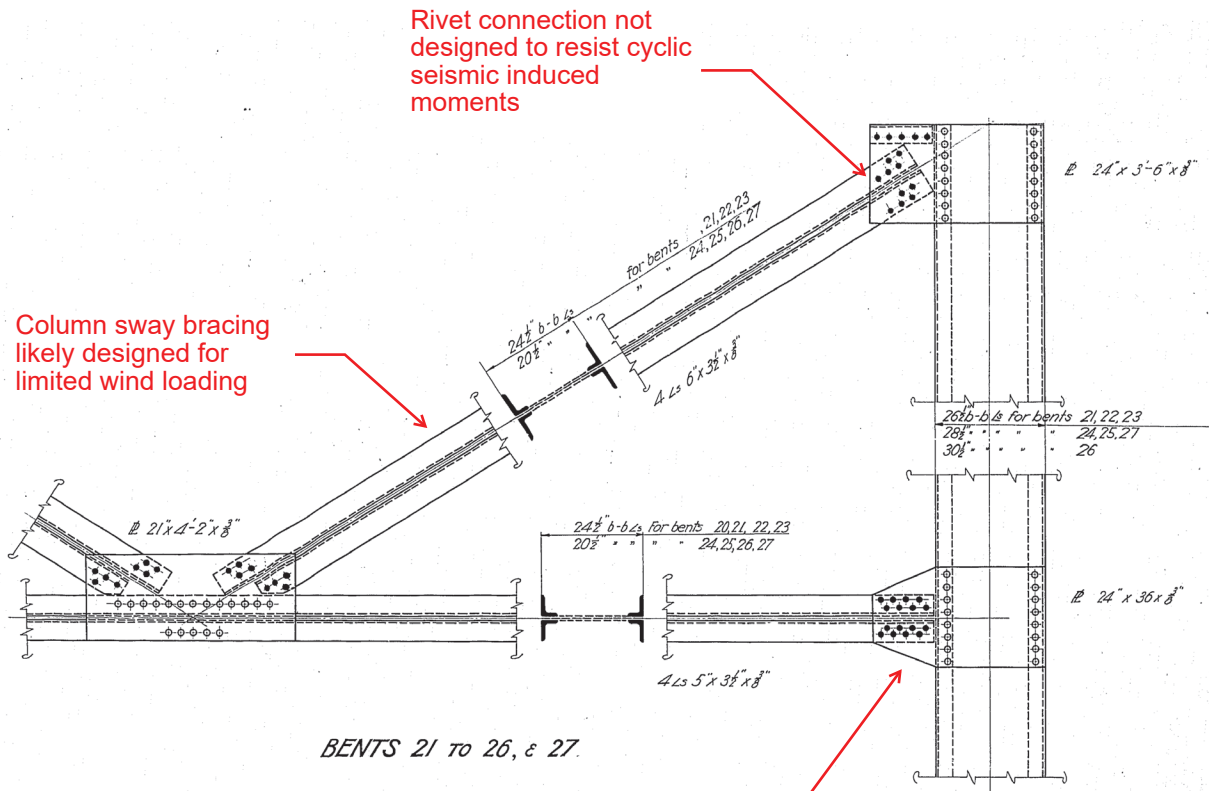


Poor confinement; Lack of seismic hooks; Ties and hoops @ 1'-3"

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



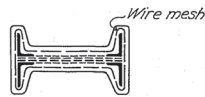
E4



Rivet connection not designed to resist cyclic seismic induced moments

Column sway bracing likely designed for limited wind loading

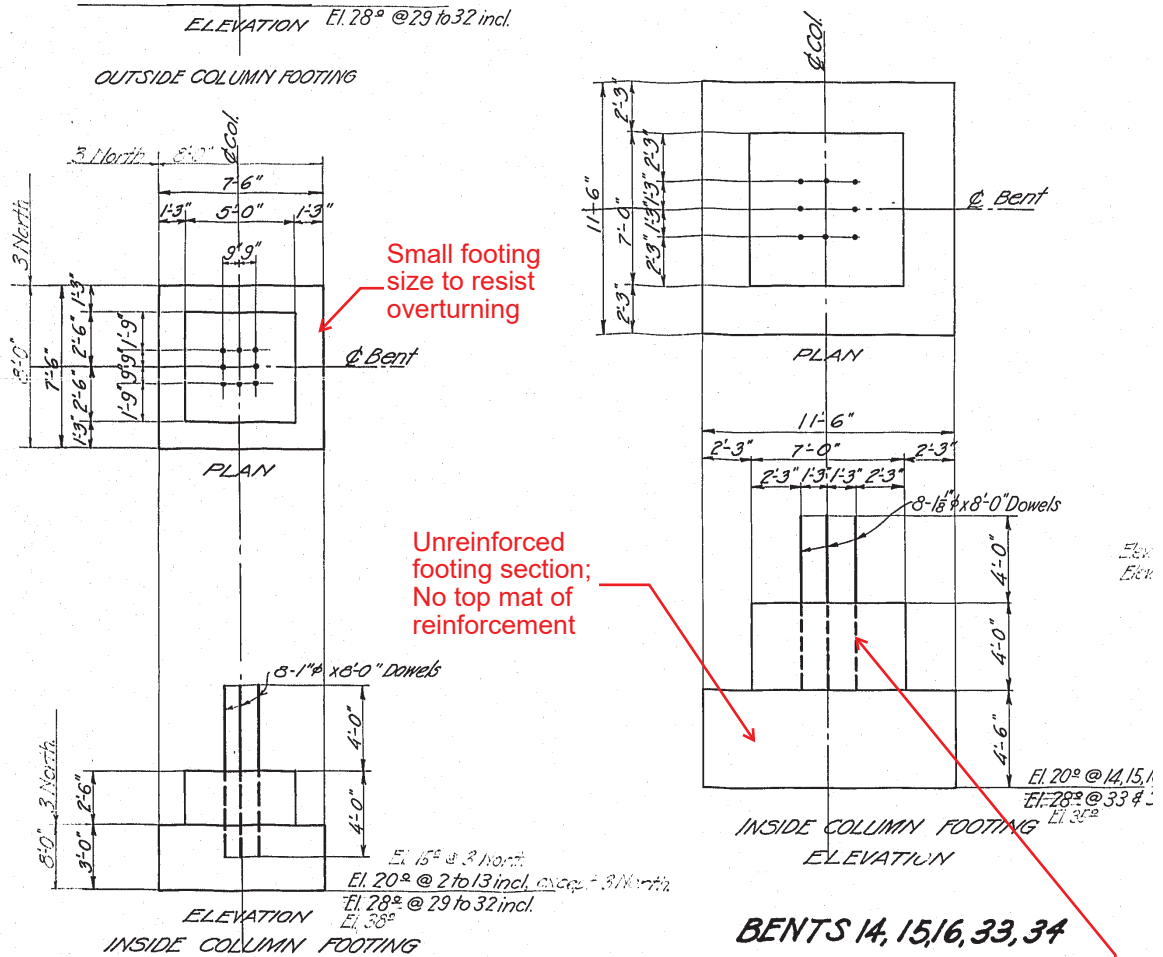
BENTS 21 to 26, & 27



Encase all members of sway bracing in concrete as above

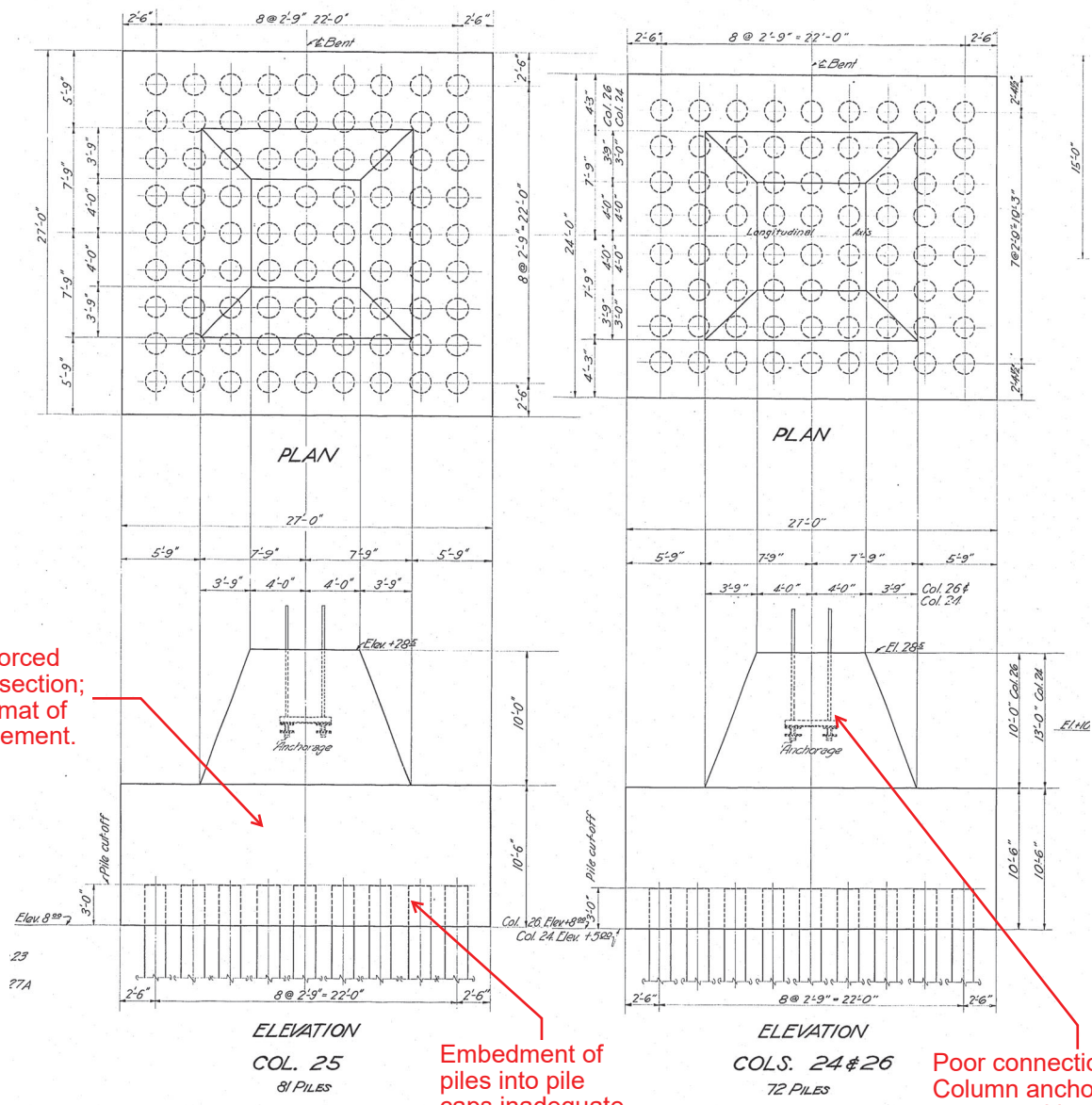
Sway bracing horizontal located at approximately mid-height of the column and stiffens the bent.

E5



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





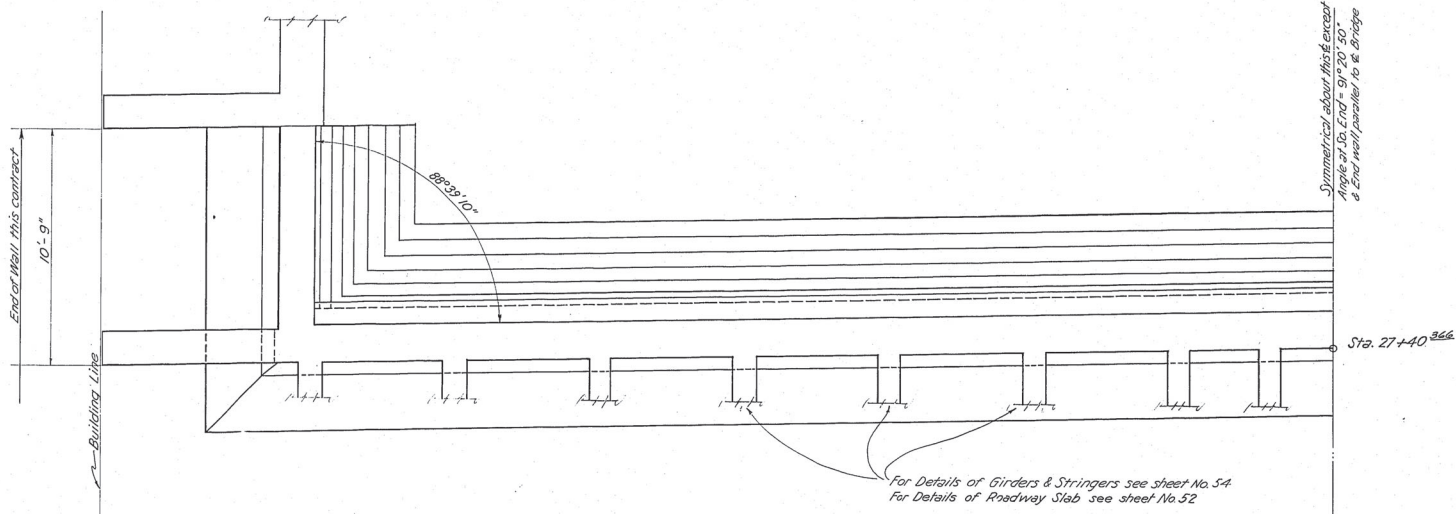
Unreinforced footing section; No top mat of reinforcement.

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

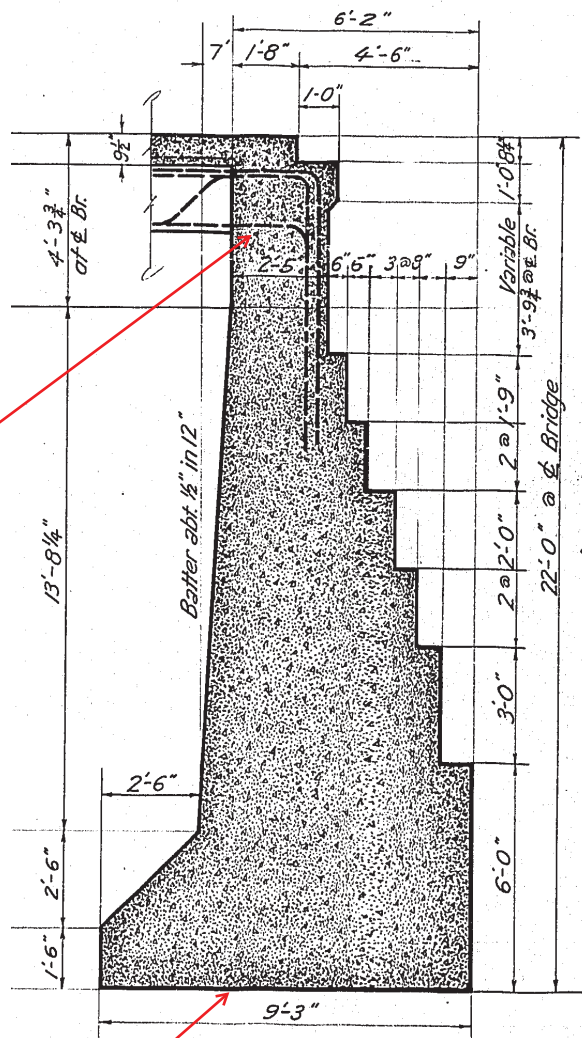
Poor connection detail; Column anchorage does not extend into pile cap with adequate embedment.

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





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

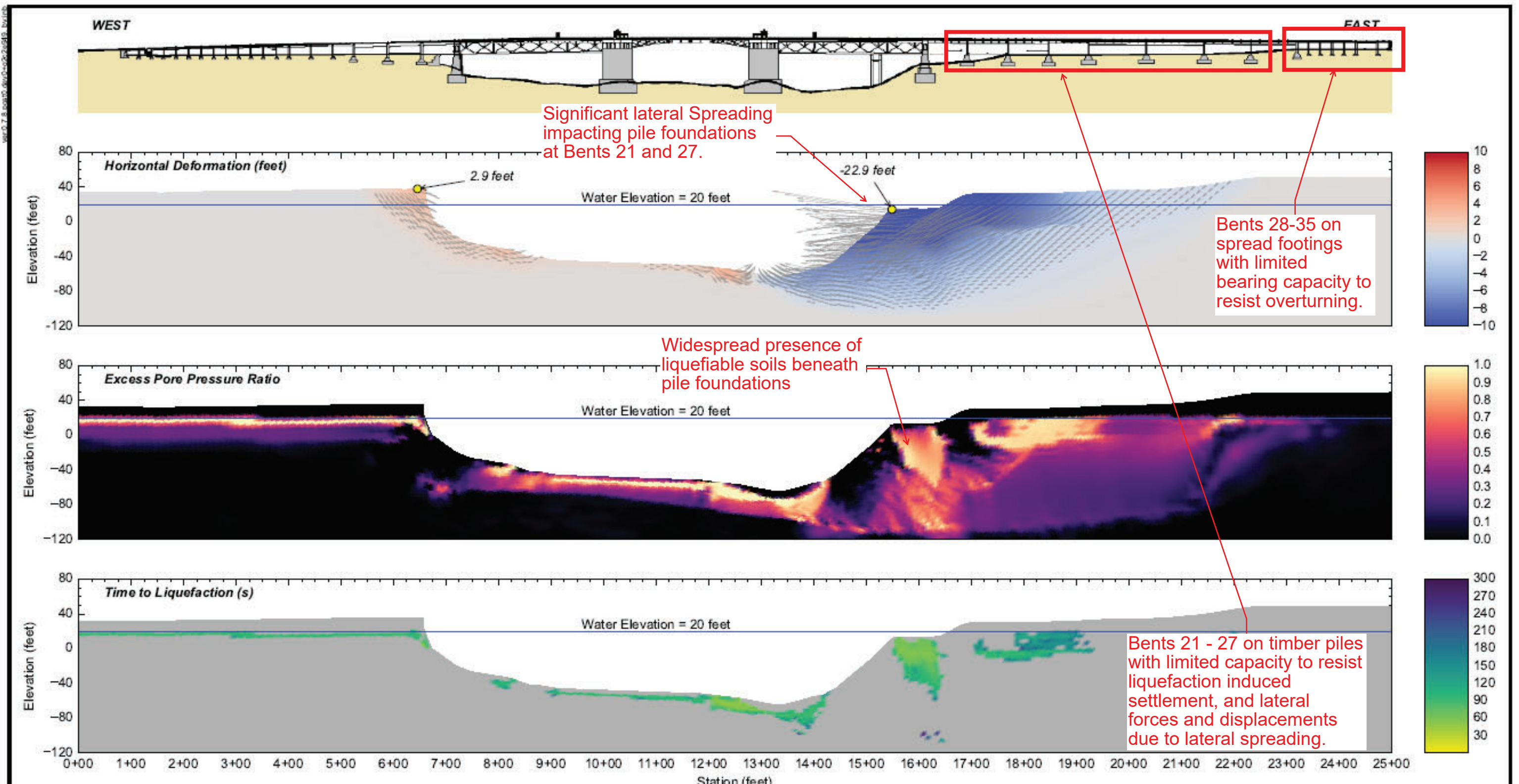


SECTION B-B

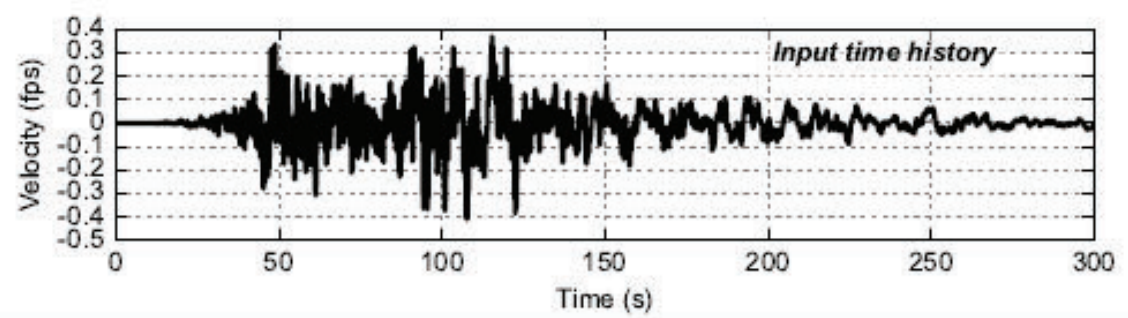
Soil Structure Interaction: Liquefaction induced settlement and lateral spreading not anticipated at Bent 35.

Narrow footing size to resist overturning.





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



Burnside Bridge
Portland, Oregon

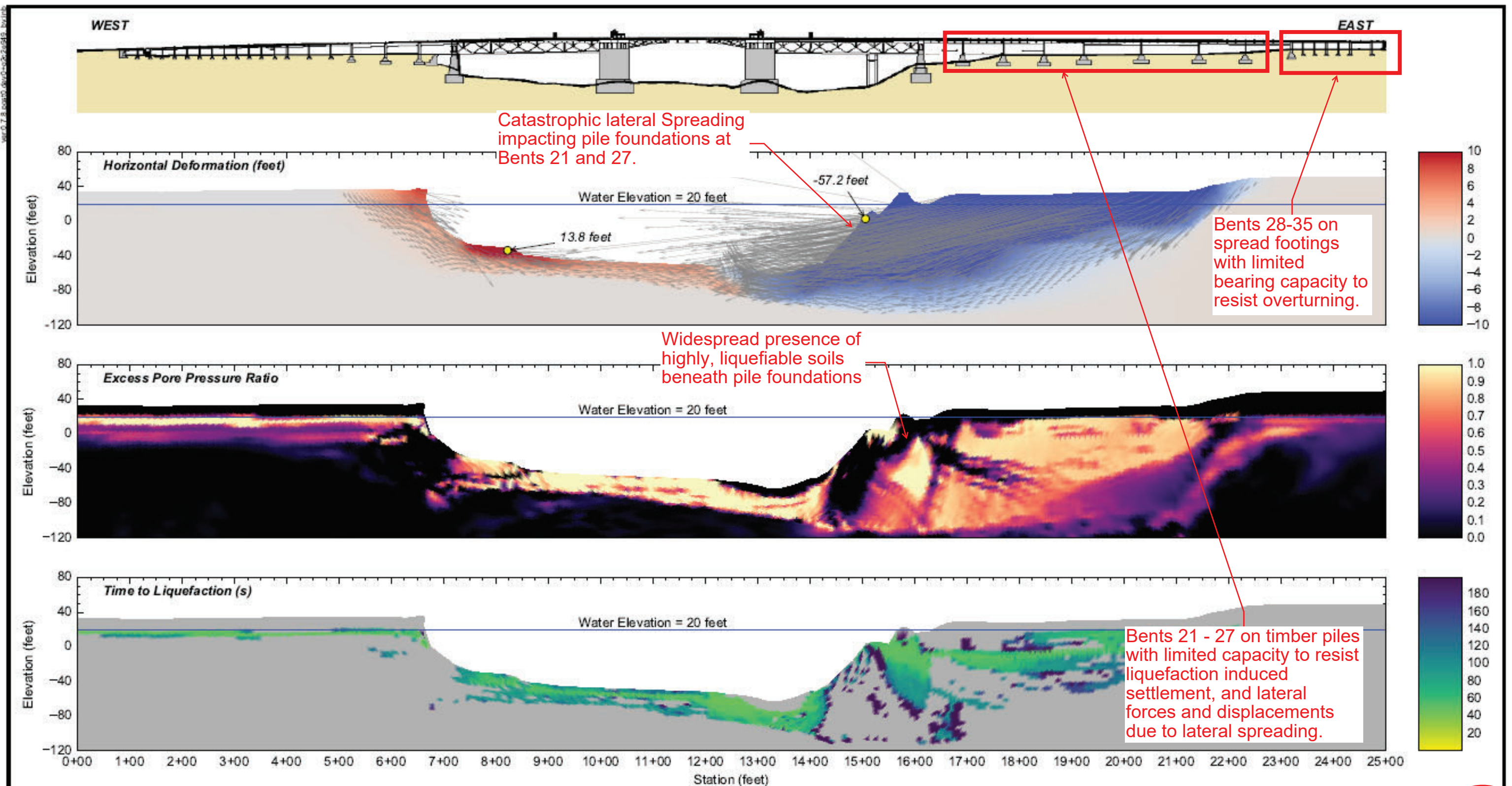
**E9
CSZ**

**MODEL RESPONSE
TIME HISTORY T-CSZ**

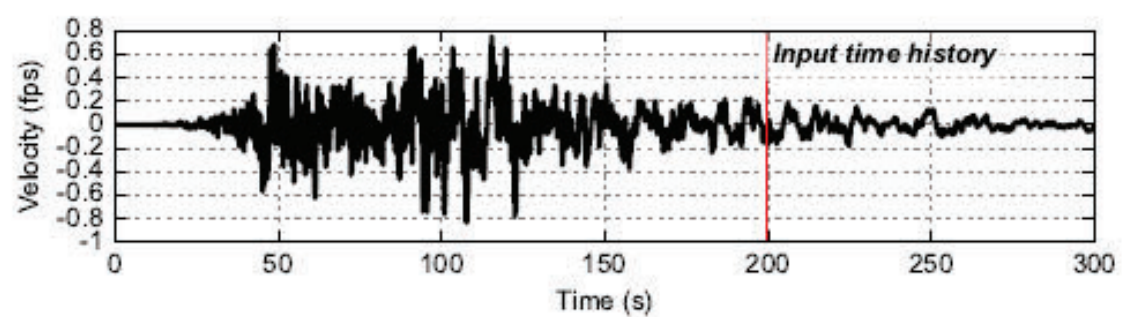
January 2017 24-1-04065-003

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6



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



Burnside Bridge
Portland, Oregon

**MODEL RESPONSE
TIME HISTORY T-1000**

January 2017 24-1-04065-003

SHANNON & WILSON, INC. FIG. 2
Geotechnical and Environmental Consultants

E9
1000-yr

Burnside Bridge



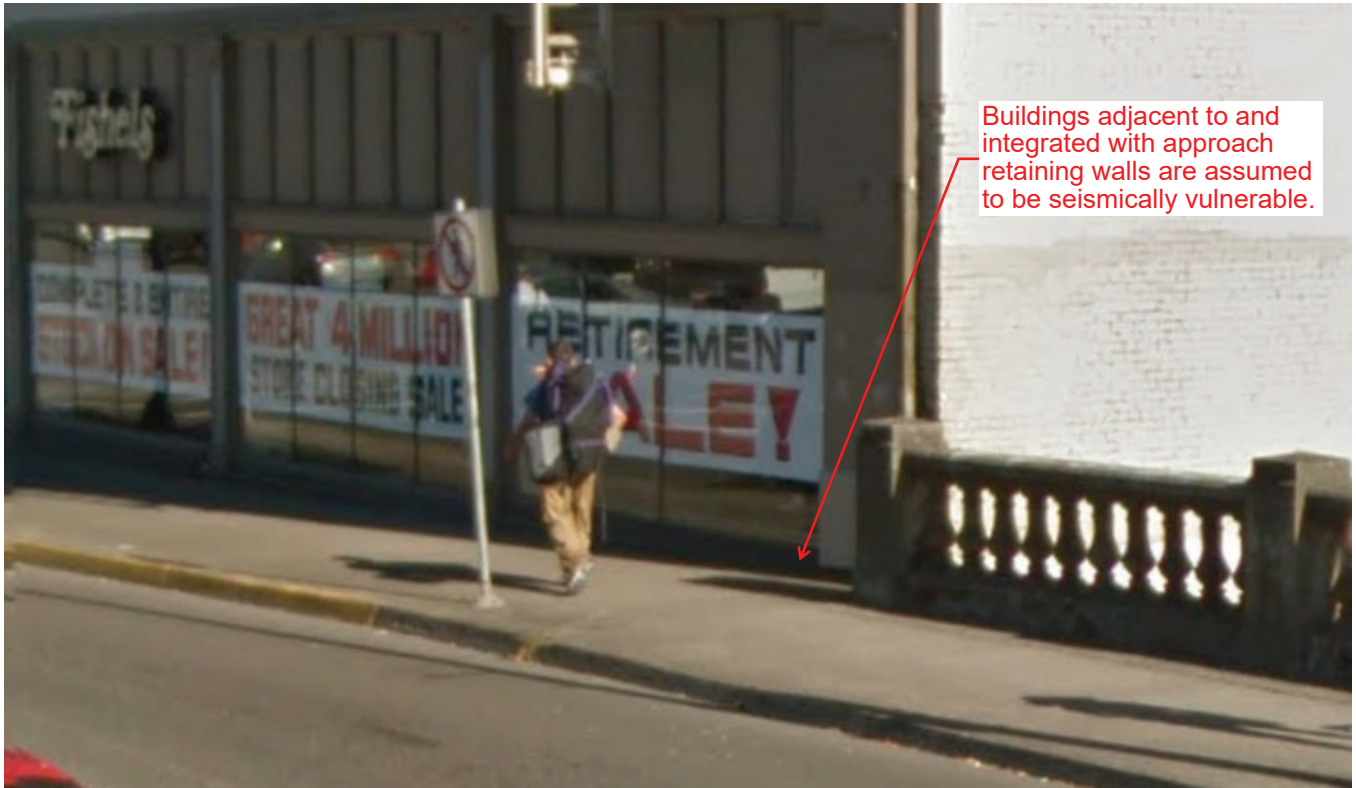
I-84 ramp to I-5 Southbound adjacent to Burnside Bridge substructure

