Preliminary Geotechnical Engineering Report
South Kelso Railroad Grade Separation Project
Kelso, Washington

September 7, 2018

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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Excellence. Innovation. Service. Value
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Submitted To:
Mr. Jason Ruth, PE
HDR Inc.
700 Washington Street, Ste 450
Vancouver, Washington 98660

By:
Shannon & Wilson, Inc.
3990 Collins Way, Suite 100
Lake Oswego, Oregon 97035

(503) 210-4750
www.shannonwilson.com

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PRELIMINARY GEOTECHNICAL ENGINEERING REPORT
SOUTH KELSO RAILROAD GRADE SEPARATION PROJECT
KELSO, WASHINGTON

1.0 INTRODUCTION

1.1 Project Overview

This report presents the preliminary results of our field explorations, laboratory testing, geotechnical design evaluations and recommendations, and construction considerations for the proposed South Kelso Grade Separation project in Kelso, Washington. The location of the project site is shown on the Vicinity Map, Figure 1.

The City of Kelso is the project owner and HDR Engineering, Inc. (HDR), is leading the project design. Shannon & Wilson, Inc. (Shannon & Wilson), is providing preliminary geotechnical investigation services for the project under a subcontract to HDR. The preliminary results of our field explorations, laboratory testing, geotechnical design evaluations and recommendations, and construction considerations provided herein are for project 30 percent design level. For final design, additonal field explorations, including borings for bridge piers and Cone Penetrometer (CPT) tests for stone column areas, would be required.

1.2 Scope of Services

The purpose of the geotechnical investigation is evaluating the subsurface soil conditions, seismic hazards, and bridge and retaining wall foundation alternatives to support the project 30 percent design efforts including the bridge and retaining wall type size, and locations. Shannon & Wilson's services were conducted in accordance with the scope of services defined in the Master Subconsultant Agreement Task Order No. 0010 issued by HDR, dated October 26, 2017. The completed geotechnical services for the project consisted of the following tasks:

- Review available existing information and visit the site to observe existing site conditions, geologic hazards, site access for the field explorations, and mark proposed exploration locations;
- Develop a field exploration and testing work plan;
- Explore the subsurface conditions with 11 geotechnical borings, four (4) Cone Penetration Tests, six (6) Dynamic Cone Penetrometer, and collect soil samples from the

borings;

- Conduct laboratory testing on selected soil samples to characterize soils and develop soil properties for evaluation;
- Evaluate the site-specific seismic hazards, including ground motion, liquefaction potential, and other seismic-related hazards;
- Provide conceptual engineering mitigation alternative for liquefaction hazard;
- Evaluate the proposed bridge approach retaining walls, including site-specific seismic hazards, external stability including global stability, bearing resistance, and settlement;
- Provide design parameters for the proposed bridge approach retaining walls;
- Evaluate settlement of the foundation soils for the embankments and retaining walls;
- Evaluate stability of embankment slopes;
- Evaluate bridge foundation alternatives and provide design recommendations for the selected foundation type;
- Provide up to two new pavement sections for Hazel Street and South 3rd Avenue extension;
- Provide geotechnical construction considerations for earthwork, including site preparation, excavation, cut and fill slopes, structural fill material, fill placement, compaction, and wet weather construction;
- Prepare a Geotechnical Engineering Report summarizing our explorations, laboratory testing, geotechnical design recommendations, and construction considerations; and
- Prepare a Final Summary of Geotechnical Conditions memorandum.

2.0 PROJECT UNDERSTANDING

2.1 Site Description

The proposed project site can be separated into an east and west portion by an existing railroad that travels north-south through the project area. East of the existing railroad consists of three asphalt paved roads designated as South Pacific Avenue, Hazel Street and Douglas Street. Hazel Street and Douglas Street consist of two-lane roads that travel east-west, and South Pacific Avenue is currently a two-lane road that travels north-south. Adjacent to the three roads is a commercial development. North of Hazel Street is a mix of commercial and residential development.