I-5 Southbound adjacent to Burnside Bridge substructure

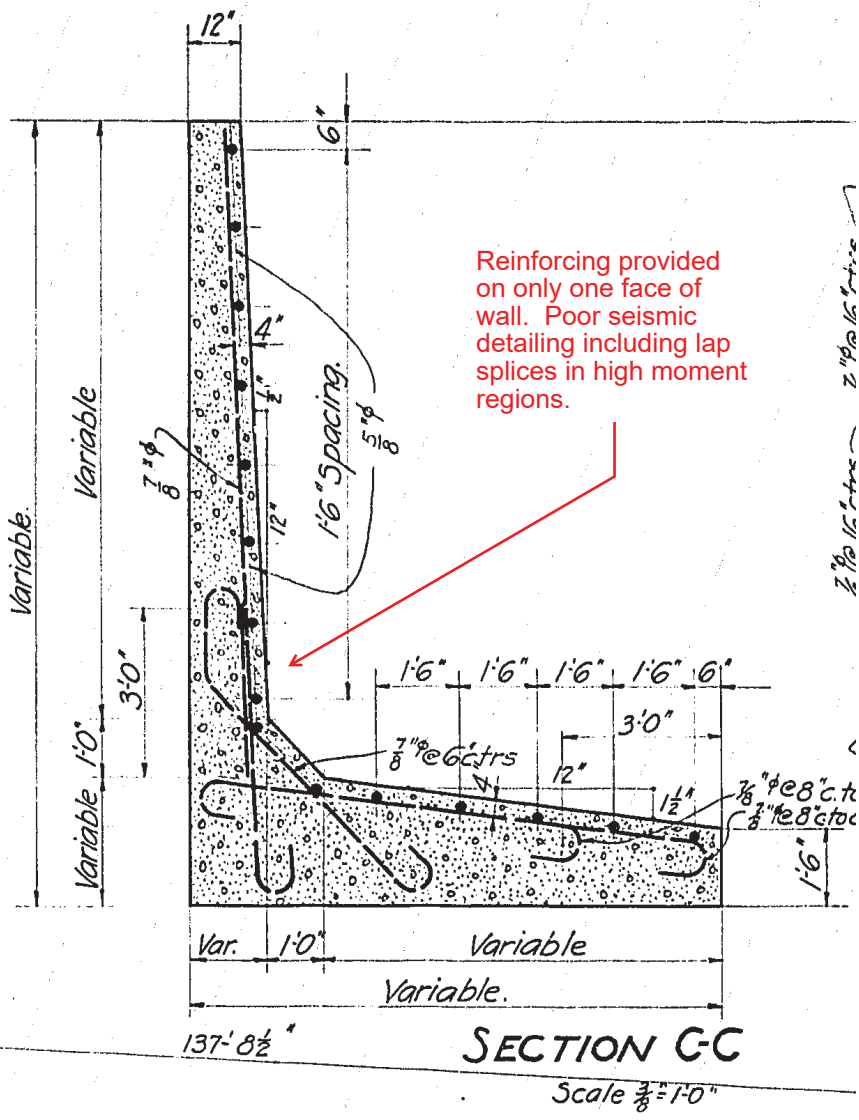
I-5 Northbound ramp to I-84 and I-5 Northbound adjacent to Burnside Bridge substructure

I-5 Northbound ramp to I-84 adjacent to Burnside Bridge substructure

E10



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

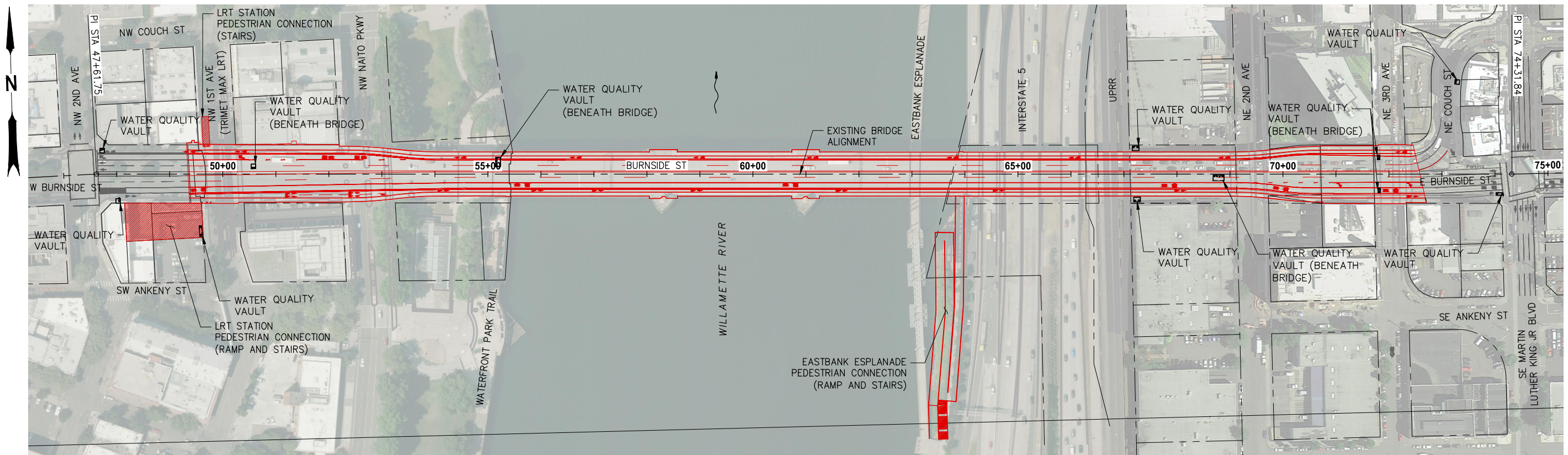


Reinforcing provided on only one face of wall. Poor seismic detailing including lap splices in high moment regions.

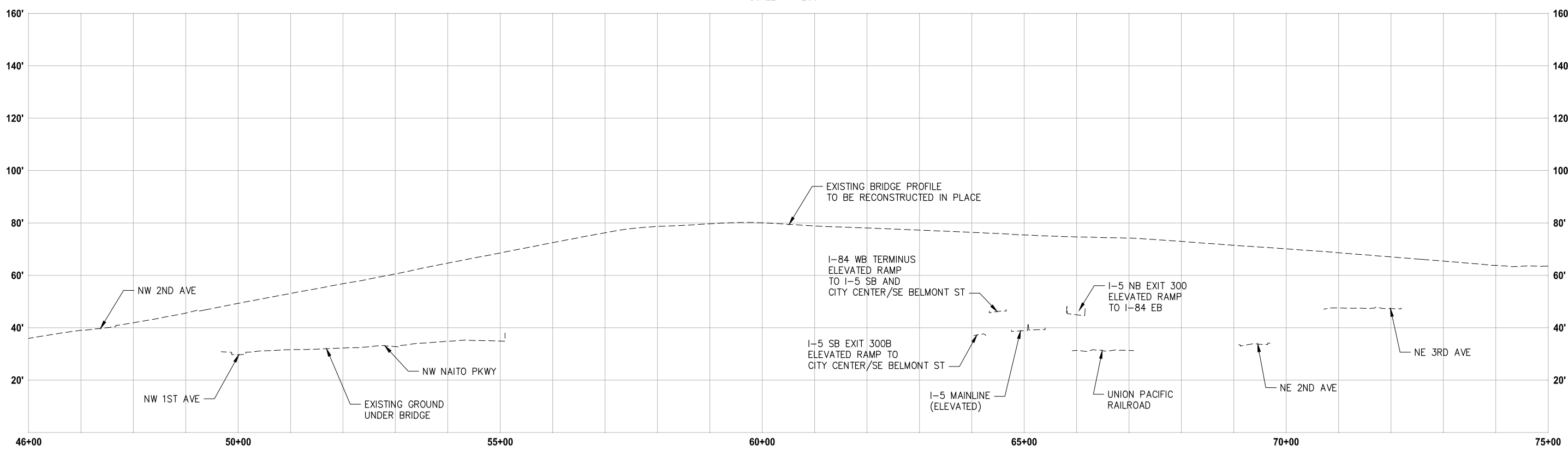
Soil Structure Interaction: Liquefaction induced settlement and lateral spreading not anticipated at East Approach retaining walls.

E11

Appendix C. Enhanced Seismic Retrofit Conceptual Plans

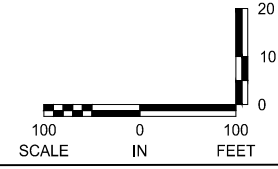


PLAN
SCALE: 1"=200'



PROFILE
HORIZONTAL SCALE: 1"=200'
VERTICAL SCALE: 1"=40'

CONCEPTUAL PLANS
JUNE 2020



MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97235-5989

Earthquake Ready Burnside Bridge NEPA Phase
Enhanced Seismic Retrofit

MULTNOMAH COUNTY
DATE: _____ DATE: _____
PROJECT NO.: Proj No _____

MULTNOMAH COUNTY
COUNTY ENGINEER
IAN B. CANNON P.E.

NO.	DATE	REVISIONS

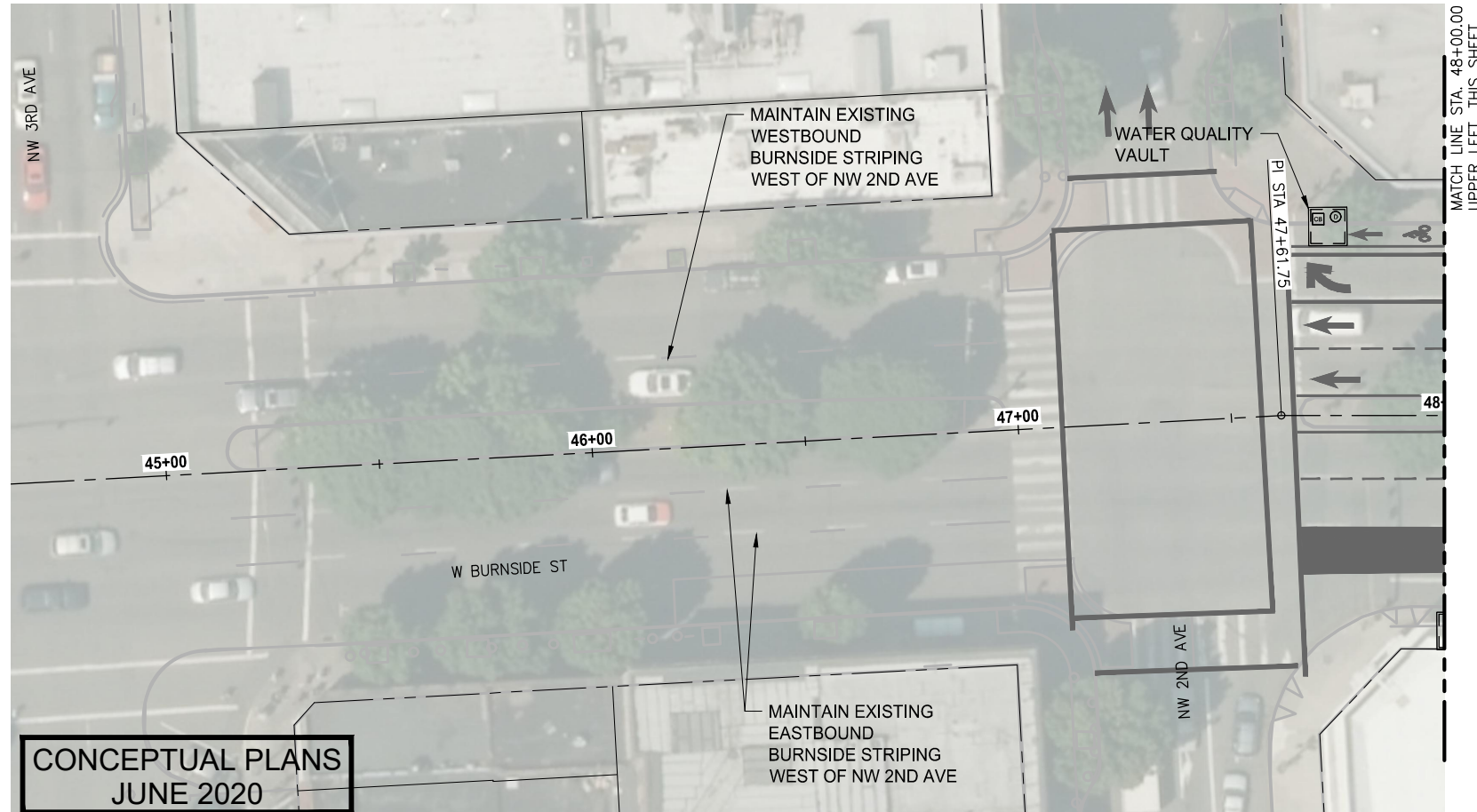
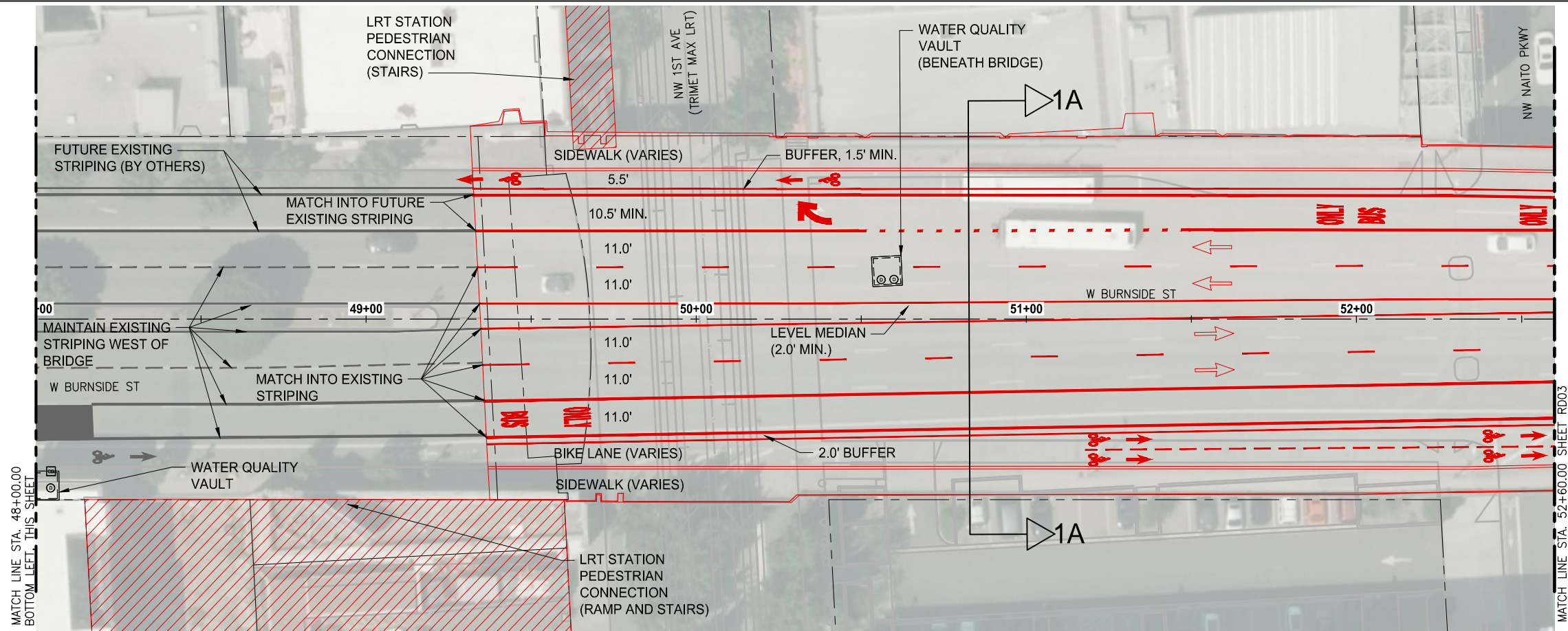
DESIGNED BY: _____
DRAFTED BY: _____
CHECKED BY: _____

Sheet No. **RD01**



PLAN
SCALE: 1"=40'

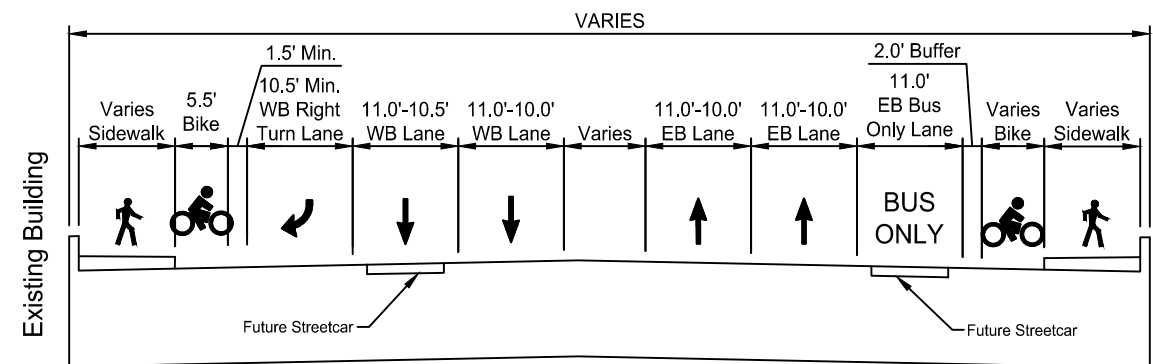
20 0 20
SCALE IN FEET



CONCEPTUAL PLANS
JUNE 2020

PLAN
SCALE: 1"=40'

20 0 20
SCALE IN FEET

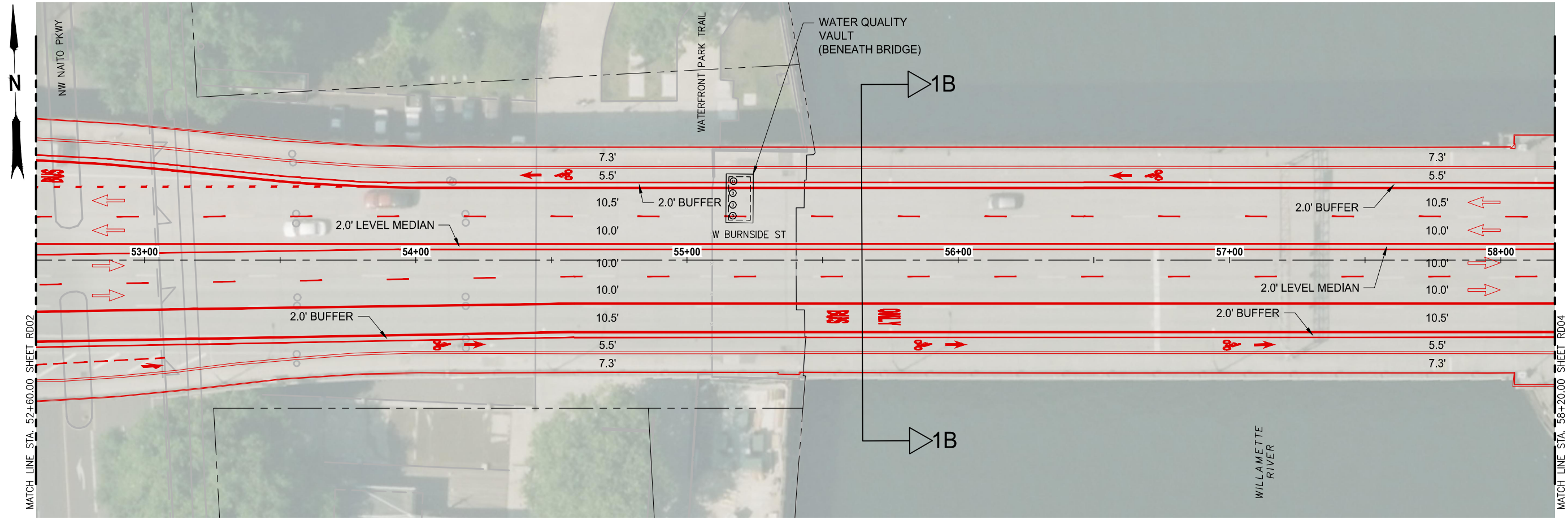


TYPICAL SECTION 1A
WEST APPROACH ON STRUCTURE
LOOKING EAST - NO SCALE

Earthquake Ready Burnside Bridge NEPA Phase
Enhanced Seismic Retrofit
MULTNOMAH COUNTY
DATE: _____ DATE: _____
PROJECT NO.: Proj No

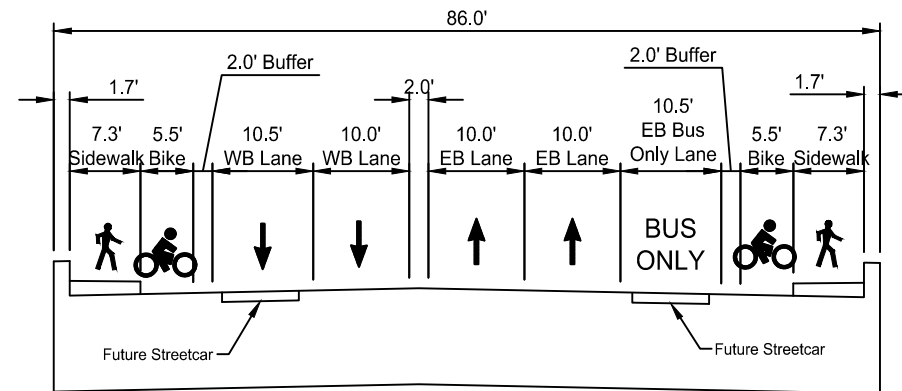
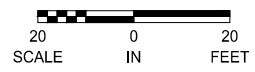
MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97235-5989
IAN B. CANNON P.E.
COUNTY ENGINEER

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DRAFTED BY:	_____ NO DATE:
CHECKED BY:	_____ NO DATE:
REVISIONS	
NO DATE:	
Sheet No.	RD02



PLAN

SCALE: 1"=40'



TYPICAL SECTION 1B

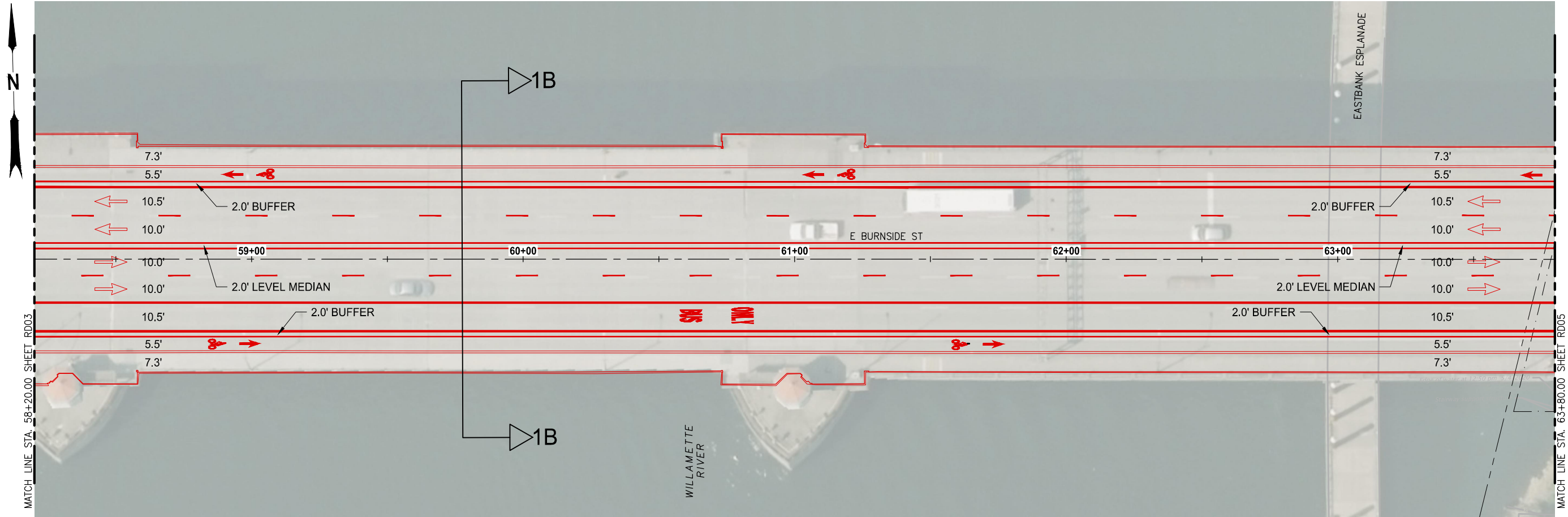
BRIDGE SPANS
LOOKING EAST - NO SCALE

**CONCEPTUAL PLANS
JUNE 2020**

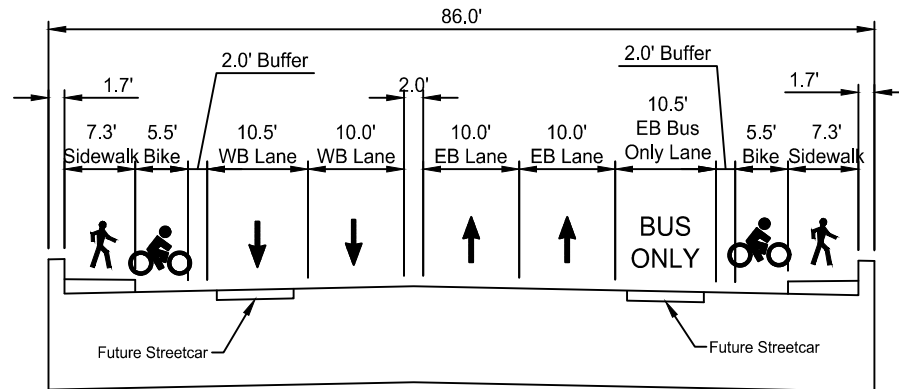
Earthquake Ready Burnside Bridge NEPA Phase
Enhanced Seismic Retrofit
MULTNOMAH COUNTY
DATE: Date PROJECT NO.: Proj No

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999
IAN B. CANNON P.E.
COUNTY ENGINEER

DESIGNED BY:	
DRAFTED BY:	
CHECKED BY:	
NO. DATE:	
REVISIONS	
Sheet No.	RD03



PLAN
SCALE: 1"=40'



TYPICAL SECTION 1B
BRIDGE SPANS
LOOKING EAST - NO SCALE

CONCEPTUAL PLANS
JUNE 2020

Earthquake Ready Burnside Bridge NEPA Phase
Enhanced Seismic Retrofit
MULTNOMAH COUNTY
DATE: _____ DATE: _____
PROJECT NO.: Proj No

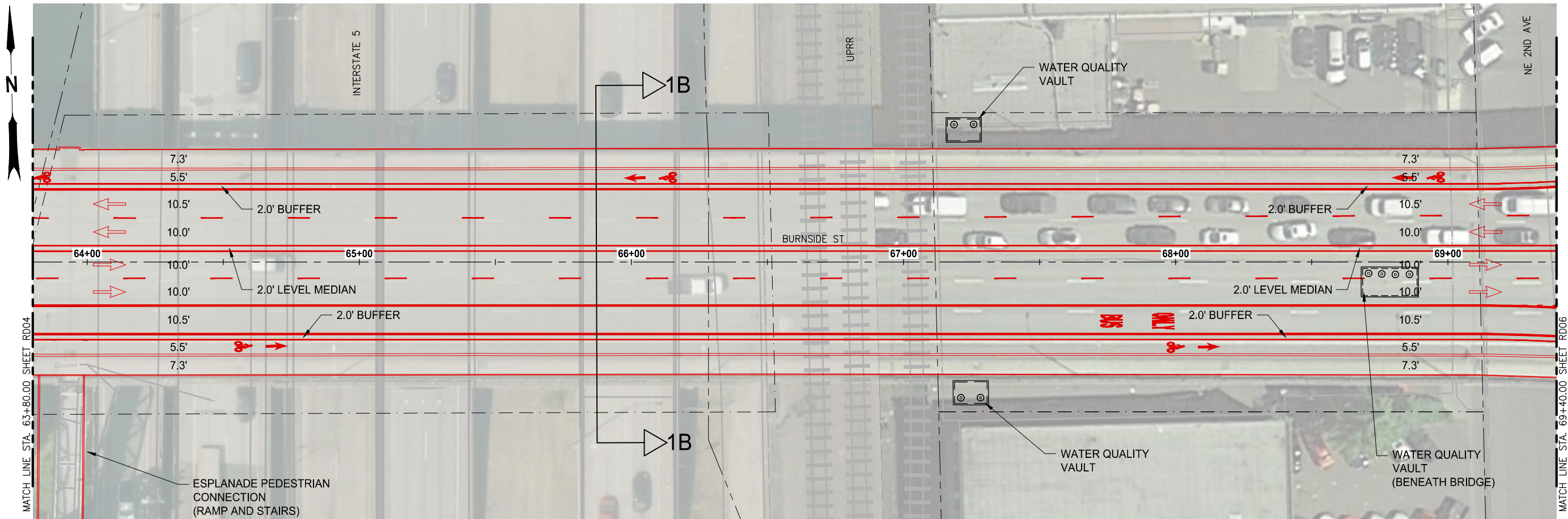
MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97235-5989

IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: _____
DRAFTED BY: _____
CHECKED BY: _____

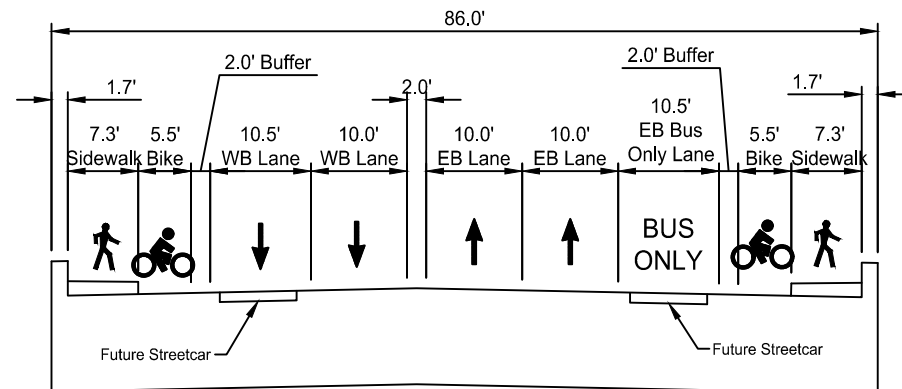
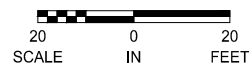
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NO. DATE:	

Sheet No. **RD04**



PLAN

SCALE: 1"=40'




TYPICAL SECTION 1B

BRIDGE SPANS
LOOKING EAST - NO SCALE

**CONCEPTUAL PLANS
JUNE 2020**

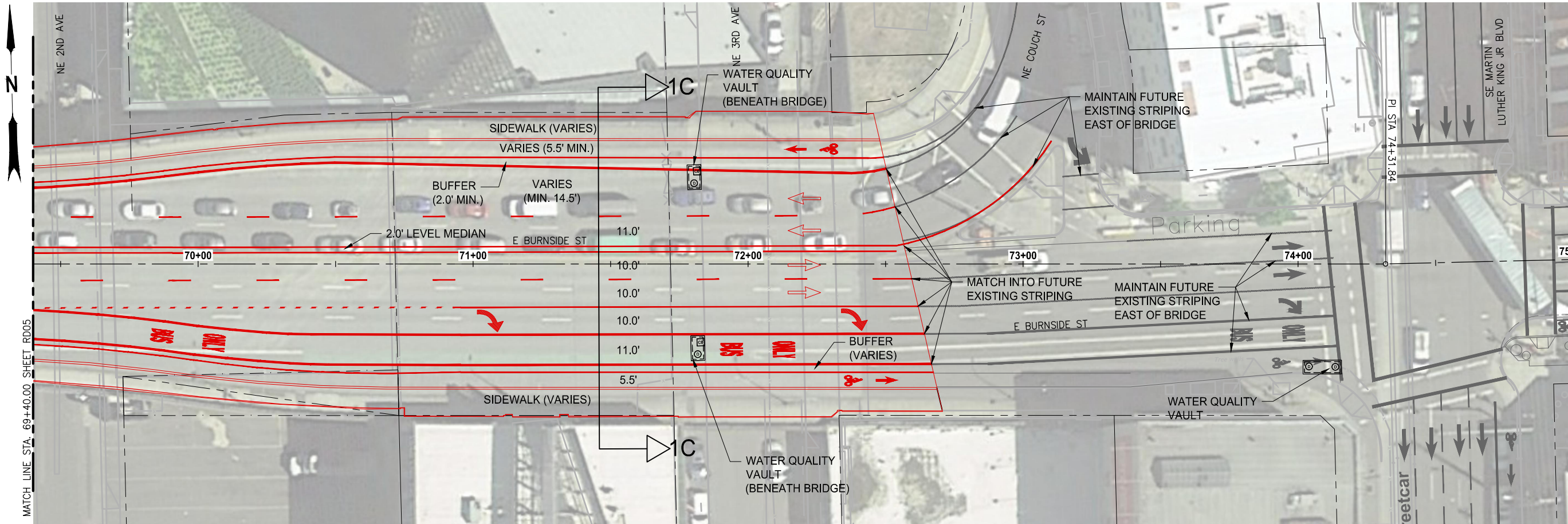
Earthquake Ready Burnside Bridge NEPA Phase
Enhanced Seismic Retrofit
MULTNOMAH COUNTY
DATE: _____ DATE: _____
PROJECT NO.: Proj/No

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999

IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: _____
DRAFTED BY: _____
CHECKED BY: _____

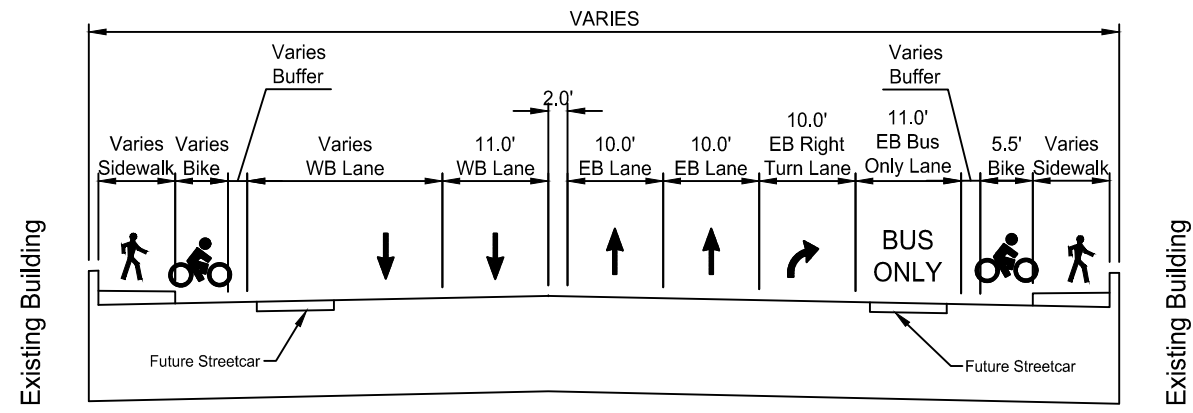
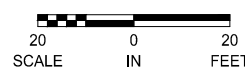
NO.	DATE:	REVISIONS

Sheet No. **RD05**



PLAN

SCALE: 1"=40'

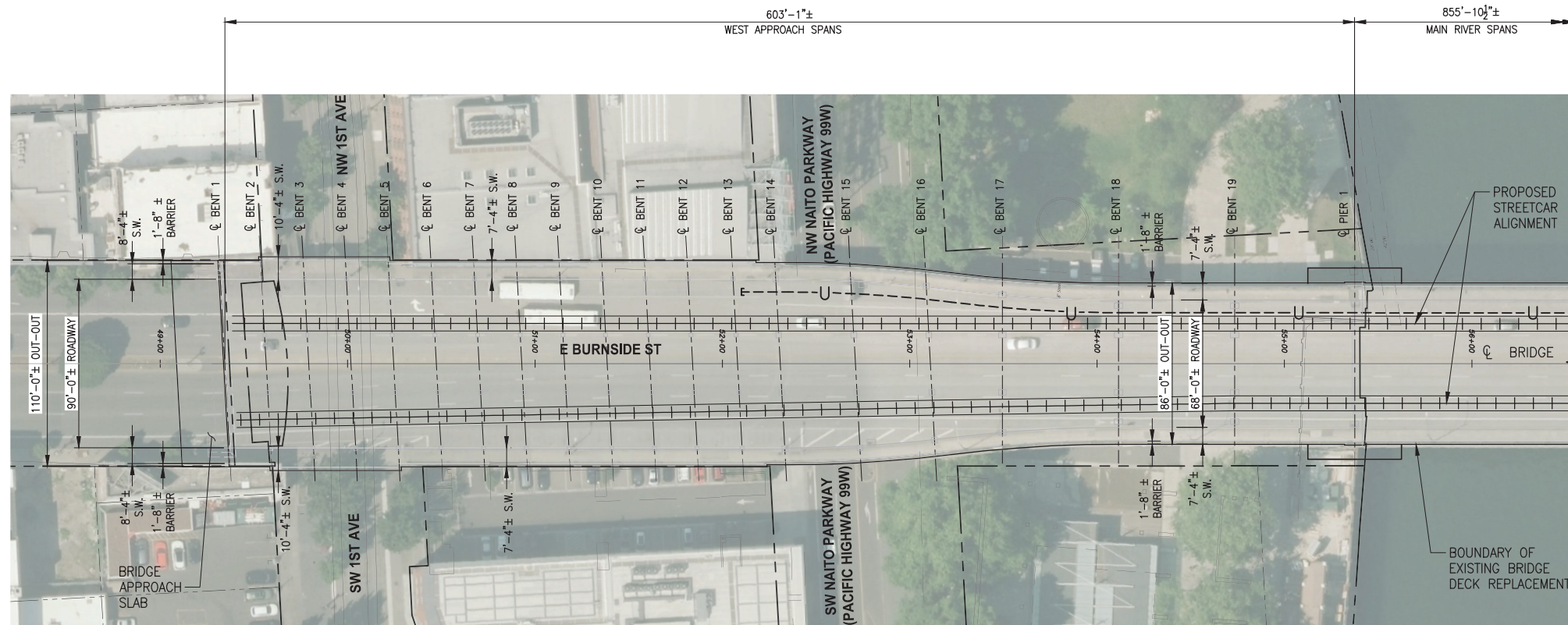


TYPICAL SECTION 1C

EAST APPROACH ON STRUCTURE
LOOKING EAST - NO SCALE

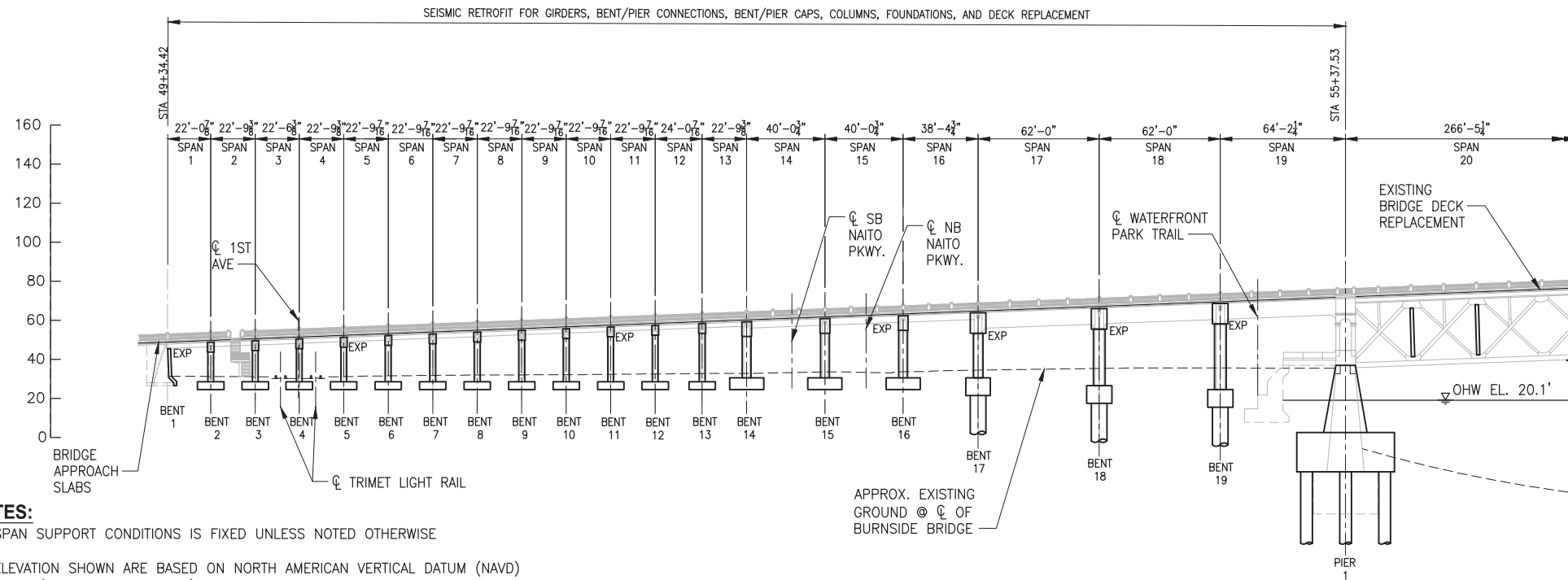
CONCEPTUAL PLANS
JUNE 2020

<p>MULTNOMAH COUNTY DEPARTMENT OF COMMUNITY SERVICES TRANSPORTATION DIVISION 1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999</p> <p>IAN B. CANNON P.E. COUNTY ENGINEER</p>		<p>MULTNOMAH COUNTY</p>															
<p>Earthquake Ready Burnside Bridge NEPA Phase Enhanced Seismic Retrofit</p>		<p>DATE: Date PROJECT NO.: Proj No</p>															
<p>DESIGNED BY: _____</p> <p>DRAFTED BY: _____</p> <p>CHECKED BY: _____</p>	<p>REVISIONS</p> <table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th style="width: 10%;">NO.</th> <th style="width: 10%;">DATE:</th> <th style="width: 80%;">REVISIONS</th> </tr> </thead> <tbody> <tr><td> </td><td> </td><td> </td></tr> <tr><td> </td><td> </td><td> </td></tr> <tr><td> </td><td> </td><td> </td></tr> <tr><td> </td><td> </td><td> </td></tr> </tbody> </table>		NO.	DATE:	REVISIONS												
NO.	DATE:	REVISIONS															
<p>Sheet No. RD06</p>		<p>PROJECT NO.: Proj No</p>															



PLAN

SCALE: 1" = 40'



ELEVATION

SCALE: 1" = 40'

LEGEND:

- EXISTING
- RETROFIT
- - - ROW

NOTES:

1. SPAN SUPPORT CONDITIONS IS FIXED UNLESS NOTED OTHERWISE
2. ELEVATION SHOWN ARE BASED ON NORTH AMERICAN VERTICAL DATUM (NAVD) 1988. (TYP FOR ALL SHEETS)
3. ANGLE FOR BENTS 1-16 = 93°05'30"
4. ANGLE FOR BENTS 17-19 AND PIER 1 = 90°00'00"

CONCEPTUAL PLANS
JUNE 2020



EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
PLAN AND ELEVATION
WEST APPROACH

MULTNOMAH COUNTY
 DEPARTMENT OF COMMUNITY SERVICES
 TRANSPORTATION DIVISION
 1620 S.E. 190th AVE. PORTLAND, ORE. 97235-5999

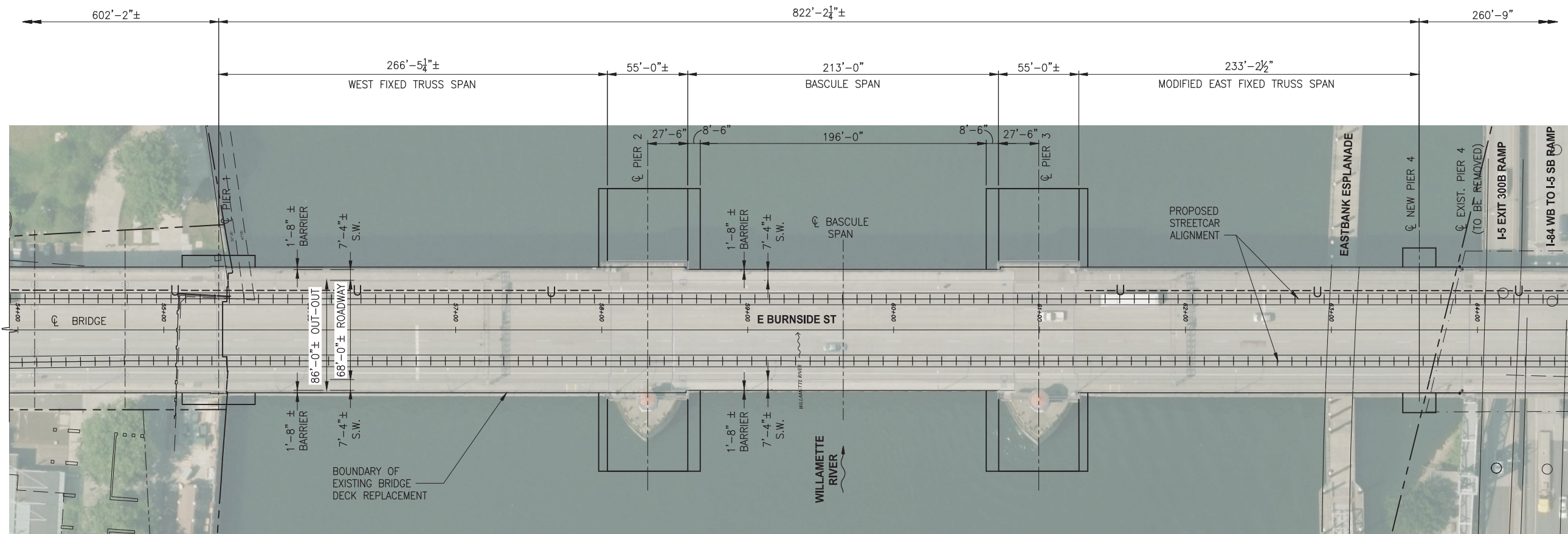
IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BJS/CRK
 DRAFTED BY: CRK/BRP
 CHECKED BY: YY

REVISIONS	
NO.	DATE:

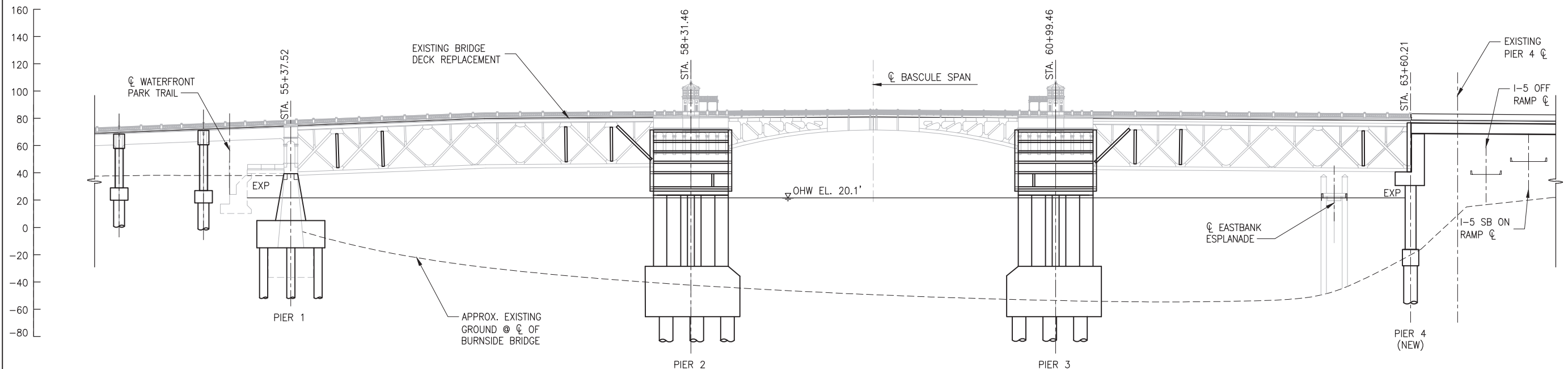
Sheet No. **S01**

DATE: 1/31/20 PROJECT NO.: Proj/No



PLAN

SCALE: 1" = 40'



LEGEND:

- EXISTING
- RETROFIT
- - - ROW

NOTES:

1. SPAN SUPPORT CONDITIONS IS FIXED UNLESS NOTED OTHERWISE
2. ELEVATION SHOWN ARE BASED ON NORTH AMERICAN VERTICAL DATUM (NAVD) 1988.

ELEVATION

SCALE: 1" = 40'

CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
PLAN AND ELEVATION
RIVER SPANS

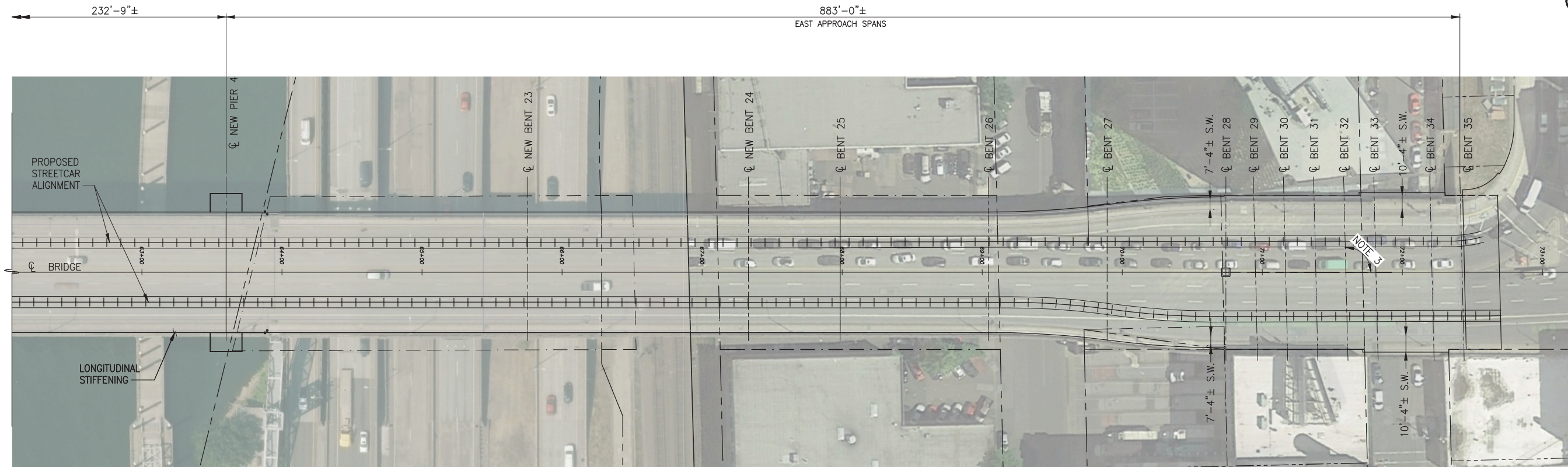
DATE: 1/31/20 PROJECT NO.: Proj/No

MULTNOMAH COUNTY
 DEPARTMENT OF COMMUNITY SERVICES
 TRANSPORTATION DIVISION
 1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5989

IAN B. CANNON P.E. COUNTY ENGINEER

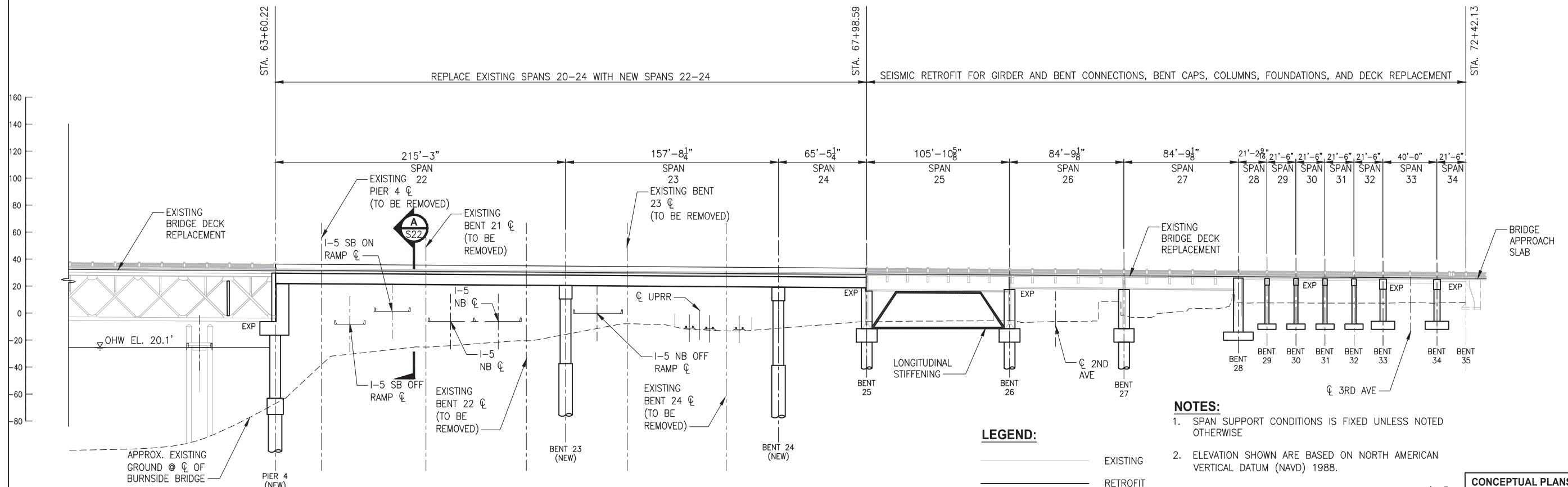
DESIGNED BY: BJS/CRK
 DRAFTED BY: CRK/BRP
 CHECKED BY: YY

REVISIONS	
NO.	DATE:



PLAN

SCALE: 1" = 40'



ELEVATION

SCALE: 1" = 40'

LEGEND:

- EXISTING
- RETROFIT
- - - ROW

NOTES:

1. SPAN SUPPORT CONDITIONS IS FIXED UNLESS NOTED OTHERWISE
2. ELEVATION SHOWN ARE BASED ON NORTH AMERICAN VERTICAL DATUM (NAVD) 1988.
3. ANGLES FOR PIER 4 AND BENTS 23-28 = 90°00'00"
4. ANGLES FOR BENTS 29-35 = 88°39'26"

CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
PLAN AND ELEVATION
EAST APPROACH

MULTNOMAH COUNTY
 DEPARTMENT OF COMMUNITY SERVICES
 TRANSPORTATION DIVISION
 1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5989

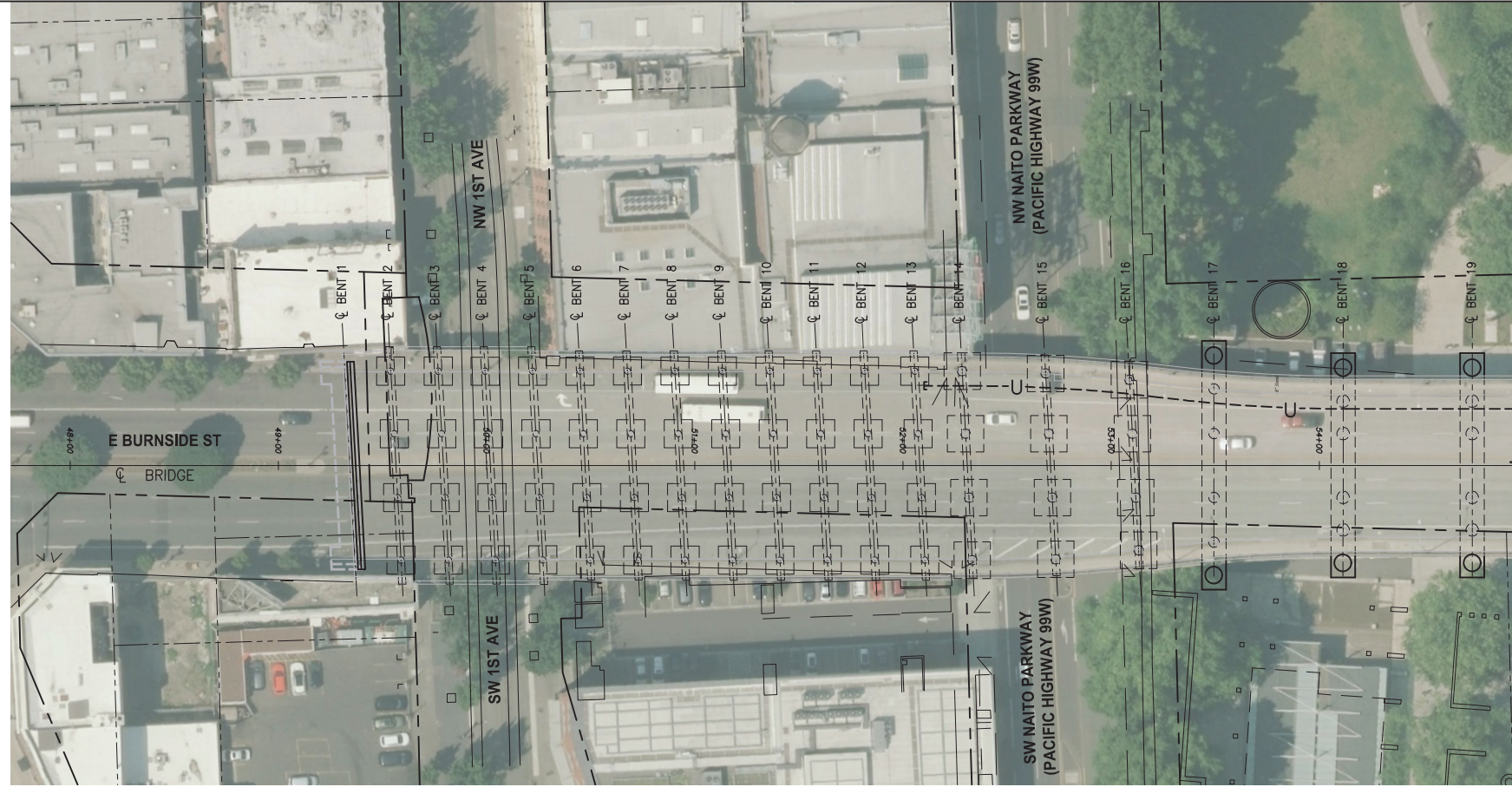
IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BJS/CRK
 DRAFTED BY: CRK/BRP
 CHECKED BY: YY