West of the railroad, the proposed project site extends along the north perimeter of an existing golf course and is currently an unpaved, vegetated area. The majority of the proposed project site west of the railroad is relatively flat. However, the golf course is at a higher elevation than the surrounding area; therefore, a portion of the proposed alignment will traverse a moderate slope to reach the lower elevation. A creek traveling north-south is also present along the west side of the project alignment.

2.2 Project Description

The proposed project will consist of extension of the existing Hazel Street overcrossing; existing South Pacific Avenue and BNSF rails to South River Road; and improvements of Douglas Street and new South 3rd Avenue extension to Hazel Street. The proposed project design elements consist of a four-span bridge overcrossing BNSF rails and South Pacific Avenue; a single span bridge or culvert overcrossing a wetland area; retaining walls to retain the bridge approach embankments for the bridge overcrossing BNSF railroad tracks and South Pacific Avenue; new pavement for Hazel Street, Douglas Street, and South 3rd Avenue extension; and water quality infiltration facilities. Fill slope for approach embankment is also proposed.

Based on the 30 percent design drawing, the proposed bridge overcrossing South Pacific Avenue and BNSF rails is approximately 400 feet in length. The bridge will have three (3) spans and four (4) piers. The proposed bridge approach embankments for the bridge overcrossing BNSF railroad tracks and South Pacific Avenue is approximately 30 feet high and approximately 2*350=700 feet long.

Both sides of the proposed approach embankments will be retained by Mechanical Stabilized Earth (MSE) retaining walls at the bridge piers 1 and 4 Preliminary bridge design loads are provided by HDR and summarized in Table 1. The project alignment and area is shown on the Site and Exploration Plan, Figure 2. Thirty (30) percent design drawings are included in Appendix F.

TABLE 1
BRIDGE FOUNDATION PRELIMINARY DESIGN LOADS

Bent Location	**Factored Axial Compressive Resistance Per Shaft (kips)**		
	Strength	**Service**	**Extreme Event[1]**
1 (West)	1000	700	N/P
2 and 3 (Interior)	2700	1900	N/P
4 (East)	1000	700	N/P

Note:
1) Not provided.

3.0 SITE GEOLOGY AND SEISMIC SETTING

3.1 Site Geology

The City of Kelso, Washington, is situated on a former floodplain on the east bank of the Cowlitz River, just upstream from its confluence with the Columbia River. Here a thick section of alluvial sediment lies over bedrock formations. The Cowlitz River has deposited fine- to coarse-grained sands that include dacite gravel and pumice eroded from Mount St. Helens, in a large delta that interfingers with the predominantly fine-grained micaceous sands of the Columbia River.

Bedrock units include sedimentary rock and basalt flow rocks of the Cowlitz Formation and the Goble Volcanics. Lava flows of the middle Miocene age Columbia River Basalt Group overlie the oldest formations; and conglomerate and sandstone of the Pliocene Troutdale Formation overlie The Columbia River Basalt Group, the Goble Volcanics, and the Cowlitz Formation. Collectively these older formations comprise thousands of feet and tens of millions of years of geologic history, and they form the upland areas that rise above both sides of the Columbia River in the Kelso/Longview, Washington, to the Rainier, Oregon, area.

During the Ice Ages of the Pleistocene epoch (the period in geologic history from about 1.8 million years ago to about 10,000 years ago), sea level was about 400 feet lower than at present. During this period, the Columbia River and its tributaries (such as the Cowlitz River) eroded deep channels into the bedrock formations over which they flowed. Then, as sea level rose at the close of the Pleistocene, the rivers began depositing sand and gravel in their channels, gradually filling the deep channels and building the floodplains that we see today. Approximately 250 feet of alluvial sediment now lies between the ground surface and the bedrock surface beneath the low-lying areas of Kelso.

In more recent times, the eruption of Mount St. Helens on May 18, 1980, carried tremendous volumes of sediment down the Cowlitz River, depositing it in the river channel through the Kelso/Longview area and raising the bottom of the river. To remove the sediment, the U.S. Army Corps of Engineers dredged the river, placing an approximate three (3) million cubic yards of material as Fill in the area that is now occupied by the Three Rivers Golf Course. The dredged material raised the elevations in areas of the golf course from 2 to 30 feet (Alvord, 2003).