REVISIONS	
NO	DATE

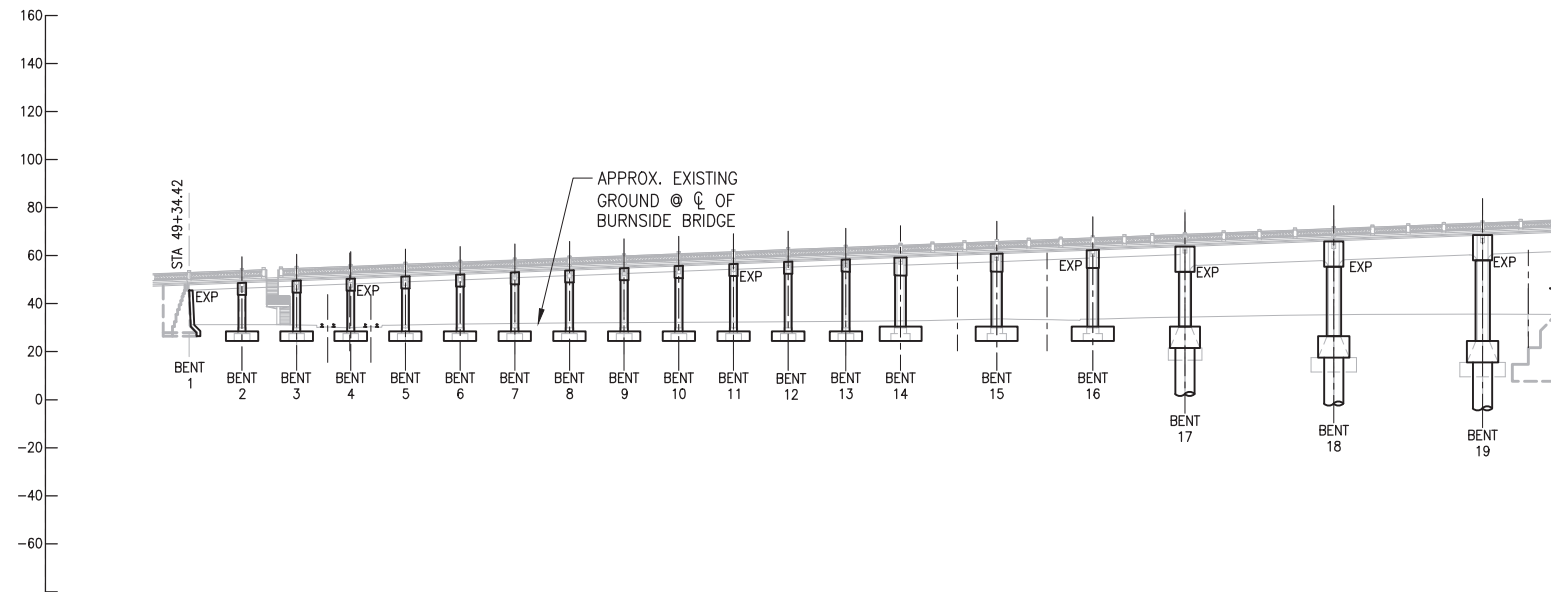
Sheet No. **S03**

DATE: 1/31/20 PROJECT NO.: Proj/No



PLAN

SCALE: 1" = 40'



ELEVATION

SCALE: 1" = 40'

LEGEND:

- EXISTING
- RETROFIT
- - - ROW

NOTES:

1. SUPERSTRUCTURE RETROFITS NOT SHOWN FOR CLARITY
2. GROUND IMPROVEMENT NOT SHOWN, SEE GEOTECHNICAL REPORT



CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
SUBSTRUCTURE RETROFIT LAYOUT
WEST APPROACH

DATE: 1/31/20 PROJECT NO.: Proj/No

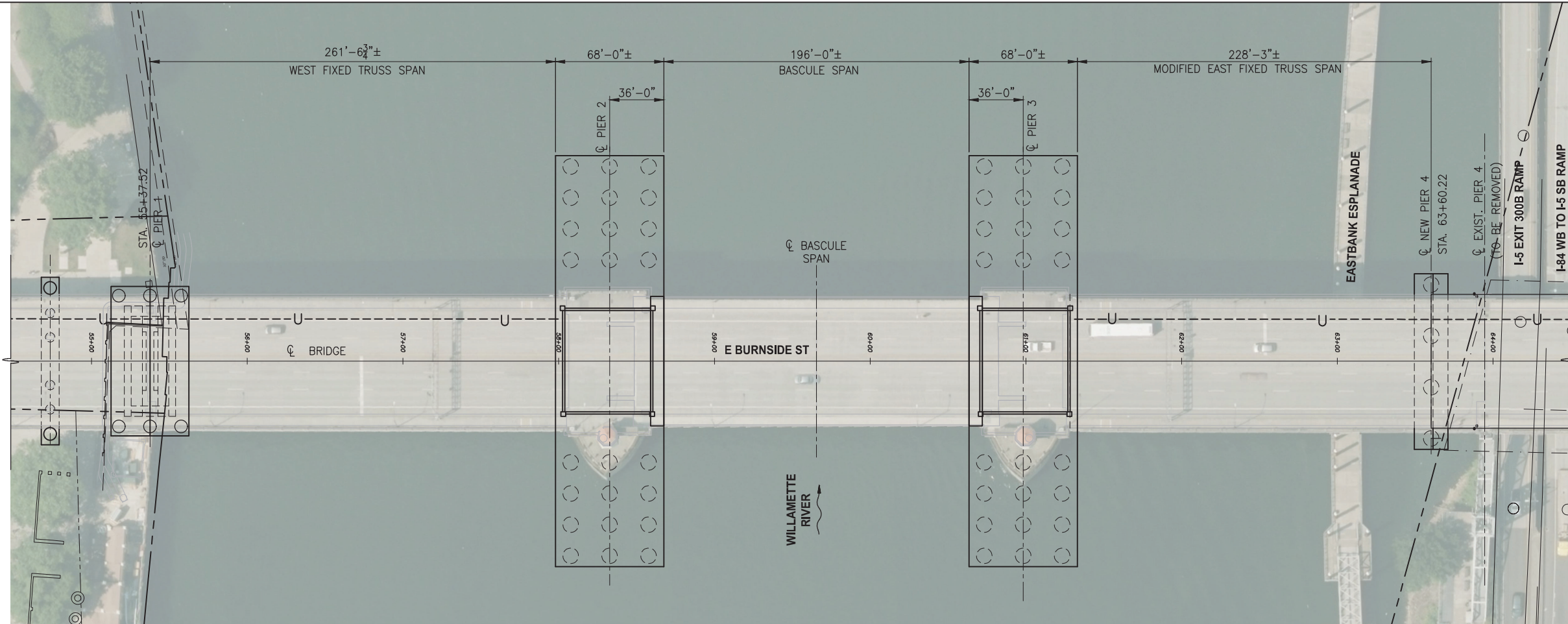
MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999

IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BJS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: YY

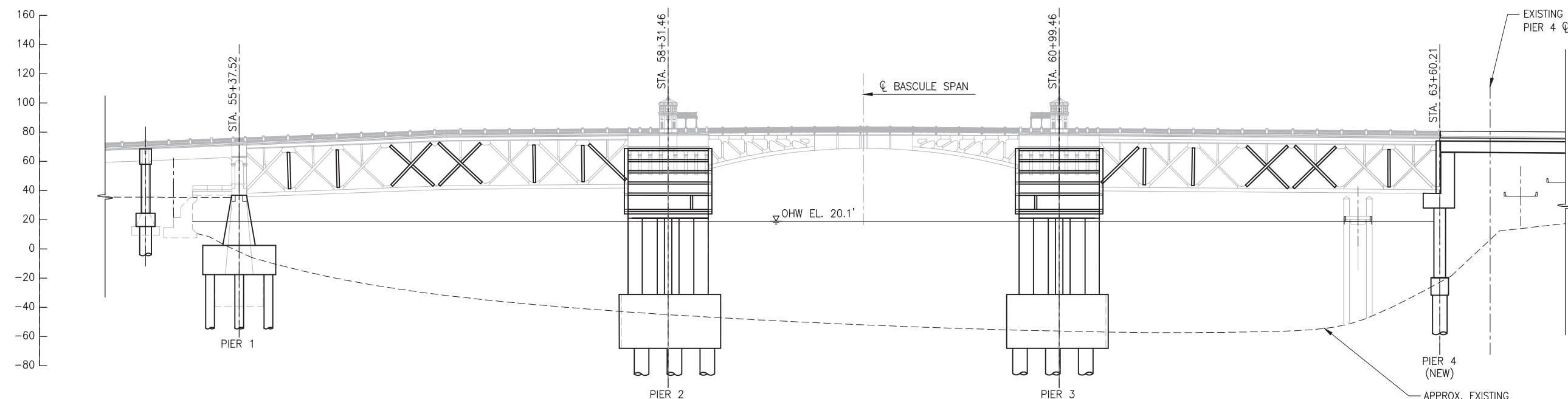
REVISIONS	
NO.	DATE:

Sheet No. **S04**



PLAN

SCALE: 1" = 40'



ELEVATION

SCALE: 1" = 40'

LEGEND:

- EXISTING
- RETROFIT
- - - ROW

NOTES:

1. BASCULE LEAVES RETROFIT NOT SHOWN FOR CLARITY
2. GROUND IMPROVEMENT NOT SHOWN, SEE GEOTECHNICAL REPORT

CONCEPTUAL PLANS
JUNE 2020



EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
SUBSTRUCTURE RETROFIT LAYOUT
 RIVER SPANS

MULTNOMAH COUNTY
 DEPARTMENT OF COMMUNITY SERVICES
 TRANSPORTATION DIVISION
 1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5989

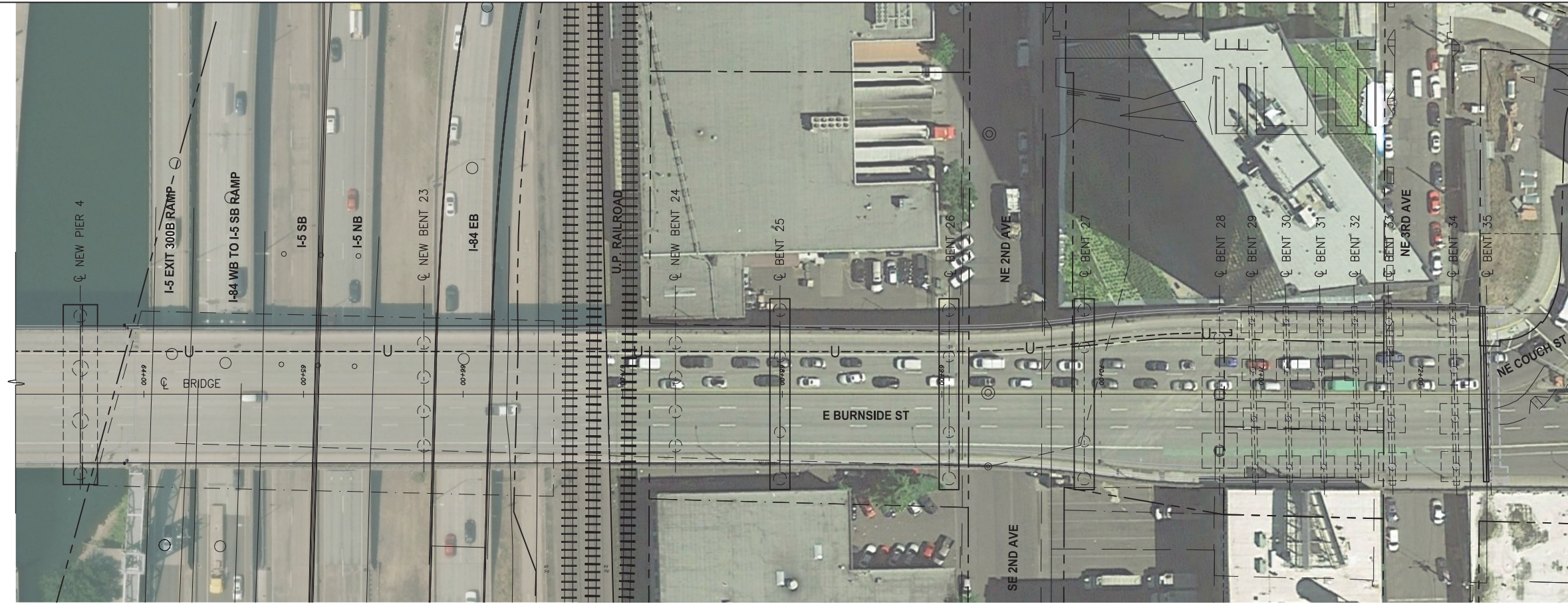
IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BJS/CRK
 DRAFTED BY: CRK/BRP
 CHECKED BY: YY

NO.	DATE:	REVISIONS

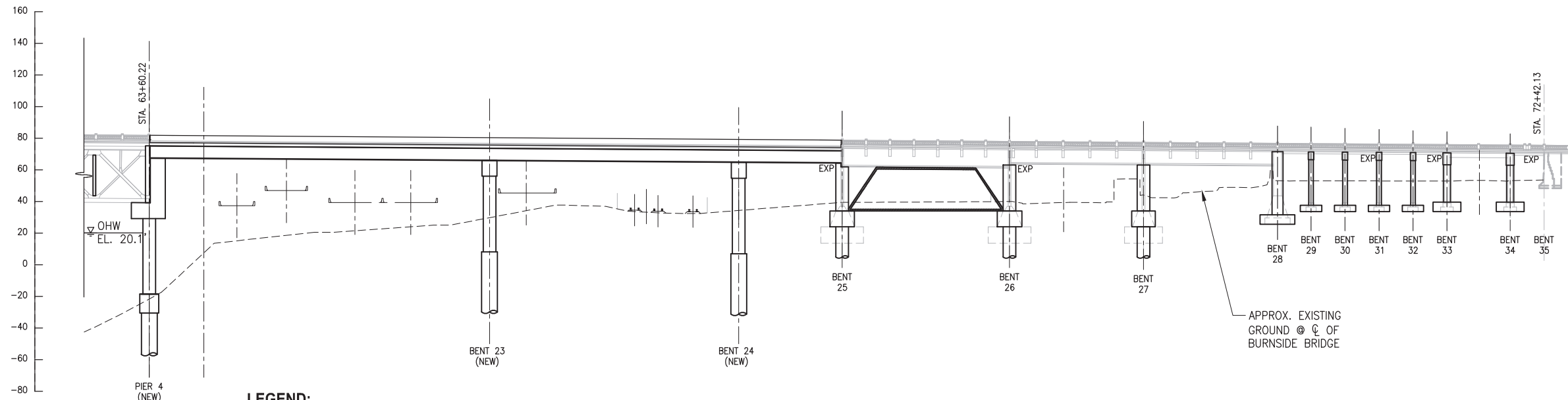
Sheet No. **S05**

DATE: 1/31/20 PROJECT NO.: Proj/No



PLAN

SCALE: 1" = 40'



ELEVATION

SCALE: 1" = 40'

NOTES:

1. BASCULE LEAVES RETROFIT NOT SHOWN FOR CLARITY
2. GROUND IMPROVEMENT NOT SHOWN, SEE GEOTECHNICAL REPORT

LEGEND:

- EXISTING
- RETROFIT
- - - ROW

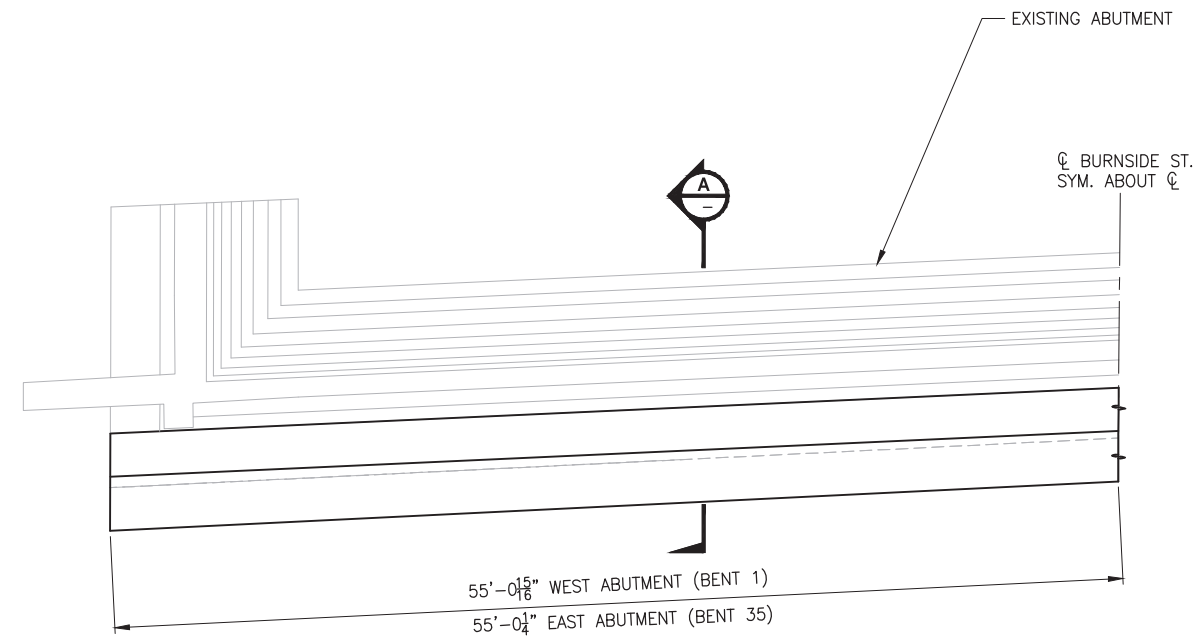
EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
SUBSTRUCTURE RETROFIT LAYOUT
EAST APPROACH

MULTNOMAH COUNTY
 DEPARTMENT OF COMMUNITY SERVICES
 TRANSPORTATION DIVISION
 1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5989

IAN B. CANNON P.E. COUNTY ENGINEER

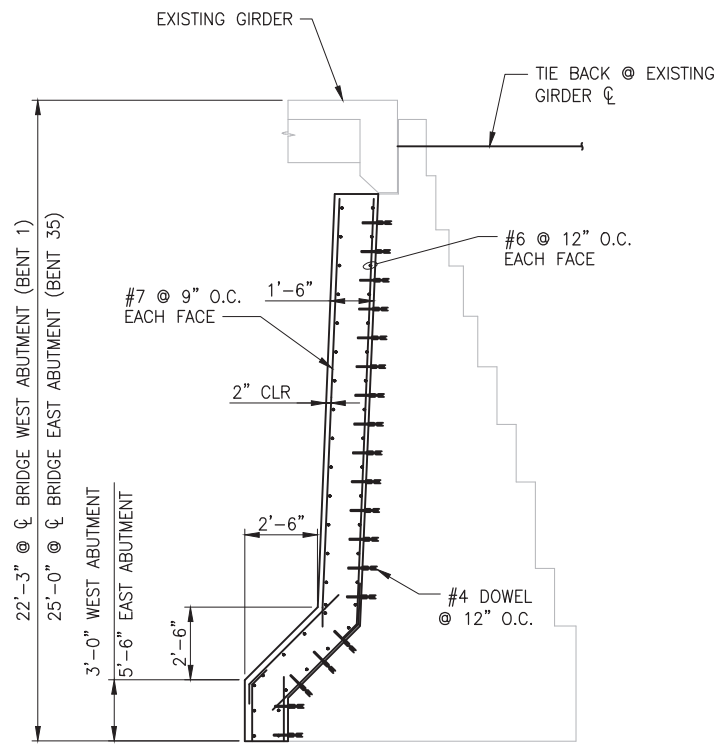
DESIGNED BY: *BJS/CRK*
 DRAFTED BY: *CRK/BRP*
 CHECKED BY: *YY*

REVISIONS	
NO.	DATE:



PLAN
SCALE: 1"=3'

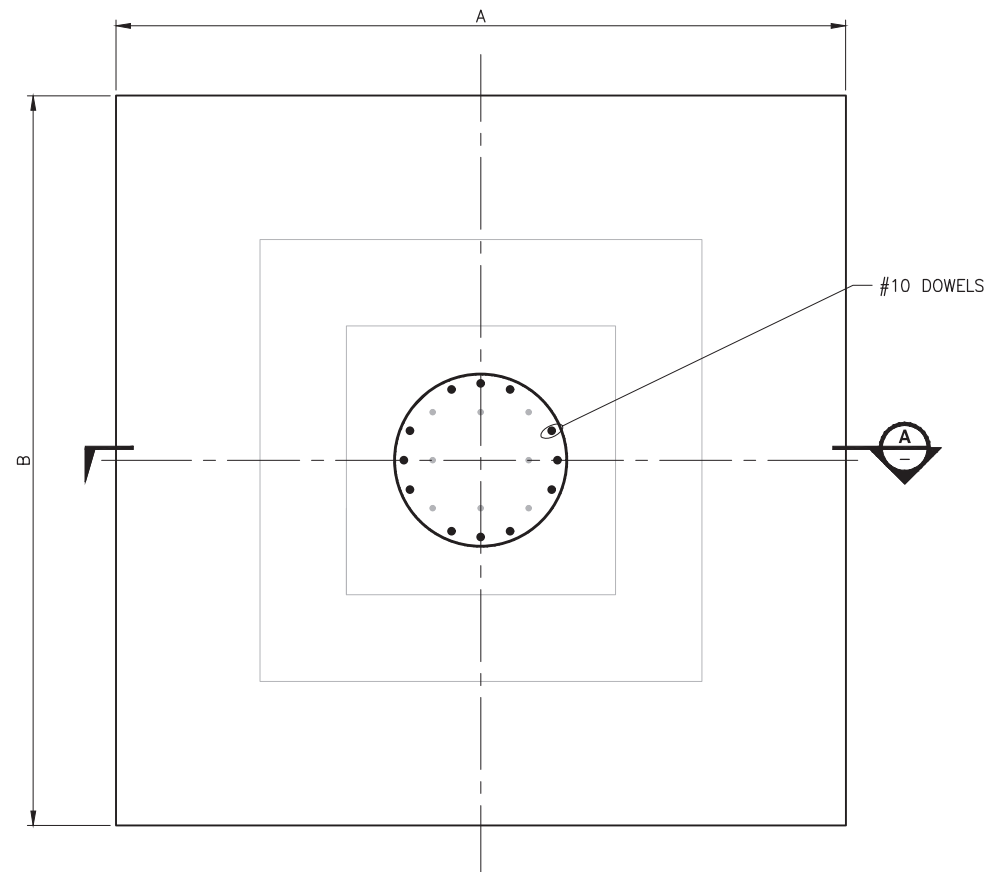
LEGEND:
 _____ EXISTING
 _____ RETROFIT



SECTION
SCALE: 1"=3'

CONCEPTUAL PLANS
JUNE 2020

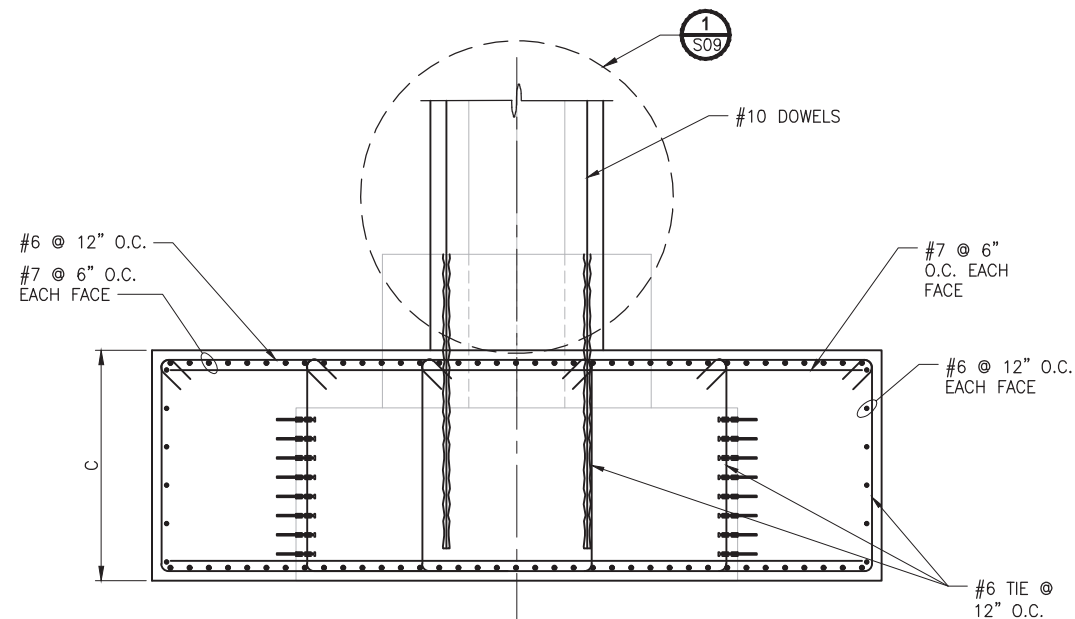
MULTNOMAH COUNTY DEPARTMENT OF COMMUNITY SERVICES TRANSPORTATION DIVISION 1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999		COUNTY ENGINEER IAN B. CANNON P.E.
		COUNTY ENGINEER
DESIGNED BY: BJS/CRK DRAFTED BY: CRK/BRP CHECKED BY: YY	REVISIONS NO. DATE:	EARTHQUAKE READY BURNSIDE BRIDGE ENHANCED SEISMIC RETROFIT ABUTMENT RETROFIT BENT 1 AND 35
Sheet No. S07		DATE: 1/31/20 PROJECT NO.: Proj No



PLAN

NOT TO SCALE

FOUNDATION RETROFIT DIMENSIONS			
BENT NUMBER	"A"	"B"	"C"
2	13'-6"	13'-6"	4'-0"
3	13'-6"	13'-6"	4'-0"
4	13'-6"	13'-6"	4'-0"
5	13'-6"	13'-6"	4'-0"
6	13'-6"	13'-6"	4'-0"
7	13'-6"	13'-6"	4'-0"
8	13'-6"	13'-6"	4'-0"
9	13'-6"	13'-6"	4'-0"
10	13'-6"	13'-6"	4'-0"
11	13'-6"	13'-6"	4'-0"
12	13'-6"	13'-6"	4'-0"
13	13'-6"	13'-6"	4'-0"
14	17'-6"	17'-6"	6'-0"
15	17'-6"	17'-6"	6'-0"
16	17'-6"	17'-6"	6'-0"
28	22'-0"	22'-0"	6'-0"
29	13'-6"	13'-6"	4'-0"
30	13'-6"	13'-6"	4'-0"
31	13'-6"	13'-6"	4'-0"
32	13'-6"	13'-6"	4'-0"
33	17'-6"	17'-6"	6'-0"
34	19'-0"	19'-0"	6'-0"



SECTION

NOT TO SCALE

LEGEND:

- EXISTING
- RETROFIT

CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE

ENHANCED SEISMIC RETROFIT
FOOTING RETROFIT
BENT 2-16 & 28-34

DATE: 1/31/20 PROJECT NO.: Proj No

MULTNOMAH COUNTY

DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999



IAN B. CANNON P.E.