The Kelso area has been extensively geologically mapped by Livingston, 1966, and Walsh and others, 1987. According to geologic mapping, the project site is underlain by Quaternary age alluvium deposited by the Cowlitz and Columbia rivers. The alluvium is described as consisting of silt, sand, and gravel, and some localized deposits of peat along streams.

3.2 Seismic Setting

Earthquakes in the Pacific Northwest occur largely as a result of the region's proximity to an active convergent plate boundary, where dense oceanic crust is subducting beneath less dense continental crust. At this subduction plate boundary, known as the Cascadia Subduction Zone (CSZ), the Explorer, Juan de Fuca, and Gorda Oceanic Plates are subducting beneath the overriding, westward-moving North American Plate.

Oblique convergence of these plates not only results in east-west compressive strain, but also results in dextral (right lateral) shear, clockwise rotation, and north-south compression of accreted crustal blocks that form the leading edge of the North American Plate (Wells and others, 1998). The CSZ extends about 750 miles from northern California to southern British Columbia, and lies approximately 125 miles west of the project site. Within the present understanding of the regional tectonic framework and historical seismicity, three broad seismogenic sources have been identified. These include the following:

- A mega-thrust source at the interface between the North American and Juan de Fuca plates in the CSZ;
- A deep intraslab source in the subducted Juan de Fuca Plate, within the CSZ; and
- A shallow crustal source within the North American Plate.

The following sections briefly describe the location, characteristics, and seismicity of each of the sources.

3.2.1 Cascadia Subduction Zone (CSZ): Mega-Thrust Interface Source

CSZ mega-thrust earthquakes originate along the interface between the subducting oceanic plates and the North American plate. Because of the significant uncertainty of the landward extent of a potential rupture surface, estimates of the closest distance between the project and potential rupture surface range from about 80 to 135 horizontal miles. Focal depths for mega-thrust earthquakes are commonly on the order of about 15 to 25 miles.

Rupture of the interface could result in earthquakes with moment magnitudes on the order of 8.5 to over 9.0, with strong shaking that lasts for several minutes. No large earthquakes have occurred in this zone during historic times (the last 170 years). However, geologic evidence suggests that coastal estuaries have experienced rapid subsidence at various times within the last 2,000 years (e.g., Atwater, 1987; Atwater and Hemphill-Haley, 1997), as a result of tectonic movement associated with mega-thrust earthquakes on the CSZ. It appears that ruptures of this zone have occurred at irregular intervals that span from about 100 to more than 1,200 years, with an average recurrence interval of about 300 to 500 years (Atwater and Hemphill-Haley, 1997). Based on historical tsunami records in Japan (Satake and others, 1996) the most recent interplate event on the CSZ was a moment magnitude (M_w) 9 event on January 26, 1700.

3.2.2 Cascadia Subduction Zone: Intraslab Source

CSZ intraslab earthquakes originate from within the subducting oceanic plates, as a result of down-dip tensional forces and bending caused by mineralogical and density changes in the plates at depth. These earthquakes typically occur 28 to 37 miles beneath the surface. The nearest seismogenic intraslab portion of the Juan de Fuca plate is approximately 30 to 60 miles below the Portland area. Ludwin and others (1991) estimate that the maximum M_w from this source zone would be about 7.5.

Ground shaking produced by intraplate earthquakes would generally be less intense and less prolonged in the Portland area than ground motions generated by large subduction zone interface earthquake events. Historic seismicity from this source zone includes the 1949 M_w 6.7 Olympia earthquake, the 1965 M_w 6.7 earthquake between Tacoma and Seattle, and the 2001 M_w 6.8 Nisqually earthquake. While intraslab events have occurred frequently in the Puget Sound area, they are historically rare in Oregon.

3.2.3 Shallow Crustal Source

Shallow crustal earthquakes within the North American Plate have historically occurred in a diffuse pattern within the Pacific Northwest, typically within the upper 4 to 19 miles of the continental crust. Mabey and others (1993) concluded, from their analysis of local geologic features, that a crustal earthquake of up to M_w 6.5 could occur virtually anywhere in the Portland area. The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades earthquake at approximate M_w 6.5 to 7.0. Other examples include the 1993 M_w 5.6 Scotts Mill earthquake and the 1993 M_w 6.0 Klamath Falls earthquake.