COUNTY ENGINEER

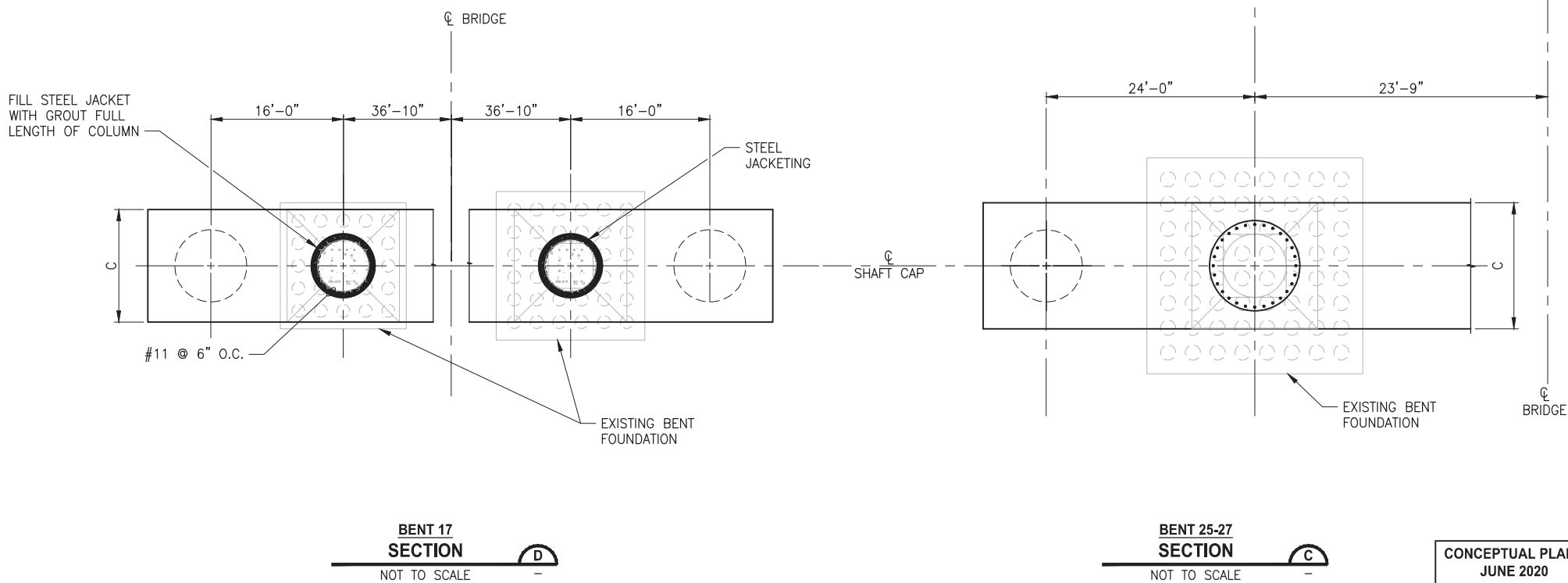
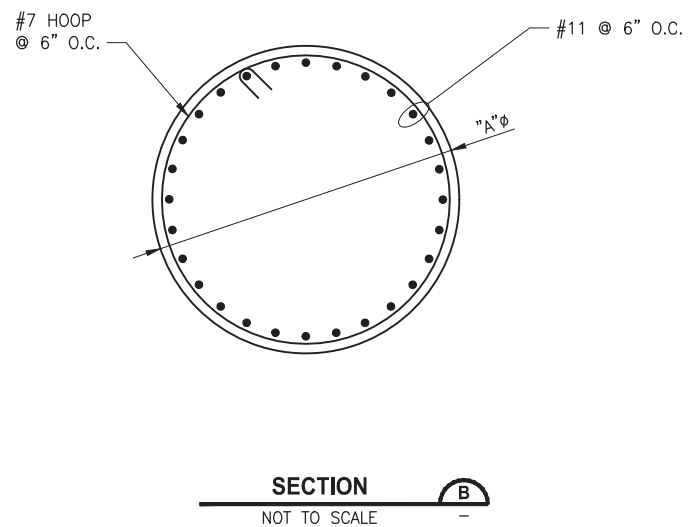
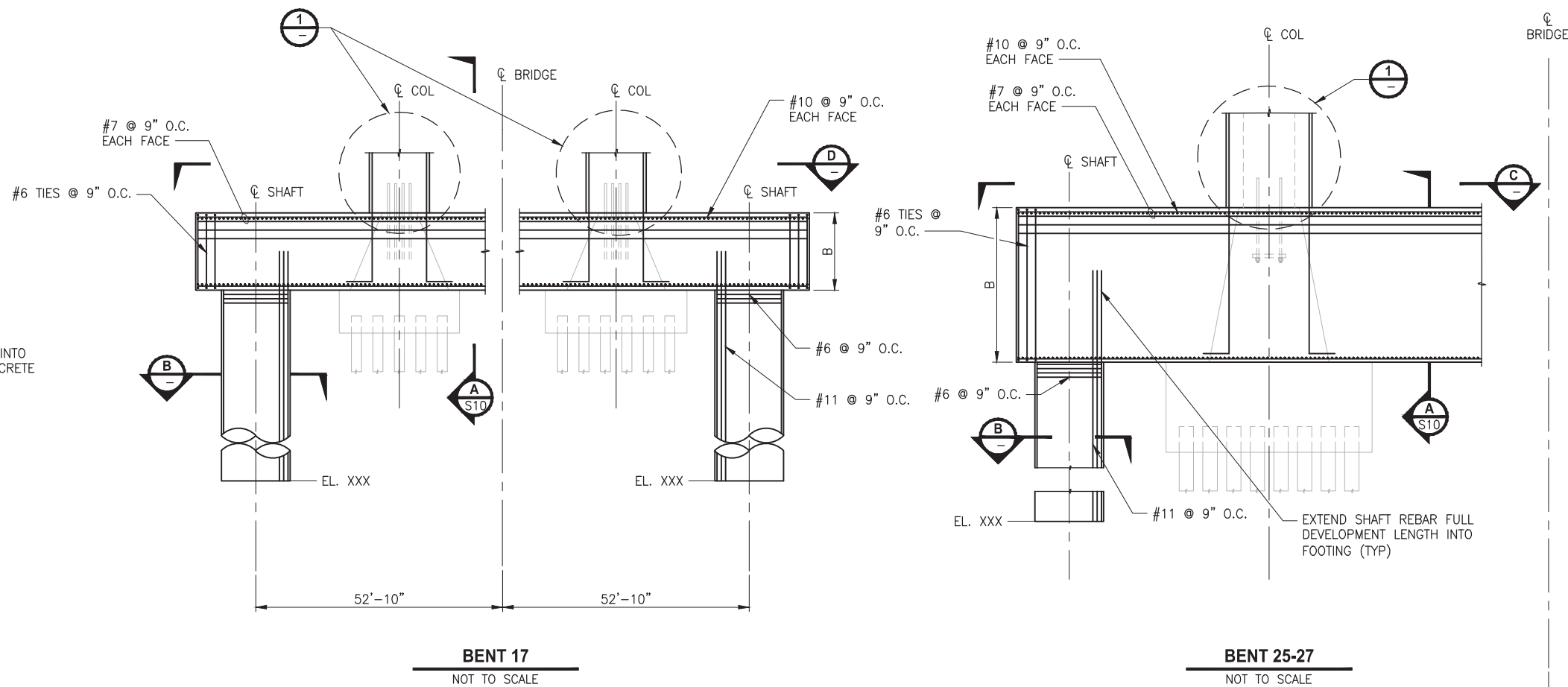
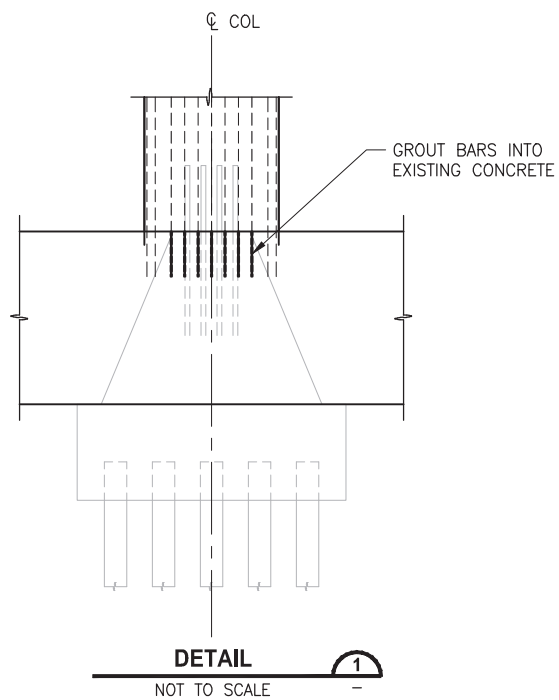
DESIGNED BY: BJS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: YY

REVISIONS

NO	DATE

Sheet No. S08

FOUNDATION RETROFIT DIMENSIONS			
BENT NUMBER	"A"	"B"	"C"
17	8'-0"	9'-0"	12'-6"
25	8'-0"	10'-0"	15'-6"
26	8'-0"	10'-0"	15'-6"
27	8'-0"	10'-0"	14'-0"



CONCEPTUAL PLANS
JUNE 2020

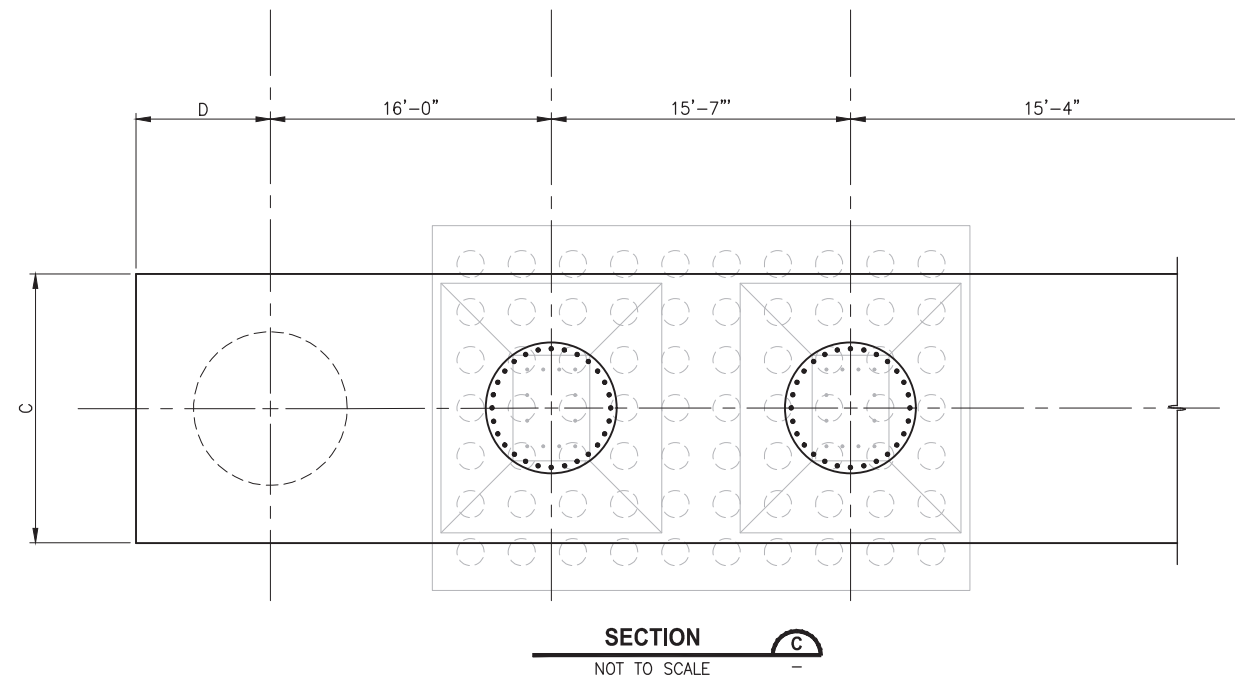
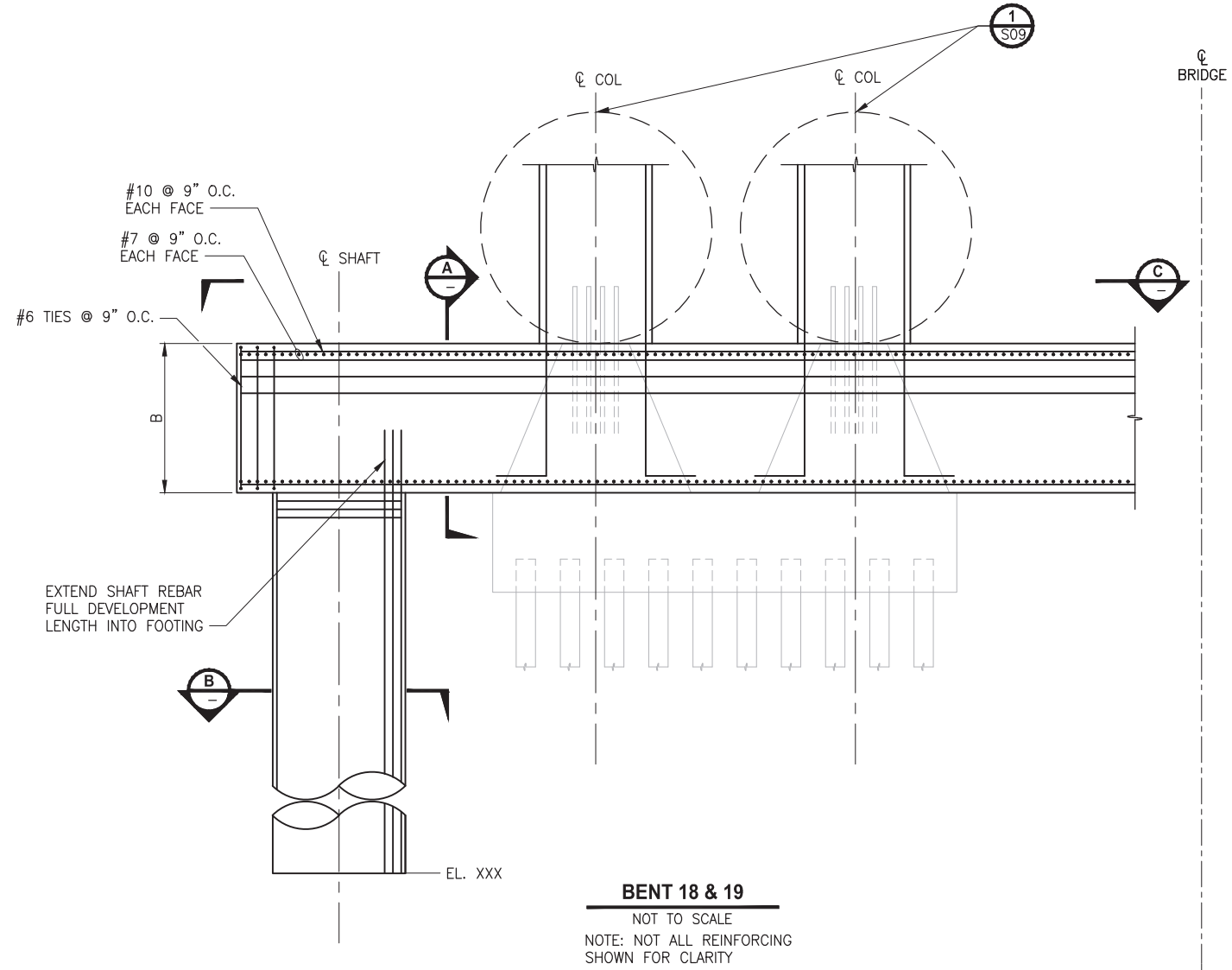
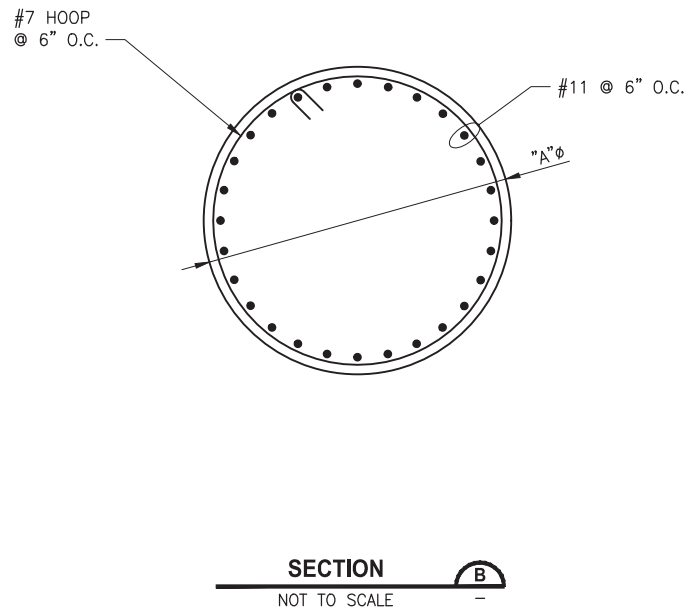
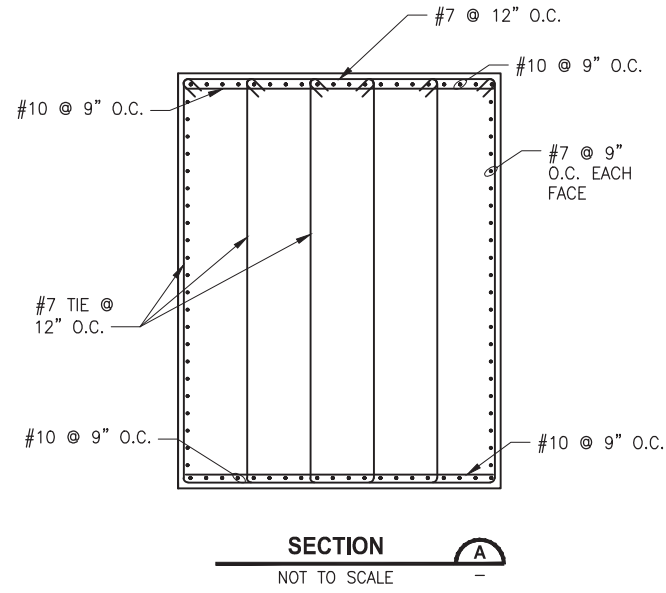
EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
FOOTING RETROFIT
BENT 17, 25-27
DATE: 1/31/20 PROJECT NO.: Proj No

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999
IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BIS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: YY

NO.	DATE	REVISIONS

FOUNDATION RETROFIT DIMENSIONS				
BENT NUMBER	"A"	"B"	"C"	"D"
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19	8'-0"	9'-0"	13'-0"	7'-0"



CONCEPTUAL PLANS
JUNE 2020

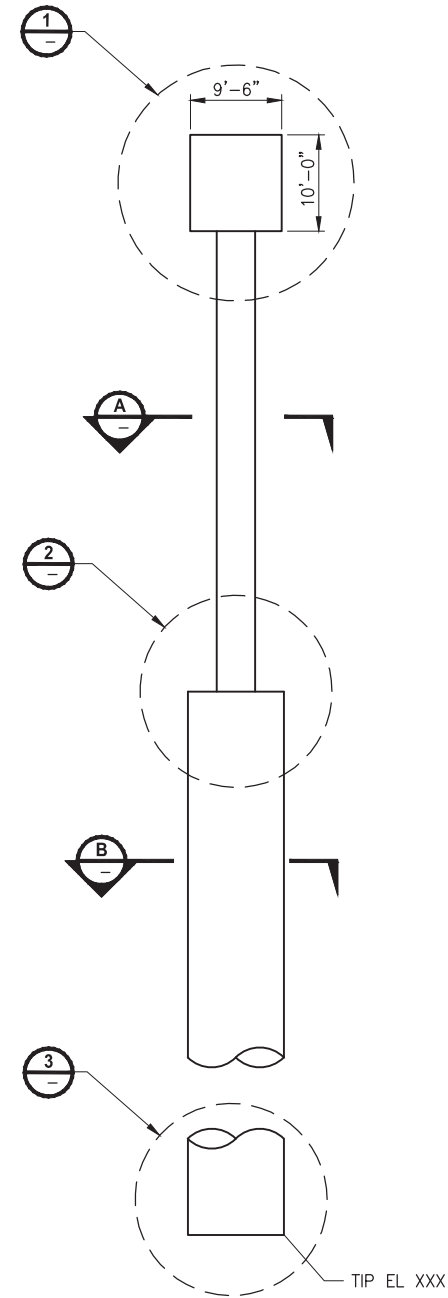
EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
FOOTING RETROFIT
BENT 18 & 19
DATE: 1/31/20 PROJECT NO.: Proj No

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97235-5999
IAN B. CANNON P.E. COUNTY ENGINEER

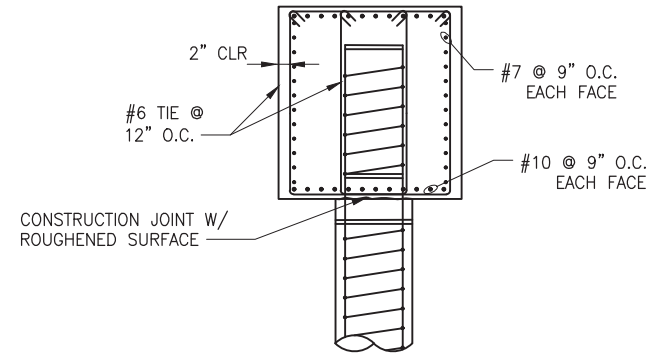
DESIGNED BY: BJS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: Y Y

NO.	DATE	REVISIONS

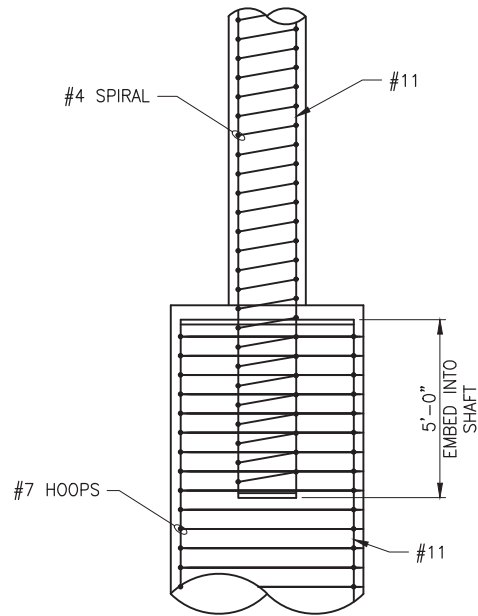
Sheet No. 510



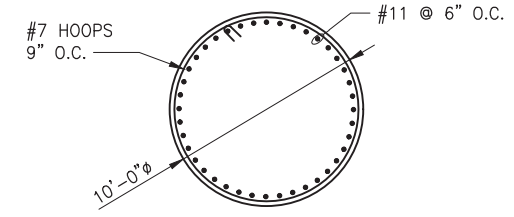
ELEVATION
SCALE: 1"=10'



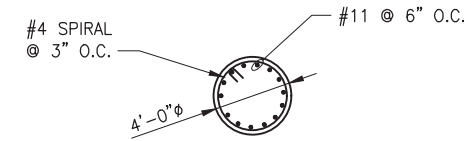
DETAIL 1
SCALE: 1"=5'



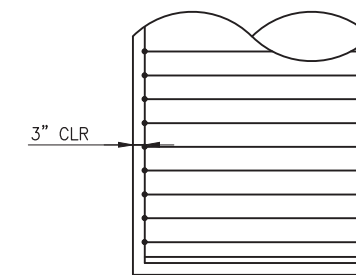
DETAIL 2
SCALE: 1"=5'



SECTION B
SCALE: 1"=5'



SECTION A
SCALE: 1"=5'



DETAIL 3
SCALE: 1"=5'

CONCEPTUAL PLANS
JUNE 2020

DESIGNED BY:
BJS/CRK
DRAFTED BY:
CRK/BRP
CHECKED BY:
YY

REVISIONS

NO.	DATE:		

Sheet No.
S11

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999



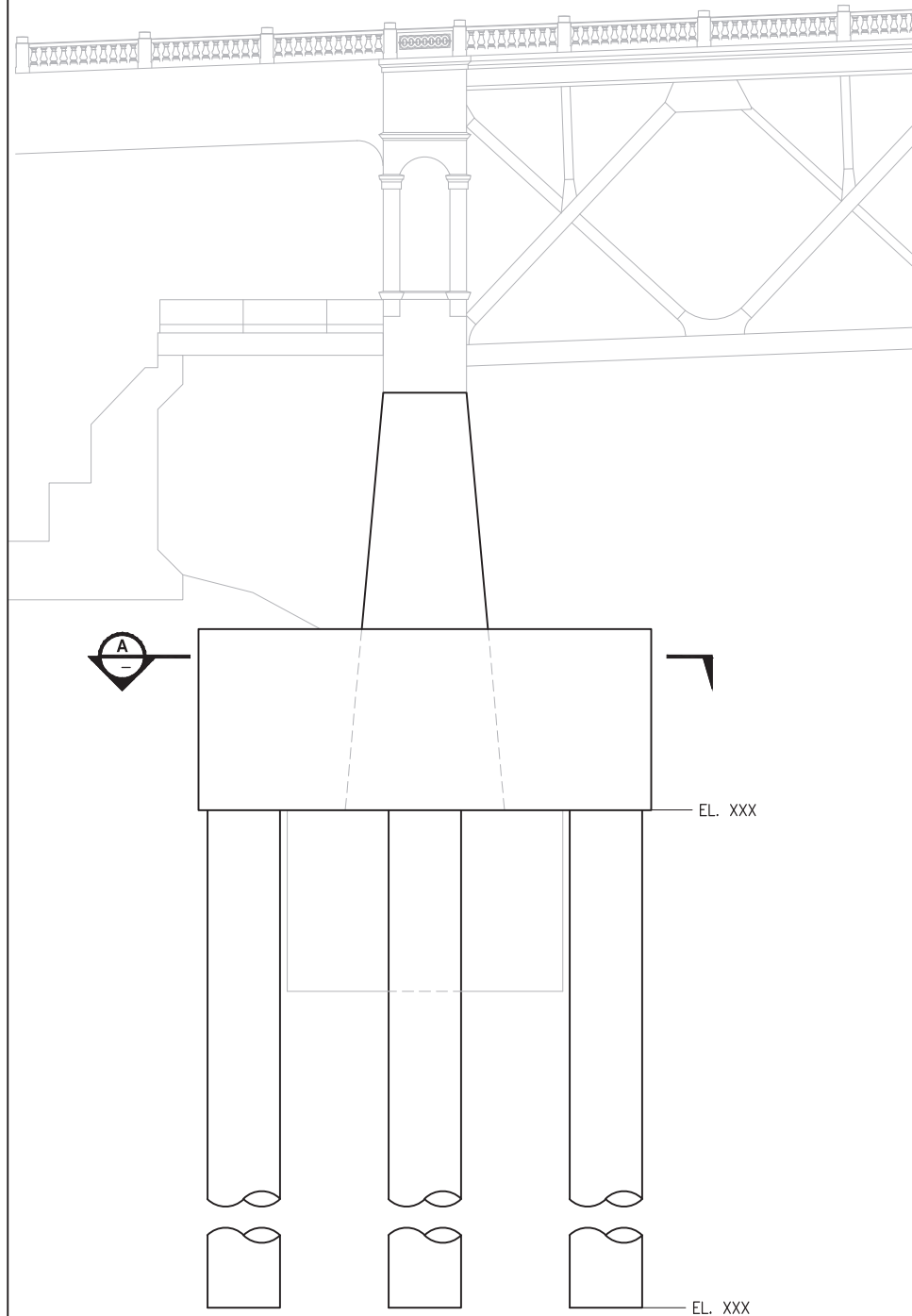
IAN B. CANNON P.E.