Shallow crustal faults and folds throughout Oregon have been located and characterized by the United States Geological Survey (USGS). The USGS provides approximate fault locations and a detailed summary of available fault information in the USGS Quaternary Fault and Fold Database.

The database defines four categories of faults, Class A through D, based on evidence of tectonic movement known, or presumed, to be associated with large earthquakes during Quaternary time (within the last 2.6 million years). For Class A faults, geologic evidence demonstrates that a tectonic fault exists and that it has likely been active within the Quaternary period. For Class B faults, there is equivocal geologic evidence of Quaternary tectonic deformation, or the fault may not extend deep enough to be considered a source of significant earthquakes. Class C and D faults lack convincing geologic evidence of Quaternary tectonic deformation, or have been studied carefully enough to determine that they are not likely to generate significant earthquakes.

According to the USGS Quaternary Fault and Fold database (USGS, 2017) there are four Class A features within approximately 30 miles of the project site. Their names, general locations relative to the site, and the time since their most recent deformation are summarized in Table 2.

The CSZ itself is approximately 125 miles west of the site, with an average slip rate of approximately 40 millimeters (1.5 inches) per year and the most recent deformation occurring about 300 years ago (Personius and Nelson, 2006).

TABLE 2
USGS CLASS A FAULTS WITHIN AN APPROXIMATE 30-MILE RADIUS OF THE PROJECT SITE

Fault Name	USGS Fault Number	Approximate Fault Length	Approximate Distance and Direction from Project Alignment[a]	Slip Rate Category[b]	Time Since Last Deformation[b]
Portland Hills fault	877	30 miles	26 miles, 177 degrees	< 0.2 mm/yr	<1.6 Ma
Gales Creek fault	718	45 miles	34 miles, 215 degrees	< 0.2 mm/yr	<1.6 Ma
Helvetia fault	714	4 miles	36 miles, 184 degrees	< 0.2 mm/yr	<1.6 Ma
Oatfield fault	875	18 miles	35 miles, 174 degrees	< 0.2 mm/yr	<1.6 Ma

Notes:

1) Approximate distance from project alignment to nearest extent of fault mapped at the ground surface.
2) Ma = "mega-annum" or million years ago; ka = "kilo-annum" or thousand years ago; mm/yr = millimeters per year.

4.0 FIELD EXPLORATIONS

The field exploration program included 11 geotechnical borings, designated B-1 through B-11; four Cone Penetration Tests (CPTs), designated CPT-1 through CPT-4; and six Dynamic Cone Penetrometer (DCP) tests, designated DCP-1 through DCP-6.

The completed geotechnical borings were drilled between April 16 and 24, 2018, and on May 8, 2018, to depths ranging from 11.5 to 141.5 feet below existing ground surface (bgs) by Western States Soil Conservation, Inc. (Western States), of Hubbard, Oregon, with a CME 350 track mounted drill rig, and CME 75 truck mounted drill rig using mud rotary and hollow-stem auger drilling techniques.

A member of the Shannon & Wilson engineering staff was on site during the explorations to locate the borings, observe drilling, collect samples, and maintain logs of the materials encountered. Details of the drilled boring exploration program, including techniques used to advance and sample the borings, logs of the materials encountered, and methods and results are presented in Appendix A. A member of the Shannon & Wilson engineering staff performed DCP tests. DCP test results are also presented in Appendix A.

The CPT portion of the explorations were completed between April 19 and 21, 2018, and ranged in depth from 10.5 to 150.6 feet bgs by Oregon Geotechnical Explorations, Inc. (OGS), of Keize, Oregon, with a truck-mounted CPT rig and a track-mounted CPT rig. Details of the CPT exploration program, and the output results, are presented in Appendix B, Cone Penetration Test (CPT) Results.

The locations and elevations of the borings, DCPs and CPTs were not surveyed and were referenced to nearby existing structures and should be considered approximate. Approximate locations of the explorations are shown on the Site and Exploration Plan, Figure 2. The elevations of the explorations were estimated from Lidar.

5.0 LABORATORY TESTING

The samples we obtained during our field explorations were transported to our laboratory for further examination. We then selected representative samples for a suite of laboratory tests. The laboratory testing program included moisture content tests, Atterberg limits tests, particle size analyses, and a one-dimensional consolidation test. Laboratory tests were performed by Shannon & Wilson in accordance with applicable ASTM International (ASTM) standard test procedures. Results of the laboratory tests and brief descriptions of the test procedures are presented in Appendix C, Laboratory Testing Results. Results are also presented graphically on the boring logs in Appendix A.

6.0 DISCUSSIONS OF SUBSURFACE CONDITIONS

The explorations and laboratory testing were performed to evaluate geotechnical soil and groundwater conditions for the proposed new alignment. Our observations are specific to the locations, depths, and times noted on the logs and may not be applicable to all areas of the site. No amount of explorations or testing can precisely predict the characteristics, quality, or distribution of subsurface and site conditions. Potential variation includes, but is not limited to, the following:

- The conditions between and below explorations may be different.
- The passage of time or intervening causes (natural and manmade) may result in changes to site and subsurface conditions.
- Groundwater levels and flow directions may fluctuate due to seasonal, irrigation-related, and recharge source variations.

If conditions different from those described herein are encountered during construction, we should review our description of the subsurface conditions and reconsider our conclusions and recommendations.

6.1 Geotechnical Units

During our field explorations we encountered material of two main geologic units: Fill and Quaternary Age Alluvium. We further grouped the geologic units into seven geotechnical units, as described below. Our interpretation of the subsurface conditions is based on the explorations and regional geologic information from published sources. The geotechnical units are as follows.

- **Fill:** stiff to very stiff SILT to Sandy SILT (ML); loose to medium dense Silty SAND (SM); medium dense to dense Poorly graded SAND with gravel (SP); and dense Poorly graded GRAVEL with sand (GP); contains organics and rootlets in some areas;
- **Silt Alluvium:** very soft/very loose to stiff/medium dense SILT to Sandy SILT (ML);
- **Silty Sand Alluvium:** very loose to dense Silty SAND (SM);
- **Sand Alluvium 1:** very loose to dense Poorly graded SAND (SP);
- **Sand with Gravel Alluvium:** very loose to dense Poorly graded SAND with trace gravel (SP);
- **Gravel Alluvium:** very loose to medium dense Poorly graded GRAVEL with sand (GP); and
- **Sand Alluvium 2:** loose to dense Poorly graded SAND (SP).

These geotechnical units were grouped based on their engineering properties, geologic origins, and their distribution in the subsurface. Our interpretation of their distribution in the subsurface is shown on the Interpretive Geologic Profile A-A’, Figure 3, and Interpretive Geologic Profile B-B’, Figure 4. The location of the profiles is shown on the Site and Exploration Plan, Figure 2. The profiles are interpretive, and variations in subsurface conditions may exist between the borings.

Contacts between the units may be more gradational than shown in the profiles and in the Logs of Test Borings in Appendix A. Standard Penetration Test (SPT) N-values presented on the logs and discussed below are in blows per foot (bpf), as counted in the field. No corrections have been applied. The sections below describe the geotechnical unit characteristics in greater detail.

6.1.1 Fill

Fill is material placed by humans, usually during land or roadway development, or, in the case of the project site, in the stockpiling of dredged material from the Cowlitz River. Fill was encountered in borings B-1 through B4, B-6 through B-8 and B-10, from the ground surface to depths ranging from about 2.5 to 20 feet.

The Fill material was generally the thickest in borings B-6 and B-7, which encountered Fill to the depths of 15.5 and 20 feet, respectively. Both borings B-6 and B-7 were performed in the Three Creek Golf Course on top of a hill created from material dredged from the Cowlitz River after the Mount St. Helens eruption.

Pavement sections included in the Fill unit were encountered at the ground surface in Borings B-3 and B-4. These sections generally consisted of an estimated 4 to 6 inches of asphalt concrete over base aggregate consisting of Poorly graded GRAVEL with sand (GP). A thin layer of Gravel placed for a parking area on the shoulder of South Pacific Ave was encountered at the surface in boring B-1.