COUNTY ENGINEER

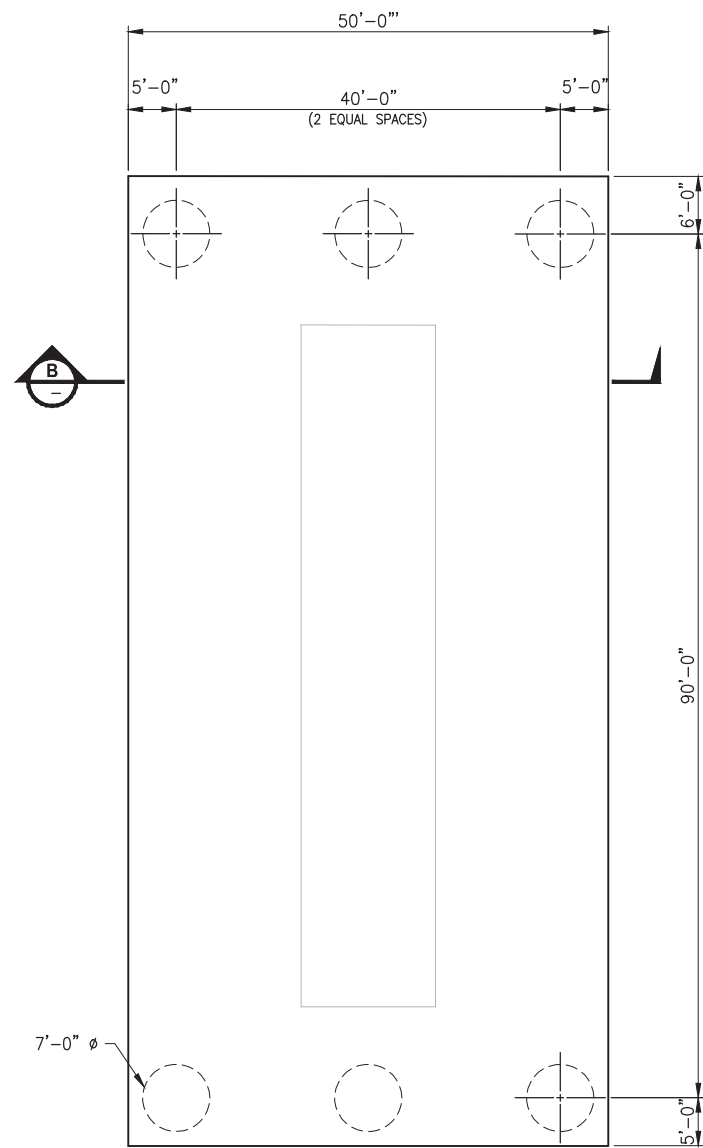
EARTHQUAKE READY BURNSIDE BRIDGE

ENHANCED SEISMIC RETROFIT
BENT AND FOOTING REPLACEMENT
NEW BENTS 23-24

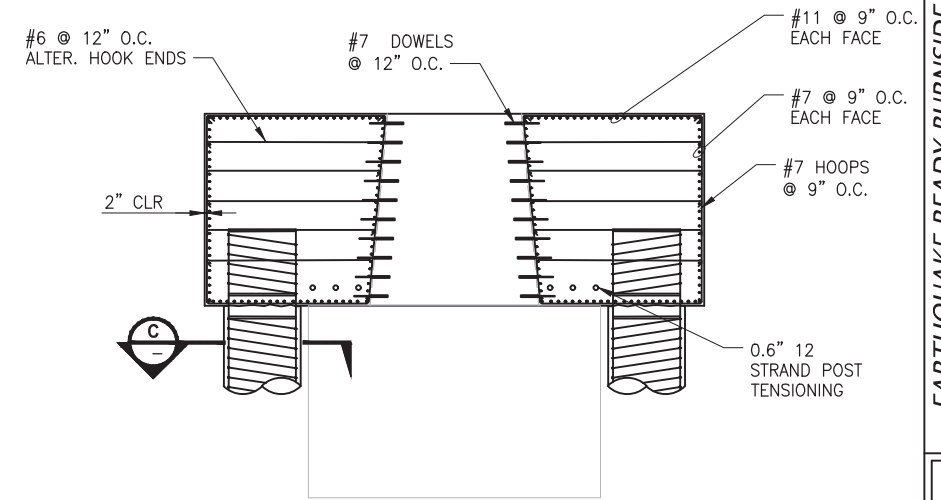
DATE: 1/31/20 PROJECT NO.: Proj No



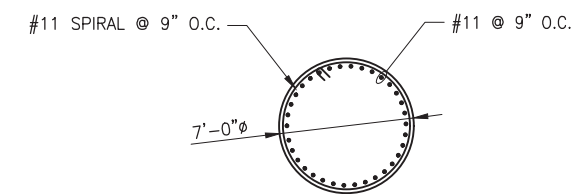
PIER 1 RETROFIT
SCALE: 1"=10'



SECTION A
SCALE: 1"=10'



SECTION B
SCALE: 1"=10'



SECTION C
SCALE: 1"=5'

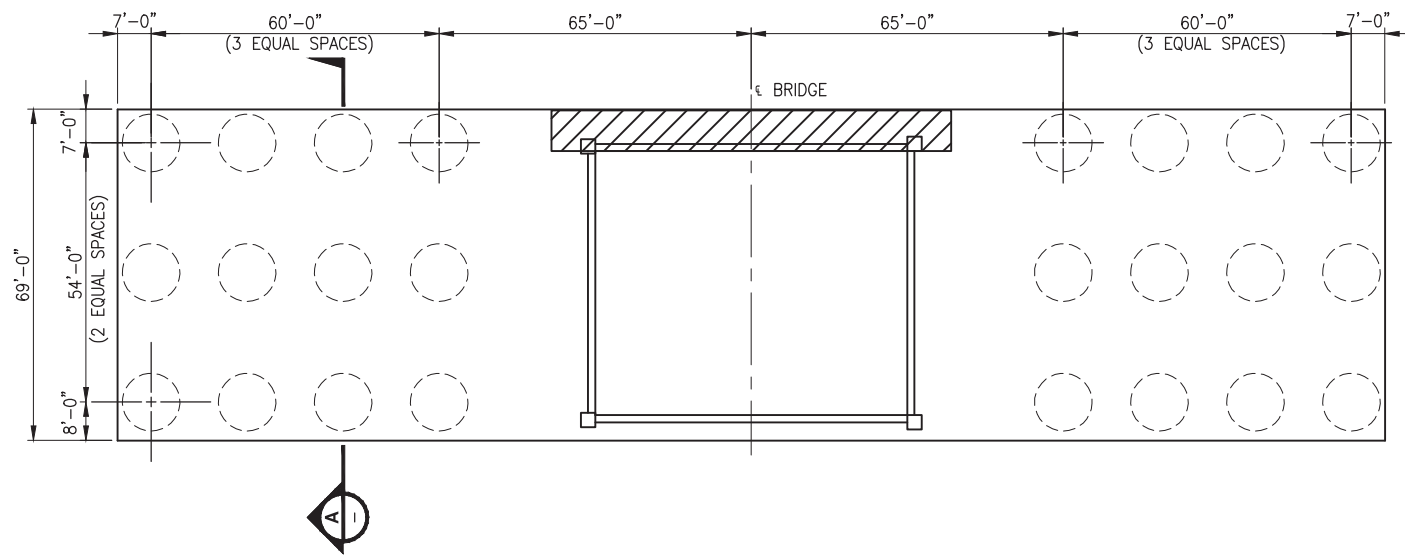
CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
FOUNDATION RETROFIT
PIER 1
DATE: 1/31/20 PROJECT NO.: Proj/No

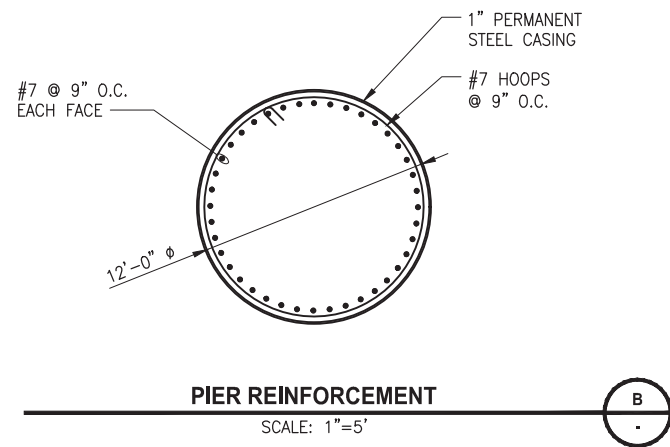
MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97235-5999
IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BJS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: YY

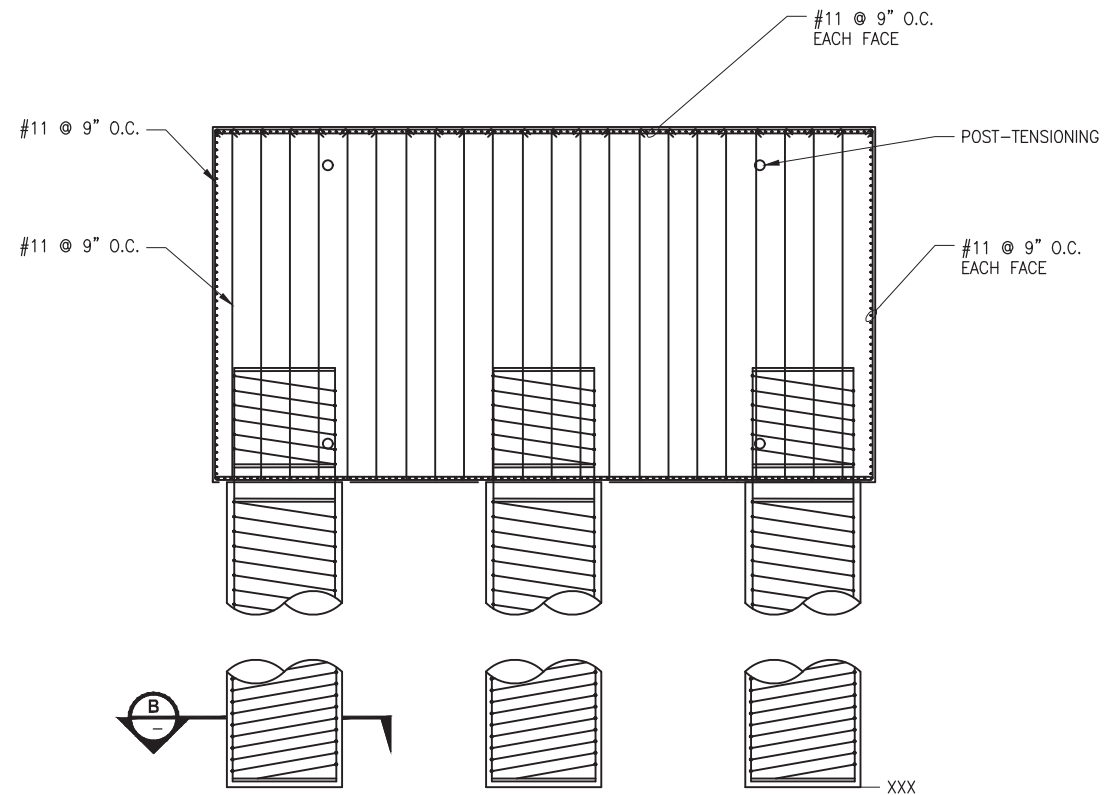
NO.	DATE:	REVISIONS



PIERS 2 & 3 FOUNDATION RETROFIT
SCALE: 1"=20'



PIER REINFORCEMENT
SCALE: 1"=5'



PIERS 2 AND 3 FOUNDATION RETROFIT
SCALE: 1"=10'

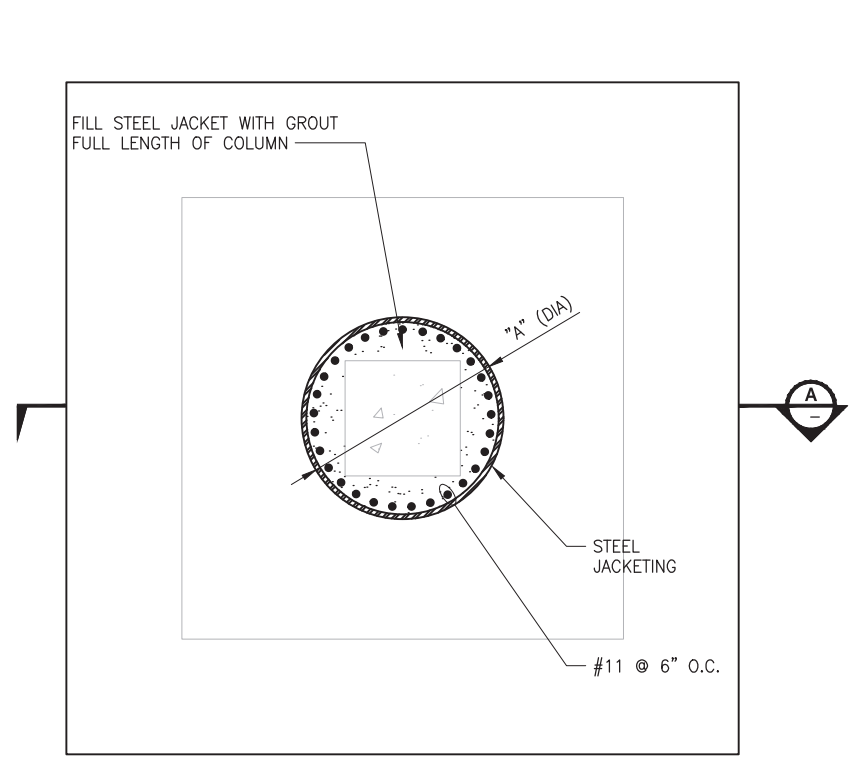
CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
FOUNDATION RETROFIT
PIER 2 & 3
DATE: 1/31/20 PROJECT NO.: Proj/No

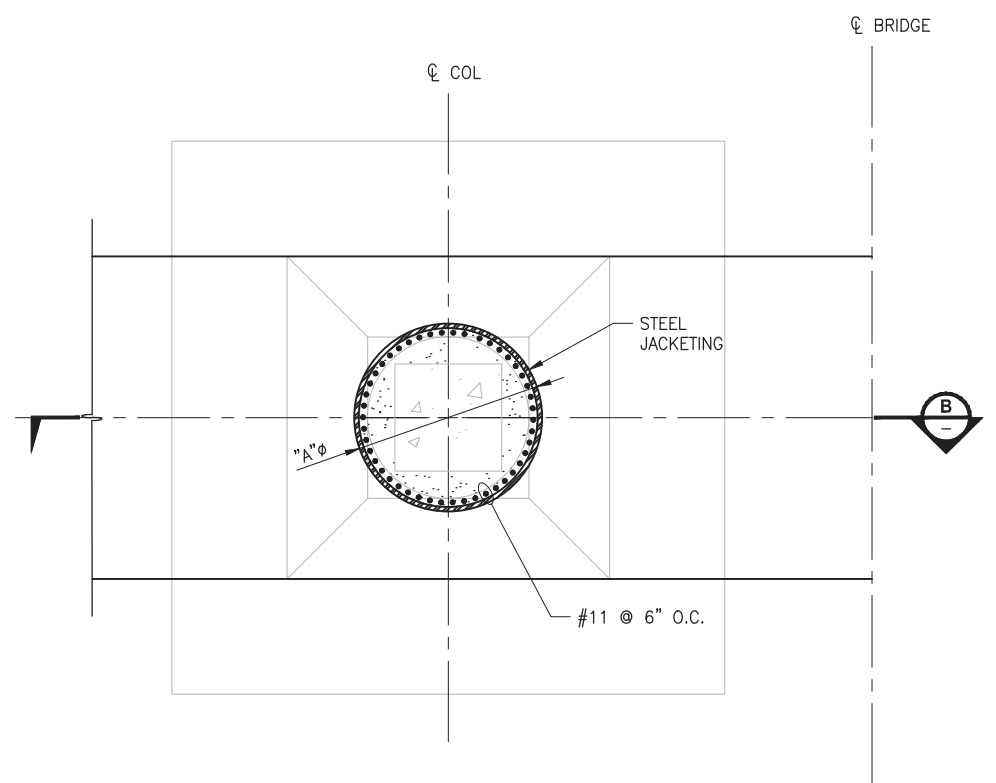
MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999
IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BJS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: YY

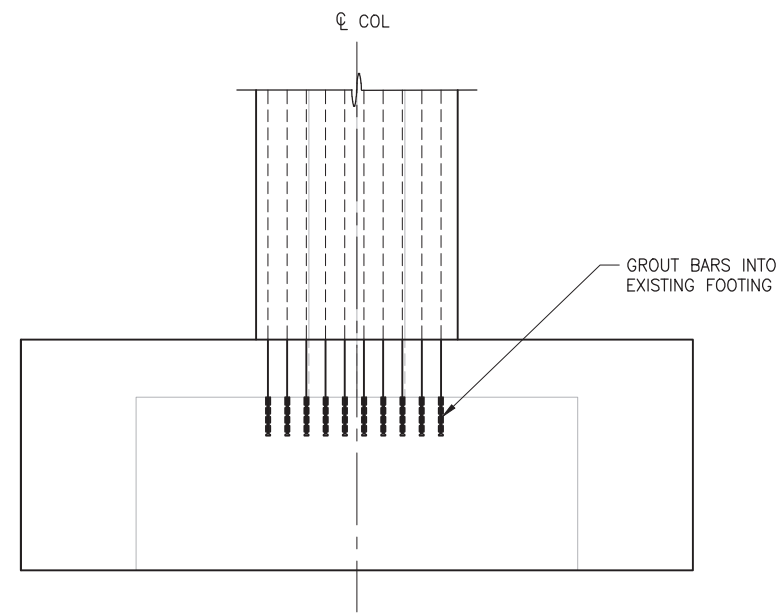
NO.	DATE:	REVISIONS



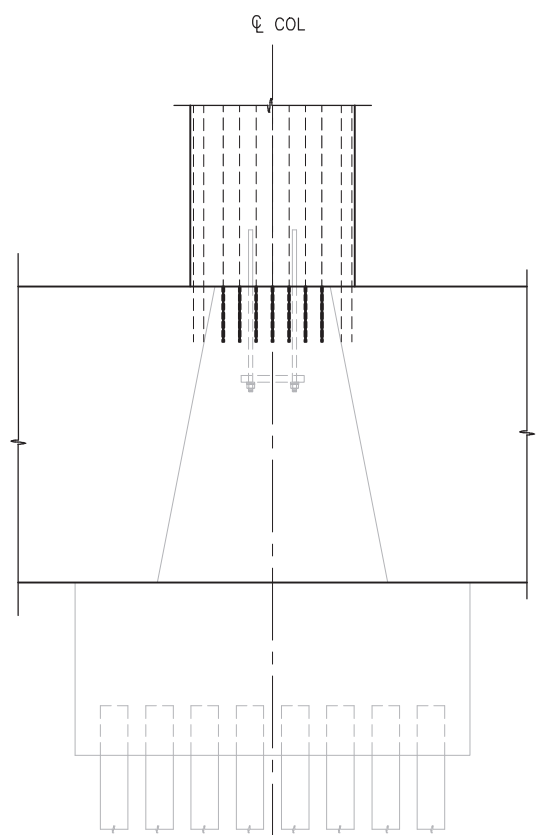
COLUMN RETROFIT BENT 2-16 & 28-34
DETAIL
 NOT TO SCALE



COLUMN RETROFIT BENT 17-19
PLAN
 NOT TO SCALE



SECTION
 NOT TO SCALE



SECTION
 NOT TO SCALE

COLUMN RETROFIT DATA	
BENT NUMBER	"A"
2	3'-0"
3	3'-0"
4	3'-0"
5	3'-0"
6	3'-0"
7	3'-0"
8	3'-0"
9	3'-0"
10	3'-0"
11	3'-0"
12	3'-0"
13	3'-0"
14	3'-9"
15	3'-9"
16	3'-9"
17	5'-9"
18	6'-3"
19	6'-3"
28	7'-0"
29	3'-0"
30	3'-0"
31	3'-0"
32	3'-0"
33	3'-9"
34	3'-9"

CONCEPTUAL PLANS
 JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE

**ENHANCED SEISMIC RETROFIT
 COLUMN RETROFIT
 BENT 2-19 & 28-34**

DATE: 1/31/20 PROJECT NO.: Proj No

MULTNOMAH COUNTY
 DEPARTMENT OF COMMUNITY SERVICES
 TRANSPORTATION DIVISION
 1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999

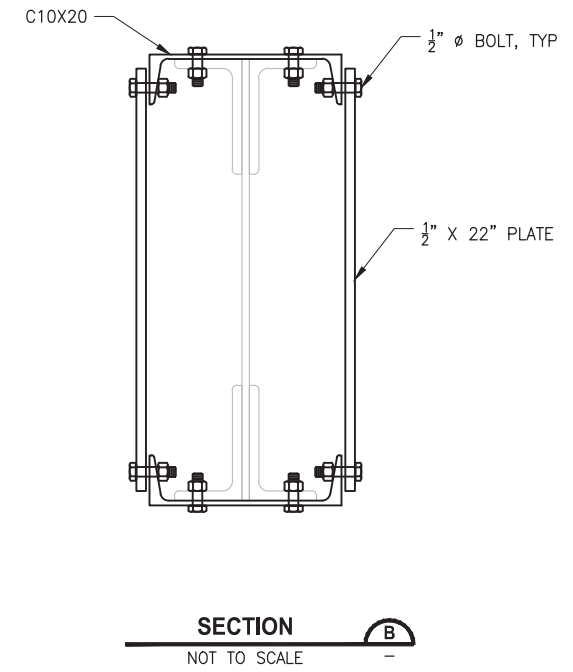
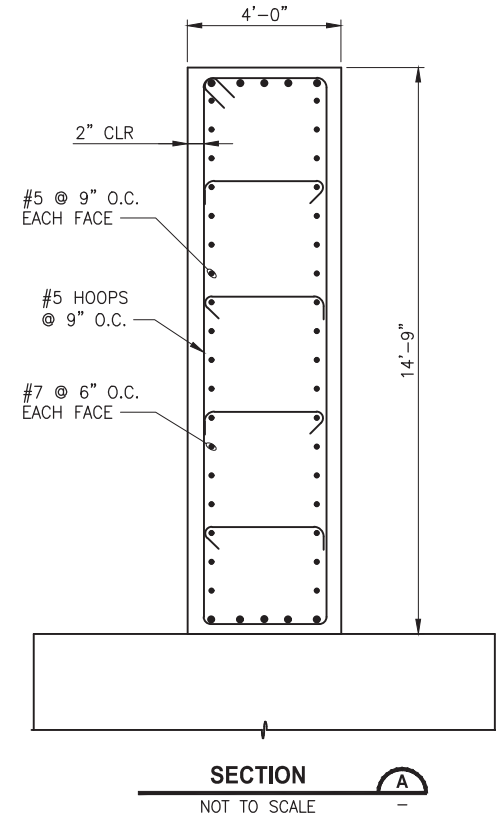
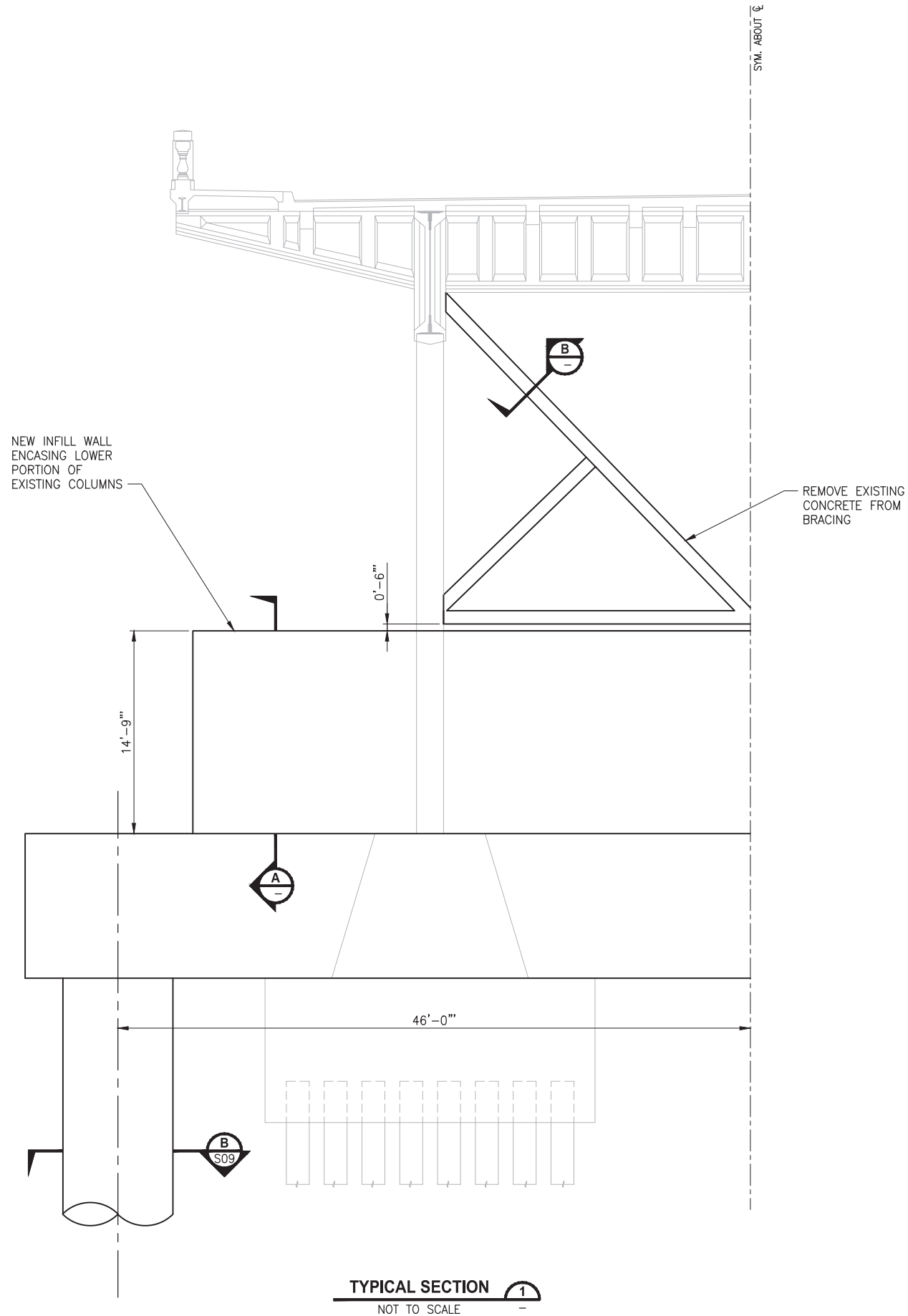
IAN B. CANNON P.E.
 COUNTY ENGINEER

DESIGNED BY: *BJS/CRK*
 DRAFTED BY: *CRK/BRP*
 CHECKED BY: *YY*

NO. DATE: _____

REVISIONS

Sheet No. **S14**



CONCEPTUAL PLANS
JUNE 2020

DESIGNED BY:
BJS/CRK

DRAFTED BY:
CRK/BRP

CHECKED BY:
YY

REVISIONS

NO. DATE:

Sheet No.
S15

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999



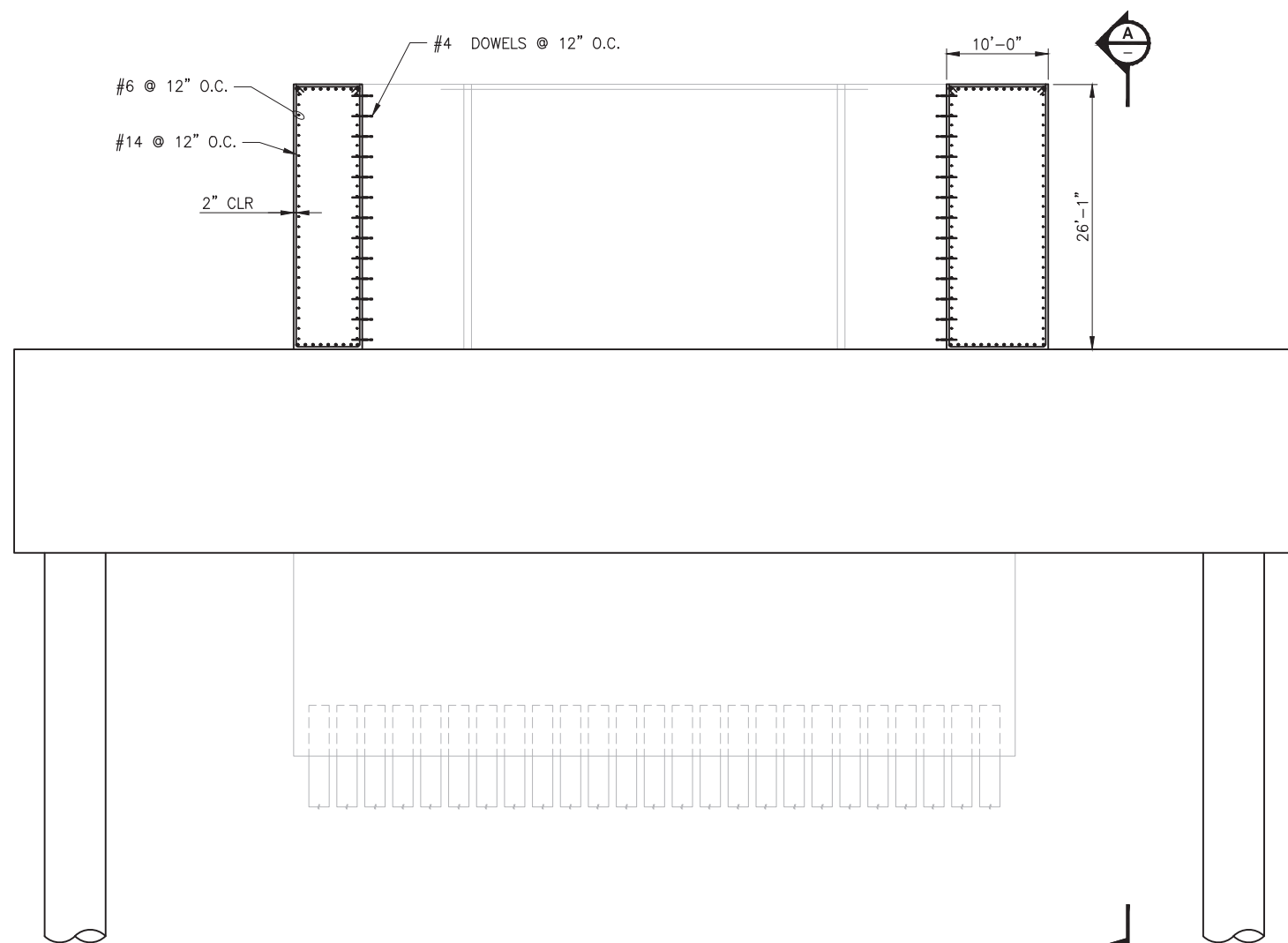
IAN B. CANNON P.E.