Composition of the Fill throughout the project site was highly variable and included brown, very soft/very loose to stiff/medium dense SILT to Sandy SILT (ML); brown and dark gray, loose to medium dense, Silty SAND (SM); brown, gray-brown and dark gray, medium dense to dense Poorly graded SAND with gravel (SP); and gray and gray-brown Poorly graded GRAVEL with sand (GP). Some samples included trace organic debris, roots and rootlets.

SPT N-values in the Fill unit ranged from 7 to 36 blows per foot (bpf) and averaged about 22 bpf. Natural moisture content tests of two specimens indicated moisture contents of 20 and 12 percent.

6.1.2 Silt Alluvium

Silt Alluvium was encountered at various depths in borings B-1, B-3, B-7, B-8 and B-9. In borings B-1, B-7 and B-8, the Silt Alluvium was encountered below Fill at the depths of 2.5, 5 and 20.5 feet respectively; and the thickness of the unit in varied from about 2.75 to 4.5 feet. In boring B-9, the Silt Alluvium was encountered underlying Silty Sand Alluvium at a depth of 5 feet and extended to a depth of 8.7 feet. In borings B-3 and B-8, Silt Alluvium was encountered below the Gravel Alluvium at depths of 50 and 45 feet respectively, and it was also encountered deeper in boring B-8 between the depths of 80.75 and 85 feet.

In general, the Silt Alluvium consists of brown, gray, dark gray, olive-brown, and light gray, very soft/very loose to stiff/medium dense SILT (ML) with varying amounts of sand. The soil is moist or moist to wet, nonplastic or nonplastic to low plasticity; and the sand constituent is typically fine to medium grained, and occasionally fine to coarse grained. Trace organics, mica flakes, or iron oxidation and staining was observed in some samples.

SPT N-values in the unit ranged from 1 to 13 bpf and averaged 6 bpf. Results of natural moisture content tests of three specimens indicated moisture contents of 41, 42, and 45 percent. Sieve analyses of three specimens indicated fines contents of 79, 52, and 82 percent (by dry weight).

6.1.3 Silty Sand Alluvium

Silty Sand Alluvium was generally encountered near the surface or under the Fill material, as thin interbeds occurring underlying the Gravel Alluvium or within the Sand Alluvium 1. In borings B1, B-7 and B-8 the Silty Sand Alluvium was encountered underlying the Silt Alluvium, and, in boring B-1, extended from 5.5 to 7.5 feet, boring B-7 from 25 to 30 feet, and in boring B-8 from 7.75 to 15 feet.

Silty Sand Alluvium was encountered beneath the Fill in boring B-2 from the depth of 7.5 to 15 feet, boring B-3 from 2.5 to 5 feet, and in boring B-6 from 15.4 to 20 feet. In borings B-5, B-9 and B-11, the material was encountered underlying the sod at the surface and extended to the depths of 5 feet, 5 feet, and 10 feet, respectively. In boring B-4, Silty Sand was not encountered near the surface, but was encountered as a thin interbed underlying a Gravel Alluvium layer at a depth of 45 feet. A thin interbed of Silty Sand Alluvium was also encountered underlying this same layer of Gravel Alluvium in boring B-2 at a depth of 55 feet. In boring B-5, Silty Sand Alluvium was also encountered between 20.3 and 25 feet, but this layer does not appear to be continuous and was not encountered in boring B-4.

In general, the Silt Sand Alluvium consists of brown, dark brown, gray-brown, gray, and red-yellow, very loose to medium dense, Silty SAND (SM). The soil is moist or wet with depth, the sand is fine-grained, or occasionally fine to medium or fine to coarse grained, and the fines' constituent is nonplastic. Trace organics or rootlets were observed in some samples and the material was often observed to be stratified.

Trace interbeds of SILT (ML) were observed in a sample in boring B-4, and fine to coarse, subangular to subrounded gravel was observed in a sample from boring B-2 at a depth of 55 feet. SPT N-values in the unit ranged from 0 to 28 bpf and averaged 10 bpf. Results of natural moisture content tests of three specimens indicated moisture contents of 43, 41, and 19 percent. Sieve analyses of same three specimens indicated fines contents of 21, 36, and 46 percent (by dry weight).

6.1.4 Sand Alluvium 1

The Sand Alluvium underlying the project site was split into two units: an upper Sand Alluvium 1 unit and a lower Sand Alluvium 2 unit. The upper unit is separated from the lower unit by a layer of Sand with Gravel Alluvium and/or Gravel Alluvium. All borings except boring B-8 encountered the Sand Alluvium 1 unit.

Sand Alluvium 1 was generally encountered underlying the Silty Sand Alluvium or Silt Alluvium at depths of between 2.5 and 30 feet, and the thickness of the layers vary from 5 to 20 feet. The Sand Alluvium 1 unit is often interbedded with Sand with Gravel Alluvium or Silty Sand Alluvium and was often encountered in two to three individual interbeds to depths extending from 30 to 50 feet. In general, Sand Alluvium 1 consists of brown, gray, dark gray, and gray-brown, very loose to medium dense, Poorly graded SAND (SP). The sand constituent is fine to medium grained and occasionally fine or fine to coarse grained, and moist or wet.

Trace amounts of sand-sized pumice fragments were observed in some samples. SPT N-values in the unit ranged from 2 to 28 bpf and averaged 10 bpf. Natural moisture contents ranged from 24 to 33 percent and averaged 29 percent. Sieve analyses of three specimens indicated fines contents of three (3), two (2) and three (3) percent (by dry weight).

6.1.5 Sand with Gravel Alluvium

Sand with Gravel Alluvium was encountered underlying Sand Alluvium 1 in borings B-1 through B-8, at depths ranging from 7.5 to 35 feet. Boring B-6 and B-7 were terminated within the unit at depths of 46.5 and 61.5 feet, respectively. Borings B-9 through B-11 did not encounter the unit but were all terminated at a depth of 11.5 feet. For the borings that penetrated through the Sand with Gravel Alluvium unit, the thickness varied from 10 to 25 feet. In boring

B-4 the unit was interbedded with Sand Alluvium 1 and was encountered from depths of 7.5 to 10 feet, 15 to 20.5 feet, and 25 to 40 feet.

In borings B-3, B-4 and B-8, the Sand with Gravel Alluvium directly overlies Gravel Alluvium, and all three borings penetrated the Sand with Gravel unit at a depth of 40 feet. In boring B-2, an approximate 8-foot-thick layer of Sand Alluvium 1 separates the overlying Sand with Gravel Alluvium from the underlying Gravel Alluvium. In general, the Sand with Gravel Alluvium consists of very loose to dense, brown-gray, gray, dark gray, and dark gray to yellow-brown, Poorly graded SAND with trace gravel to Poorly graded SAND with gravel (SP).

The gravel constituent is fine to coarse, and subangular to rounded, the sand constituent is fine to coarse, and the soil is typically wet. SPT N-values in the unit ranged from 2 to 27 bpf and averaged 13 bpf. Natural moisture contents ranged from 24 to 28 percent and averaged 26 percent. Sieve analyses indicated fines contents that ranged from 1 to 5 percent and averaged three percent (by dry weight).

6.1.6 Gravel Alluvium

Gravel Alluvium was encountered in boring B-2 at a depth of 48 feet, and in borings B-3, B-4, and B-8 at the depth of 40 feet. In boring B-2 the Gravel Alluvium was approximately seven feet thick; while in borings B-3, B-4 and B-8 the unit was approximately five feet thick. A thin layer of Silty Sand Alluvium underlies the Gravel Alluvium in borings B-2 and B-4. The Gravel Alluvium in boring B-3 is underlain by an approximate 5-foot-thick layer of Sand with Gravel Alluvium followed by Silt Alluvium, while in boring B-8 the Gravel Alluvium is directly underlain by Silt Alluvium. The Gravel Alluvium typically consists of very loose to medium dense, gray to dark gray, Poorly graded GRAVEL with sand (GP). The gravel constituent is generally fine to coarse, and subangular to subrounded. The sand constituent is fine to coarse grained, and trace organics were observed in a sample in boring B-2. SPT N-values in the unit ranged from 2 to 18 bpf and averaged 12 bpf.