COUNTY ENGINEER

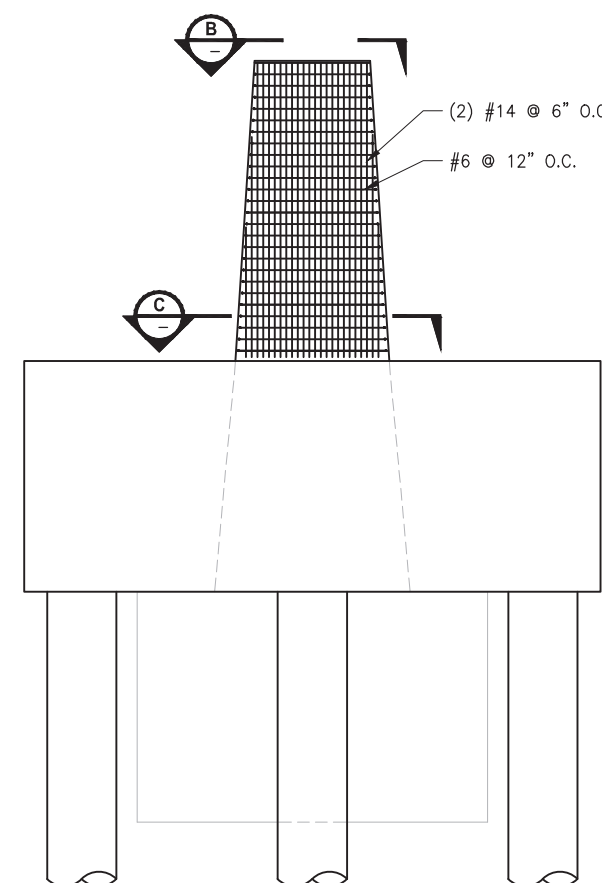
EARTHQUAKE READY BURNSIDE BRIDGE

ENHANCED SEISMIC RETROFIT
BENT RETROFIT
BENT 25-27

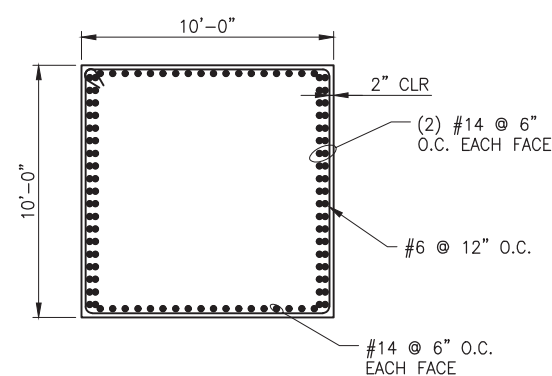
DATE: 1/31/20 PROJECT NO.: Proj No



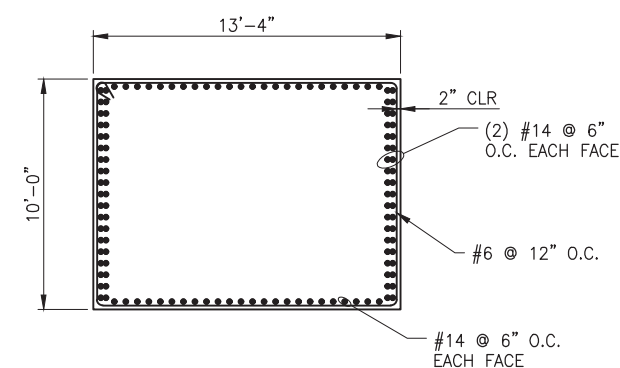
ELEVATION
SCALE: NTS



SECTION
SCALE: NTS



SECTION
SCALE: NTS



SECTION
SCALE: NTS

CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
PIER 1 RETROFIT
PIER 1

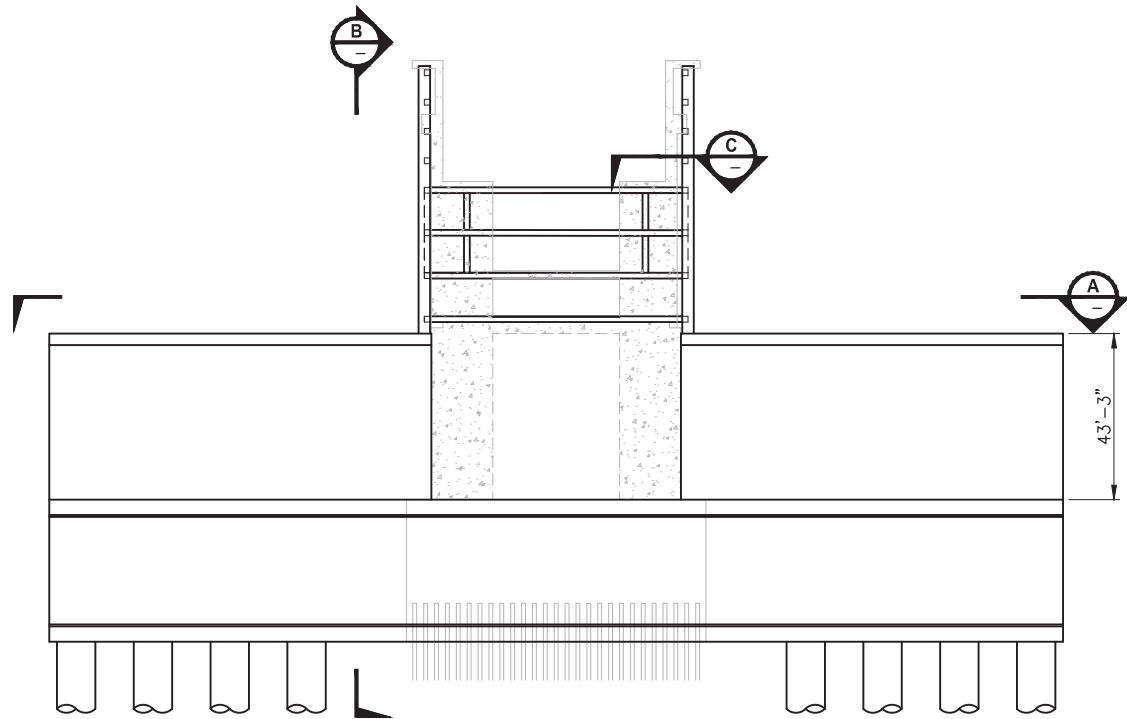
DATE: 1/31/20 PROJECT NO.: Proj/No

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999

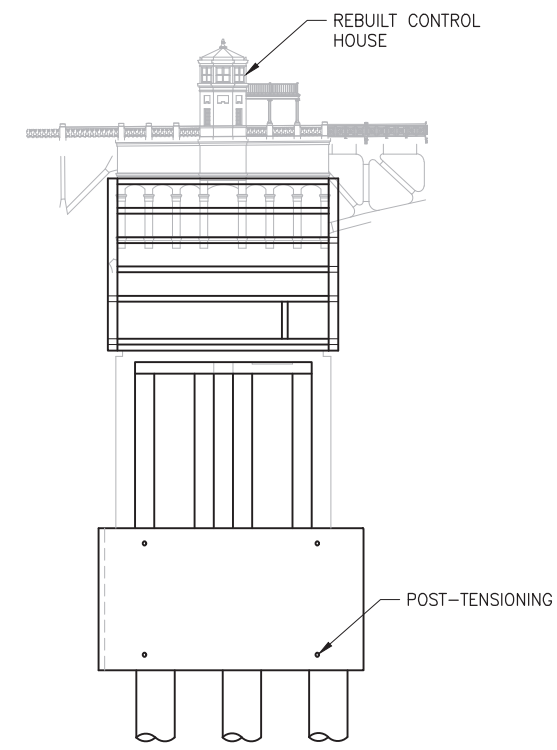
IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BJS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: YY

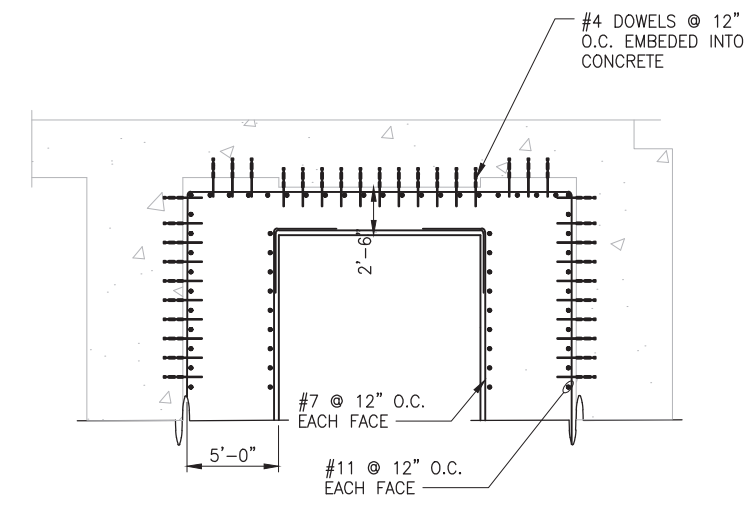
NO.	DATE:	REVISIONS



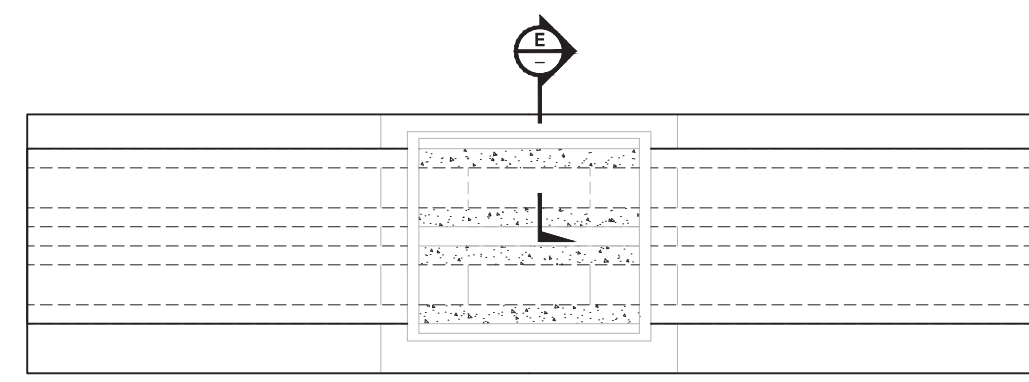
ELEVATION
SCALE: 1"=25'



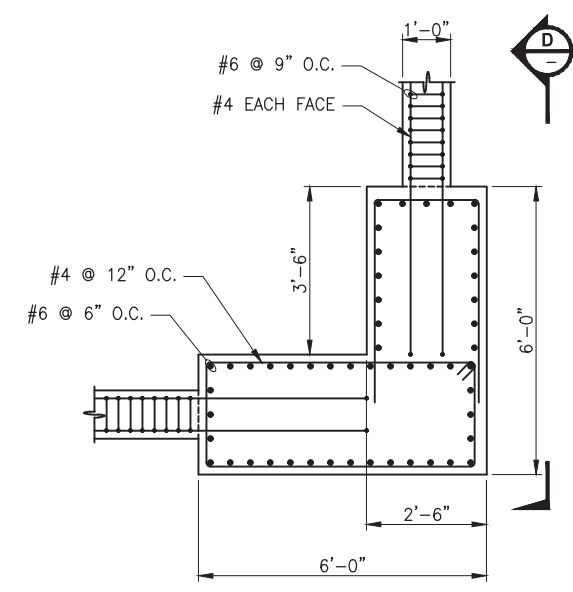
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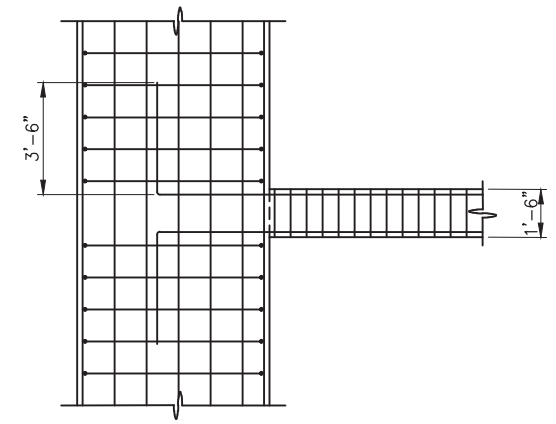
SECTION
SCALE: 1"=5'



SECTION
SCALE: 1"=30'



SECTION
SCALE: 1"=2'



SECTION
SCALE: 1"=3'

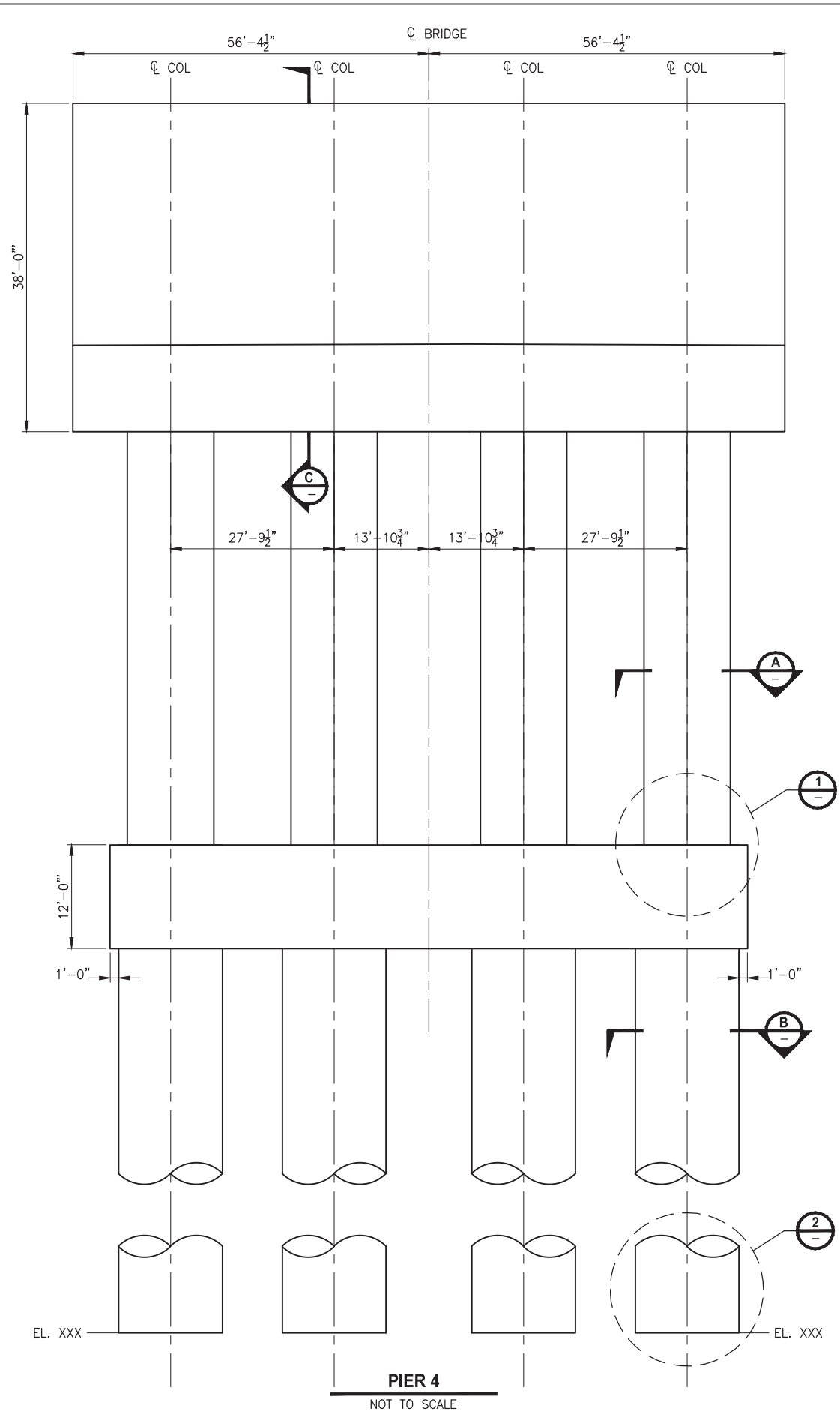
CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
BASCULE PIER RETROFIT
PIER 2 & 3
DATE: 1/31/20 PROJECT NO.: Proj/No

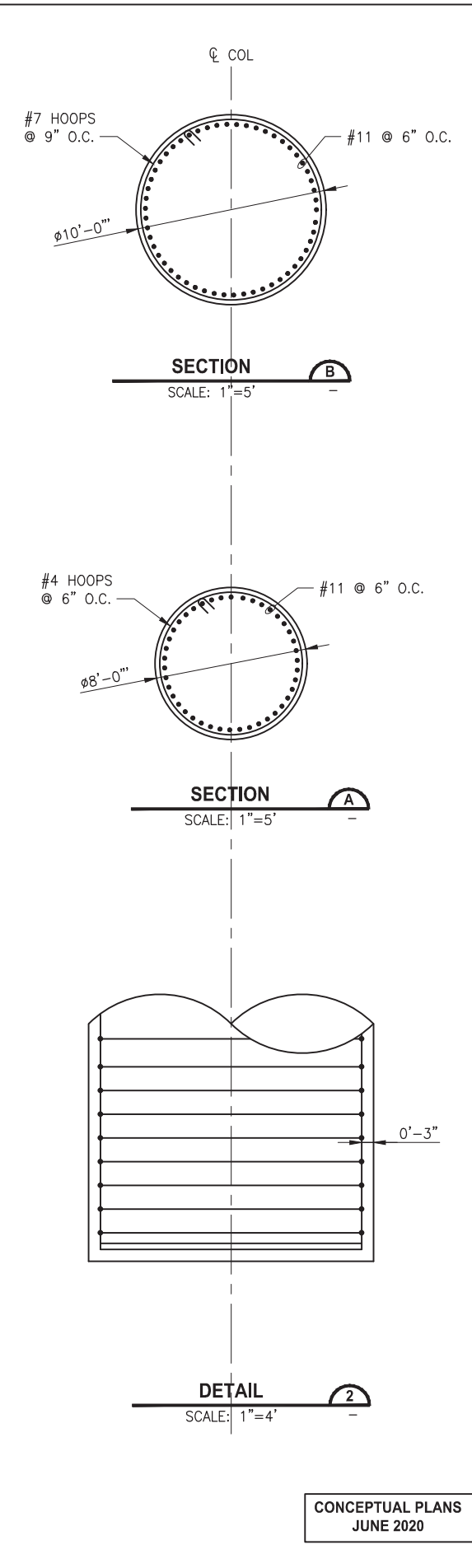
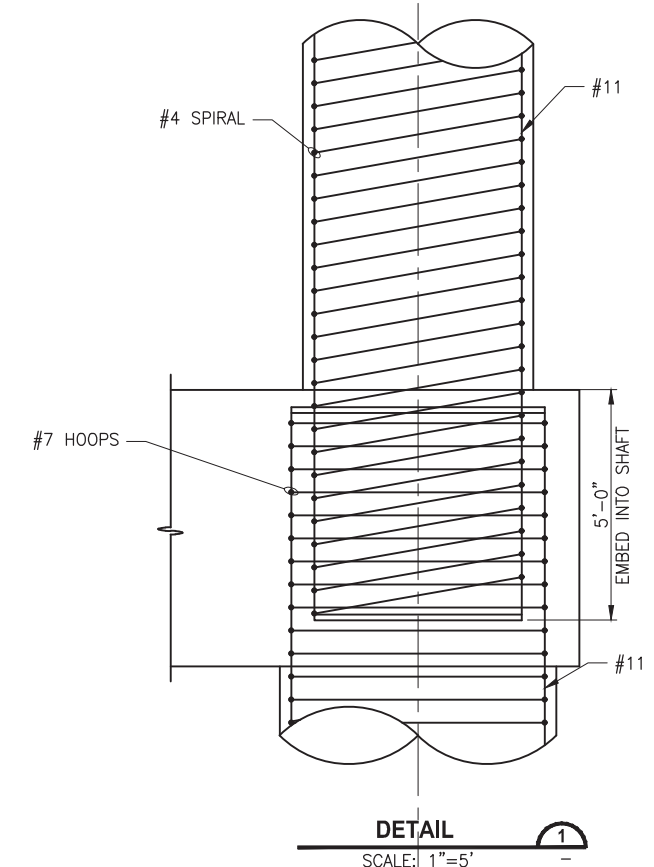
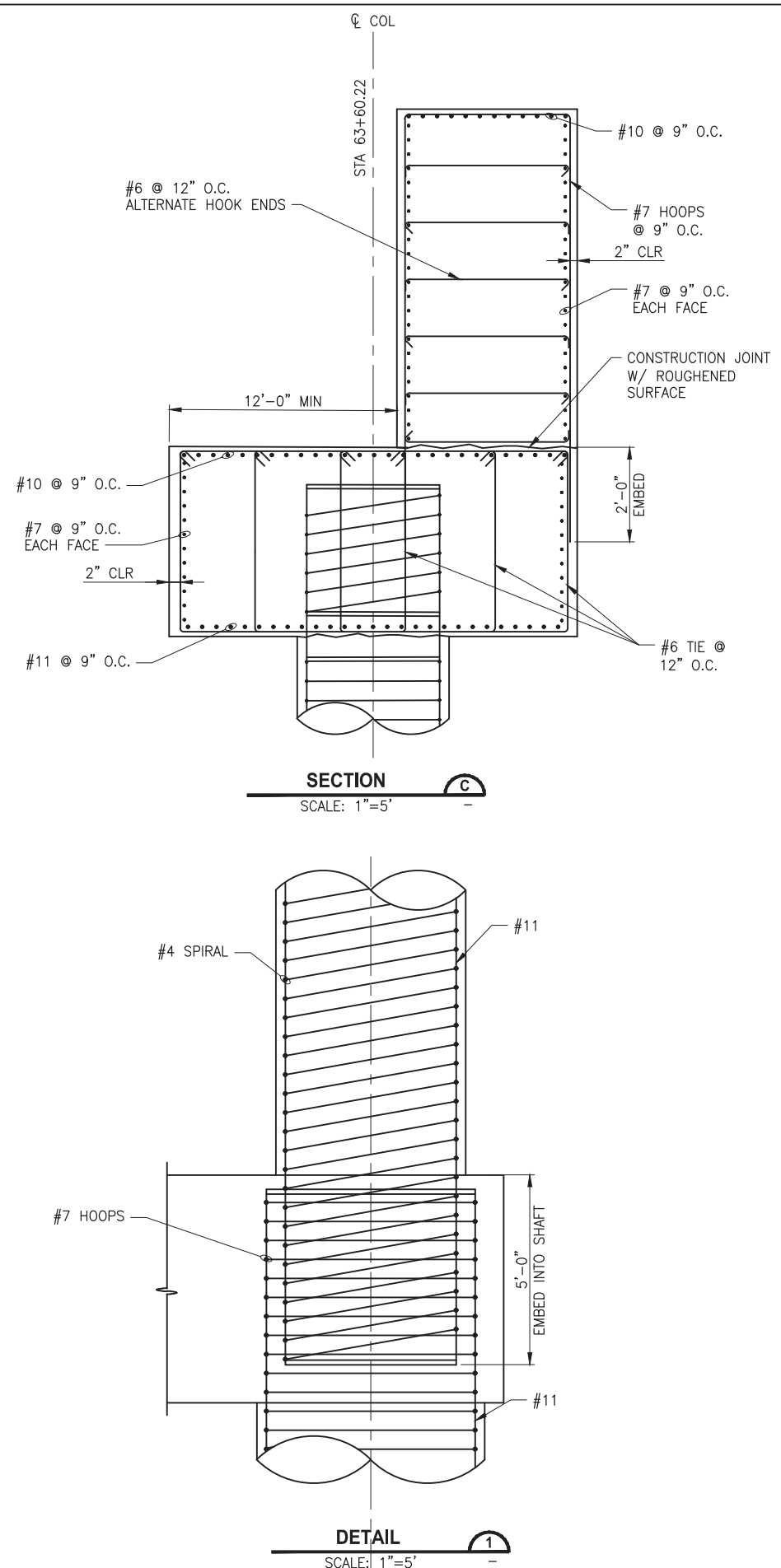
MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5989
IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BJS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: YY

NO.	DATE	REVISIONS

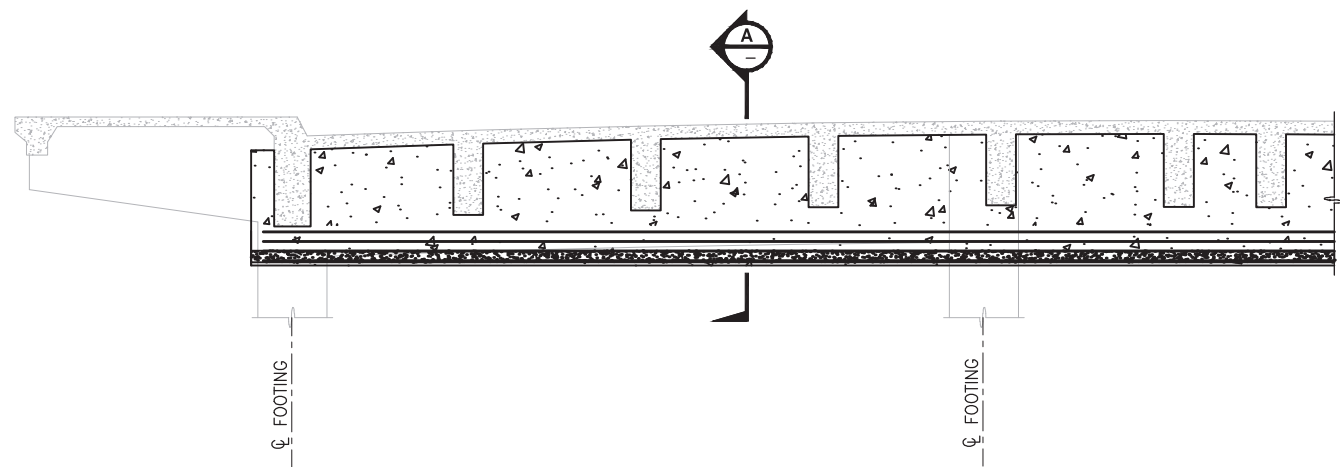


PIER 4
NOT TO SCALE

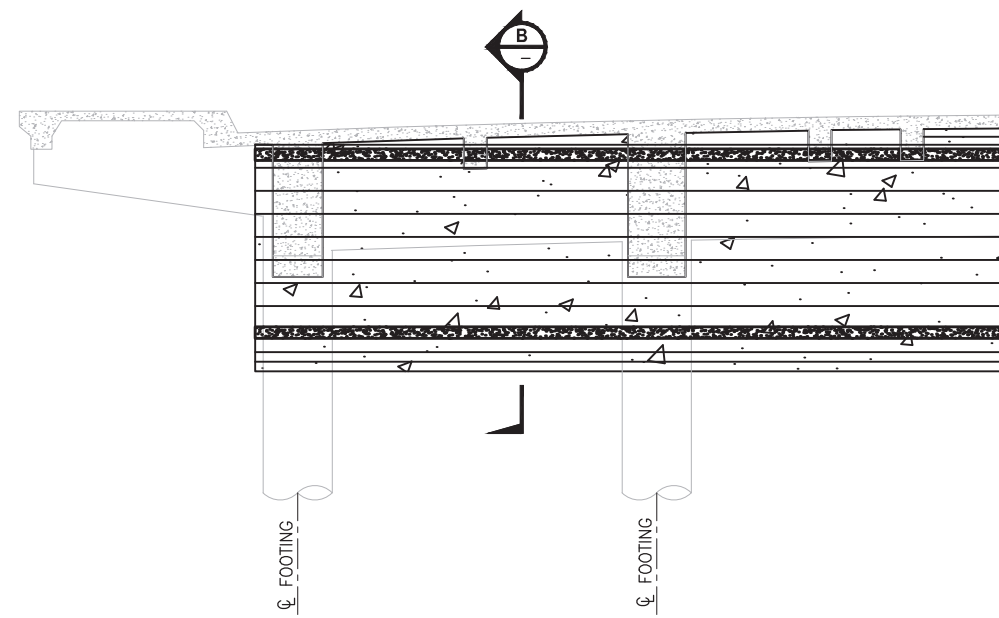


CONCEPTUAL PLANS
JUNE 2020

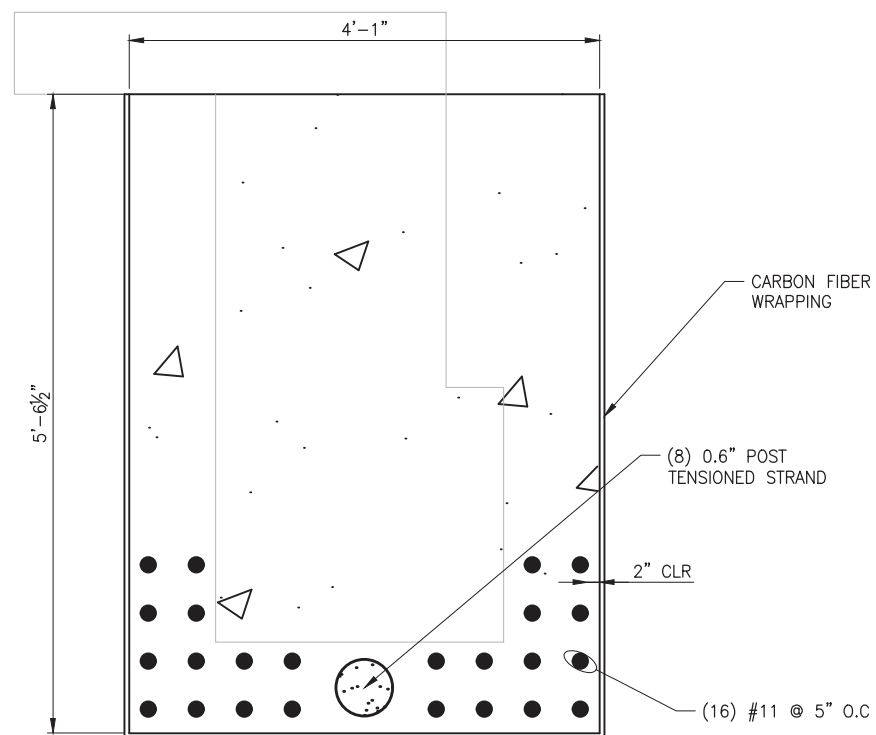
EARTHQUAKE READY BURNSIDE BRIDGE		PROJECT NO.: Proj No	
ENHANCED SEISMIC RETROFIT		DATE: 1/31/20	
PIER 4 REPLACEMENT		COUNTY ENGINEER	
		IAN B. CANNON P.E.	
MULTNOMAH COUNTY DEPARTMENT OF COMMUNITY SERVICES TRANSPORTATION DIVISION 1620 S.E. 190th AVE. PORTLAND, ORE. 97235-5999		DESIGNED BY: BJS/CRK	
COUNTY ENGINEER		DRAFTED BY: CRK/BRP	
REVISIONS		CHECKED BY: YY	
NO. DATE:		Y1 Y2	
Sheet No.		S18	



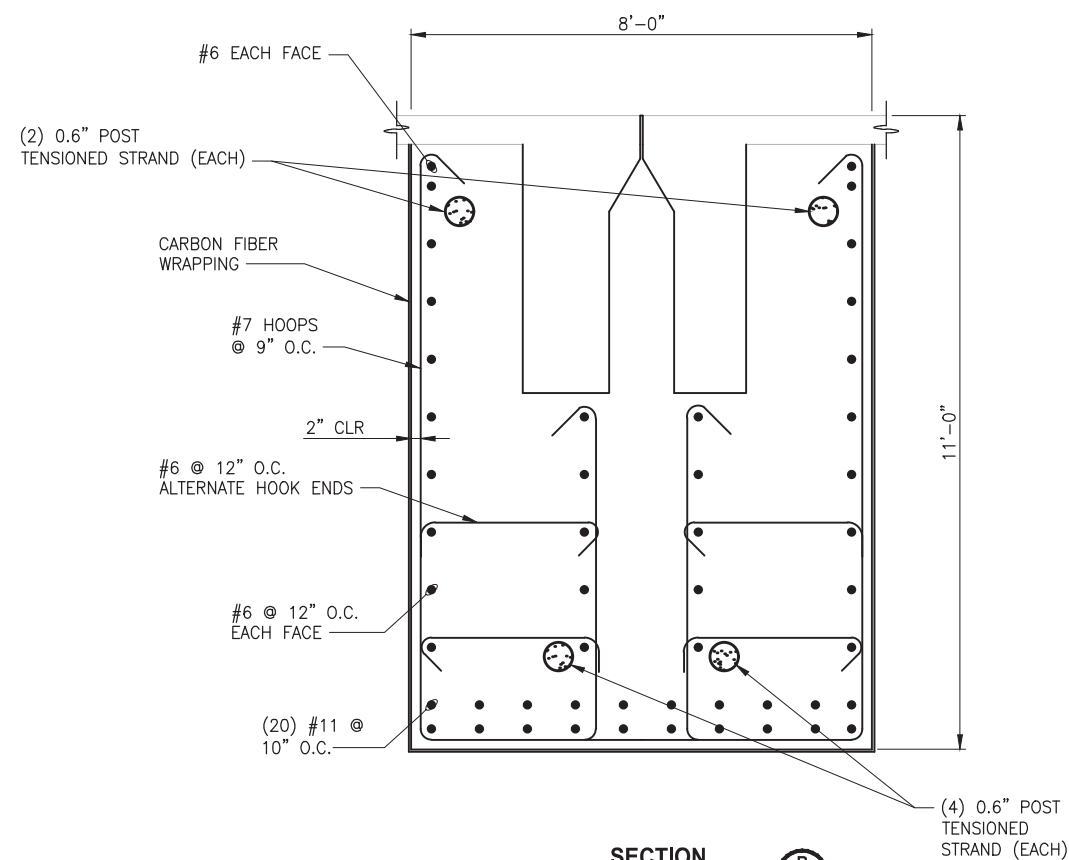
TYPICAL SPAN 2-14 & 29-34
SCALE: 1"=50'



TYPICAL SPAN 17-19
SCALE: 1"=50'



SECTION A
SCALE: NTS



SECTION B
NOT TO SCALE

CONCEPTUAL PLANS
JUNE 2020

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BJS/CRK
DRAFTED BY:
CRK/BRP
CHECKED BY:
YY

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NO.	DATE:	DESCRIPTION:

Sheet No.

S19

MULTNOMAH COUNTY

DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999



IAN B. CANNON P.E.

COUNTY ENGINEER

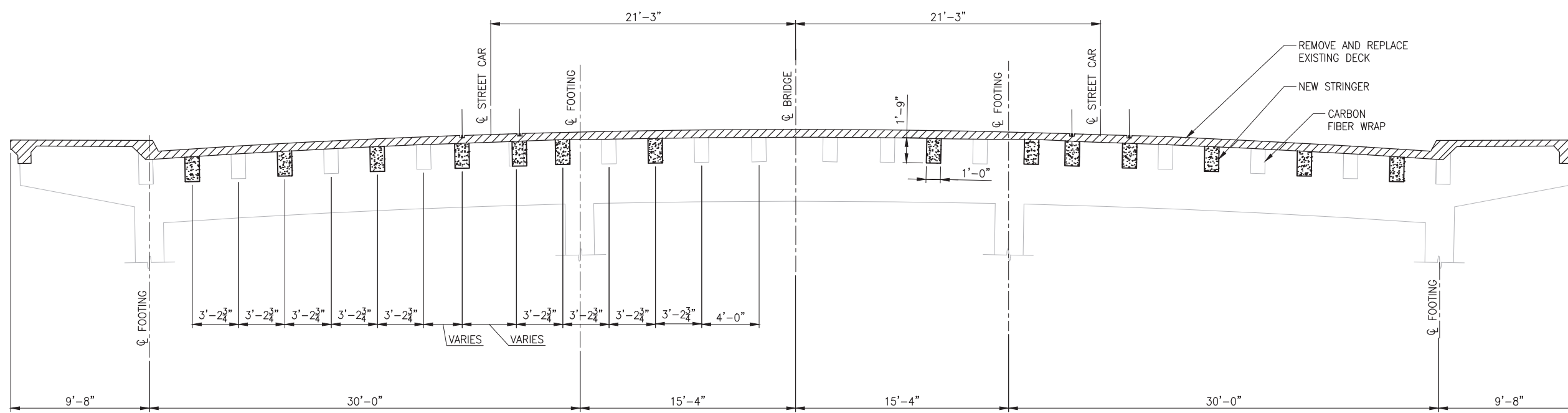
EARTHQUAKE READY BURNSIDE BRIDGE

ENHANCED SEISMIC RETROFIT

FLOOR BEAM & CONNECTION RETROFIT

BENT 2-19 & 29-34

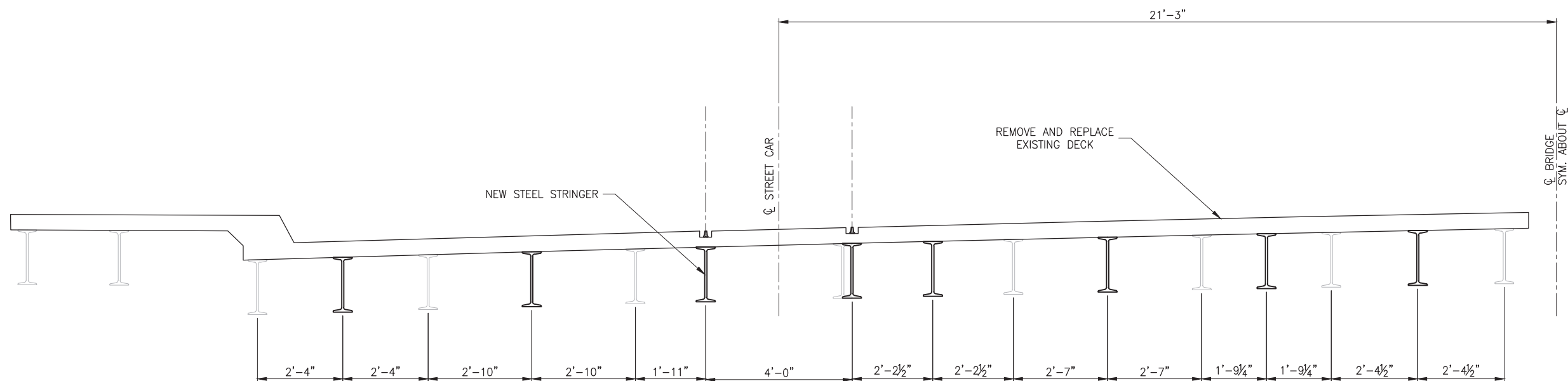
DATE: 1/31/20 PROJECT NO.: Proj No



TYPICAL SPAN 2-19 & 28-34
SCALE: 1"=25'

CONCEPTUAL PLANS
JUNE 2020

<p>MULTNOMAH COUNTY DEPARTMENT OF COMMUNITY SERVICES TRANSPORTATION DIVISION 1620 S.E. 190th AVE. PORTLAND, ORE. 97235-5999</p> <p>IAN B. CANNON P.E. COUNTY ENGINEER</p>		<p>EARTHQUAKE READY BURNSIDE BRIDGE ENHANCED SEISMIC RETROFIT BRIDGE DECK REPLACEMENT AND STRINGER RETROFIT - APPROACHES</p> <p>DATE: 1/31/20 PROJECT NO.: Proj No</p>														
<p>DESIGNED BY: BJS/CRK</p> <p>DRAFTED BY: CRK/BRP</p> <p>CHECKED BY: YY</p>	<p>REVISIONS</p> <table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th style="width: 10%;">NO.</th> <th style="width: 10%;">DATE</th> <th style="width: 80%;">REVISION</th> </tr> </thead> <tbody> <tr><td> </td><td> </td><td> </td></tr> <tr><td> </td><td> </td><td> </td></tr> <tr><td> </td><td> </td><td> </td></tr> <tr><td> </td><td> </td><td> </td></tr> </tbody> </table>	NO.	DATE	REVISION												
NO.	DATE	REVISION														
<p>Sheet No. S20</p>																



TYPICAL SPANS 25, 26, 27
NOT TO SCALE

CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
BRIDGE DECK REPLACEMENT AND
STRINGER RETROFIT - RIVER SPANS
DATE: 1/31/20 PROJECT NO.: Proj No

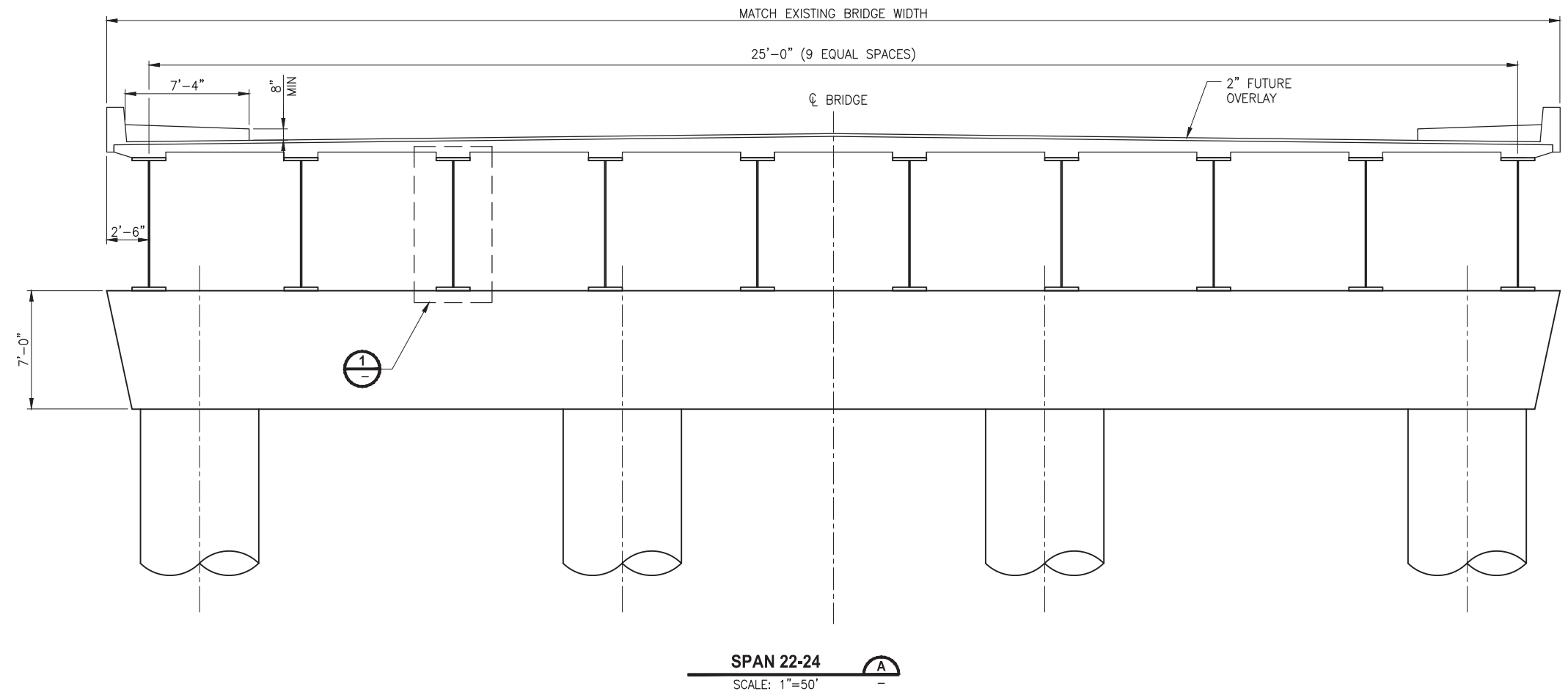
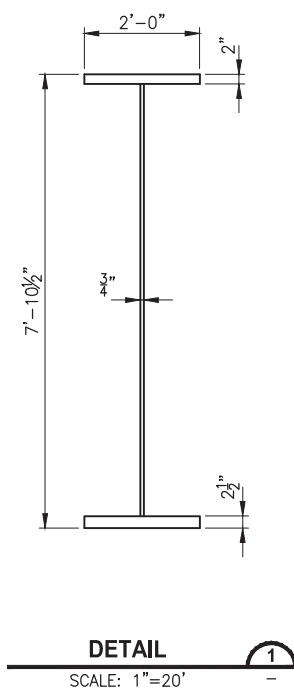
MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5989

IAN B. CANNON P.E. COUNTY ENGINEER

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DRAFTED BY: CRK/BRP
CHECKED BY: YY

NO.	DATE:	REVISIONS

Sheet No. S21



CONCEPTUAL PLANS
JANUARY 2020

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CRK/BRP
CHECKED BY:
YY

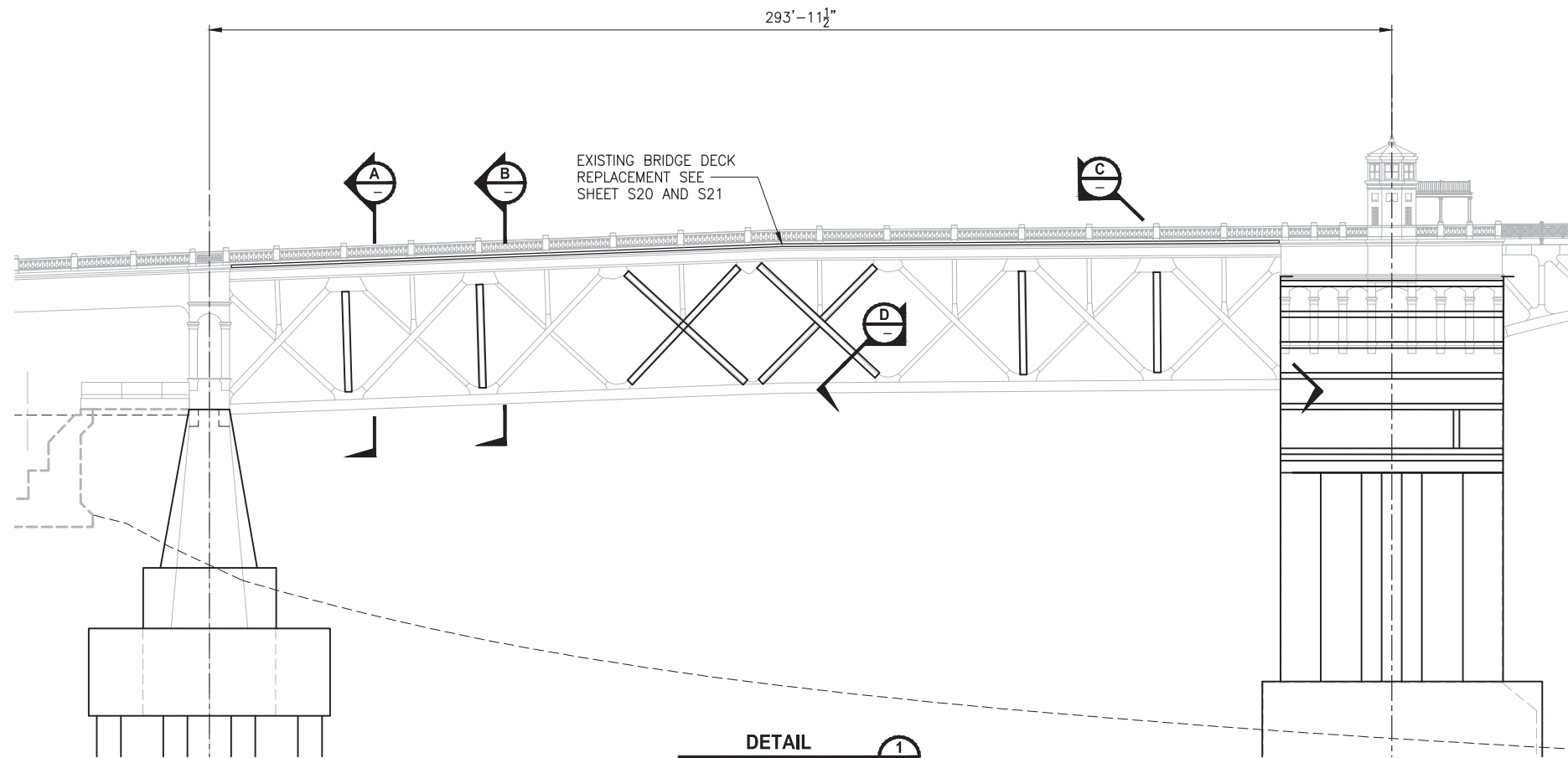
REVISIONS	
NO.	DATE:

Sheet No. S22

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999

IAN B. CANNON P.E. COUNTY ENGINEER

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED RETROFIT OF EXISTING CONDITION
BRIDGE SPAN REPLACEMENT
NEW SPAN 22, 23, 24
DATE: 1/31/20 PROJECT NO.: Proj No

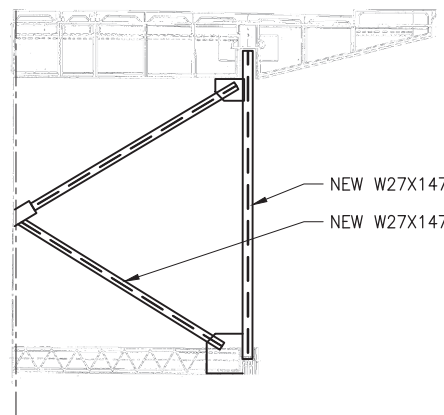


EXISTING BRIDGE DECK
REPLACEMENT SEE
SHEET S20 AND S21

DETAIL

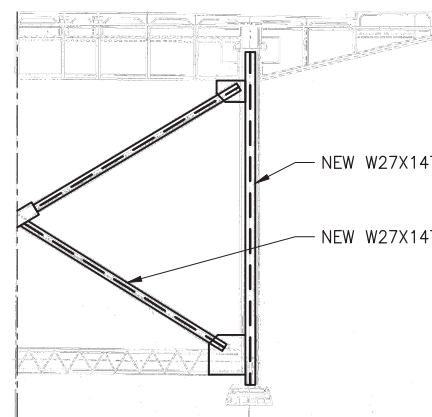
SCALE: 1"=20'

*NOTE: EAST TRUSS SHOWN, WEST TRUSS SIMILAR



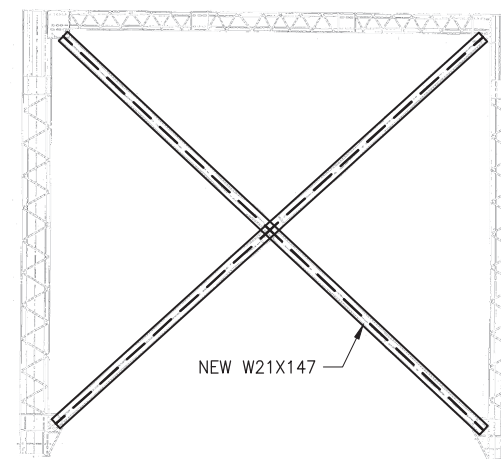
SECTION

SCALE: 1"=10'



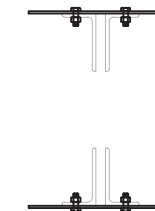
SECTION

SCALE: 1"=10'



SECTION

SCALE: 1"=10'



SECTION

SCALE: 1"=10'

CONCEPTUAL PLANS
JUNE 2020

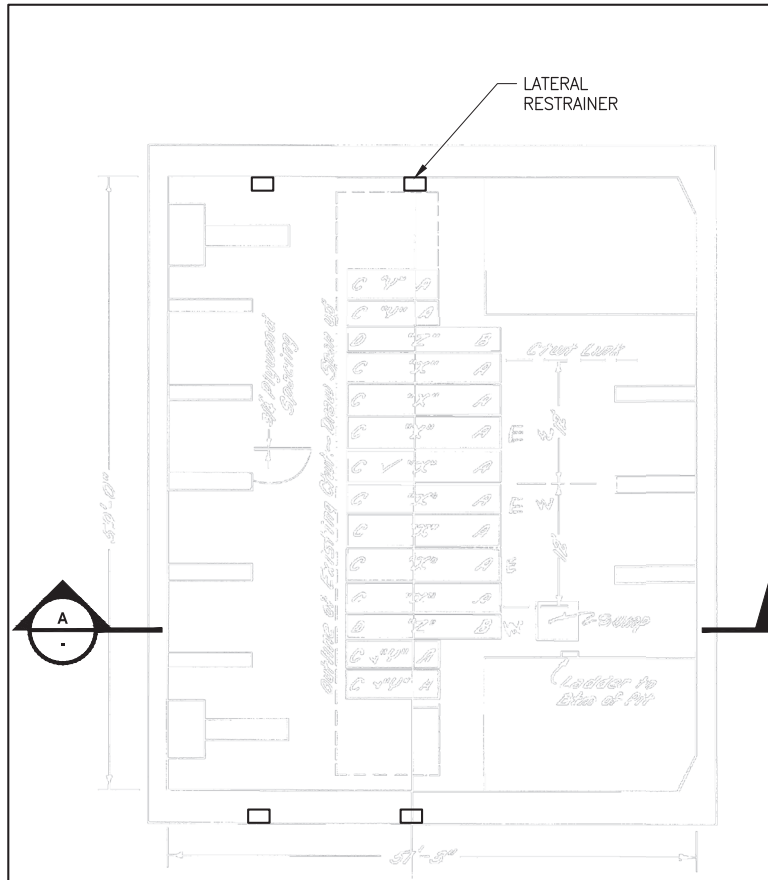
EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
STEEL TRUSS SPAN
TRUSS RETROFIT
DATE: 1/31/20 PROJECT NO.: Proj/No

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999
IAN B. CANNON P.E. COUNTY ENGINEER

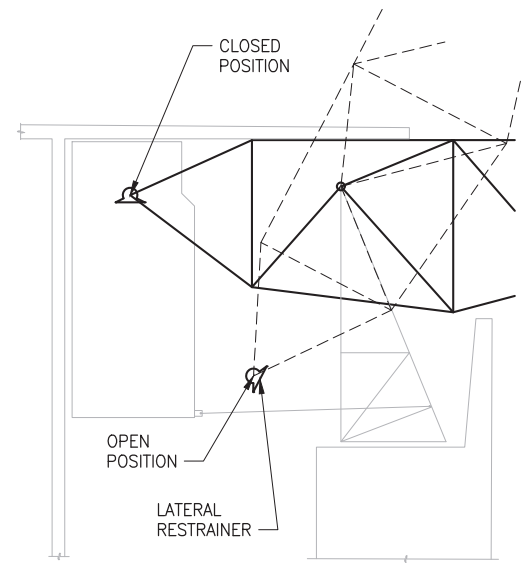
DESIGNED BY: BJS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: YY

NO.	DATE:	REVISIONS

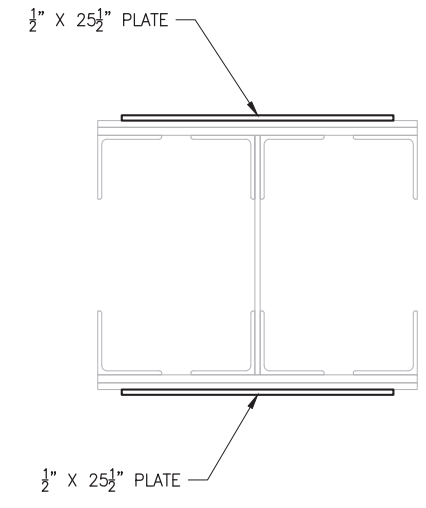
Sheet No. S23



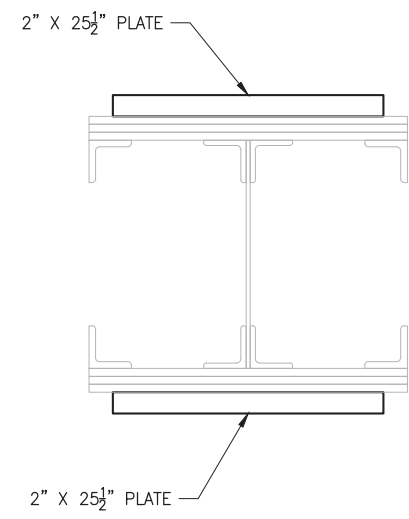
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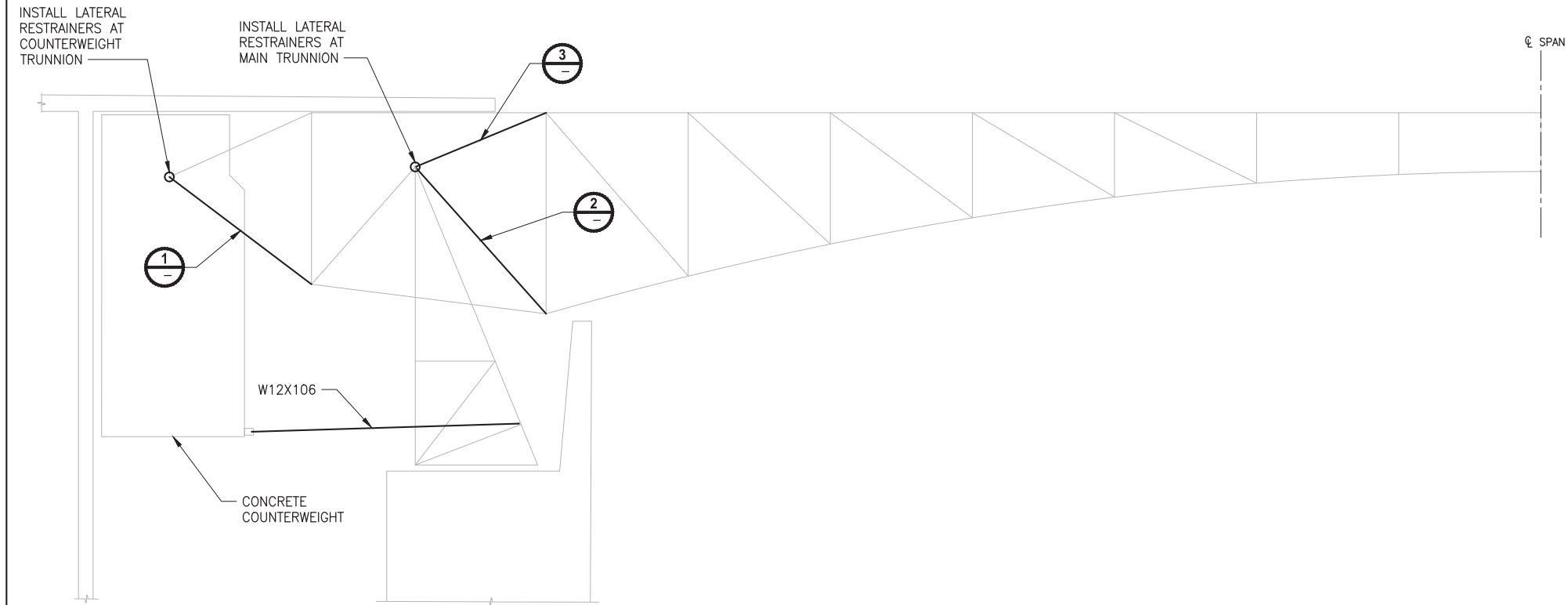
TRUNNION POSITIONS
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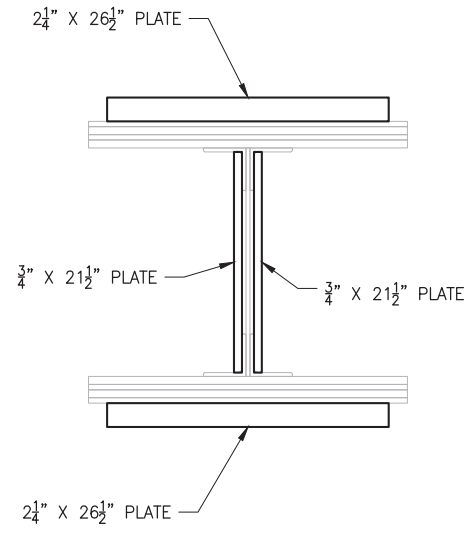
MEMBER C16
NOT TO SCALE



MEMBER T14
NOT TO SCALE



TRUSS TRUNION END
NOT TO SCALE



MEMBER T15
NOT TO SCALE

CONCEPTUAL PLANS
JUNE 2020

EARTHQUAKE READY BURNSIDE BRIDGE
ENHANCED SEISMIC RETROFIT
BASCULE LEAVES RETROFIT
TRUNNION AND COUNTERWEIGHT RETROFIT
DATE: 1/31/20 PROJECT NO.: Proj/No

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
TRANSPORTATION DIVISION
1620 S.E. 190th AVE. PORTLAND, ORE. 97233-5999
IAN B. CANNON P.E. COUNTY ENGINEER

DESIGNED BY: BJS/CRK
DRAFTED BY: CRK/BRP
CHECKED BY: YY

NO.	DATE	REVISIONS