No natural moisture content test and no sieve analyses were performed on any specimens of Gravel Alluvium.

6.1.7 Sand Alluvium 2

The Sand Alluvium 2 unit is generally separated from the upper Sand Alluvium 1 unit by a layer of Sand with Gravel Alluvium and/or Gravel Alluvium. Sand Alluvium 2 was assigned a different designation, due to its average higher SPT N-values, its position in the stratigraphic column, and by the presence of mica flakes, which may indicate the Columbia River as its depositional source (as opposed to the Cowlitz River). Borings B-1 through B-5 and B-8 encountered the unit at depths of between 45 and 73.9 feet, and all borings that encountered the unit were terminated within the unit at depths of between 51.5 and 141.5 feet. Borings B-6, B-7 and B-9 through B-11 did not encounter the unit and were terminated at the depths of 46.5, 61.5, 11.5, 11.5 and 11.5 feet, respectively.

Sand Alluvium 2 generally consists of loose to dense, gray, dark gray, and gray-brown, Poorly graded SAND (SP). The sand constituent is fine to medium grained, and occasional layers of fine to coarse grained sand were encountered; and encompassed within these layers were fine to coarse sand sized fragments of pumice. The soil is generally wet, and micaceous, and in boring B-8, the unit was interbedded with the Silt Alluvium unit and the Silty Sand Alluvium unit. SPT N-values in the unit ranged from 10 to 48 bpf and averaged 29 bpf. Natural moisture contents ranged from 24 to 31 percent and averaged 27 percent. Sieve analyses indicated fines contents that ranged from three to seven percent and averaged 5 percent (by dry weight).

6.2 Groundwater

Except for boring B-9 (which was advanced using hollow-stem auger technique) the borings were advanced using mud rotary drilling technique, which introduce drilling fluid into the boreholes. This makes it difficult to discern the depth to groundwater, if it is encountered during drilling. Groundwater was observed in the hollow-stem augers during drilling of boring B-9 and was measured at an approximate depth of 8 feet below ground surface (bgs). To monitor groundwater levels, a Vibrating Wire Piezometer (VWP) was installed in boring B-1 to a depth of 38.75 feet. Measurements of groundwater depth taken from the piezometer are included in Table 3.

TABLE 3
VIBRATING WIRE PIEZOMETER MEASUREMENTS

Measurement Date	Groundwater Depth[1] (feet)	Groundwater Elevation[1] (feet)
May 8, 2018	7.4	12.6
June 14, 2018	6.9	13.1
September 6, 2018	9.7	10.3

Notes:
1) Groundwater depth represents the depth below the existing ground surface.
2) Ground surface is assumed at elevation 20 feet.

No other instrumentation or observation wells were installed in any of the other borings. Generally, we anticipate groundwater highs at the site to occur in the winter and spring and groundwater lows to occur in the early to mid-fall season (before the onset of significant rainfall), which may fluctuate with changes in the Cowlitz River surface elevations.

7.0 SEISMIC DESIGN PARAMETERS AND HAZARD EVALUATION

7.1 Seismic Acceleration and Soil Profiles

For engineering design, the WSDOT GDM recommends that the peak ground acceleration (PGA) and other seismic ground motions be obtained from the 2014 U.S. Geological Survey (USGS) Seismic Hazard Maps for the Pacific Northwest Region. The Seismic Site Class was developed based on the recommended procedure, using SPT N-values from our explorations, in the 2017 AASHTO LRFD Bridge Design Specifications.

Our evaluation, based on the subsurface conditions described in Section 6.0, indicates that the site is classified as Class E. Site Class E corresponds to stiff soils with an average shear wave velocity less than 600 feet per second (fps), or an average SPT blow count less than 15 blows per foot in the upper 100 feet of soil. The recommended ground motion parameters corresponding to seven percent probability of exceedance in 75 years (1,000 years) are given in Table 4.

TABLE 4
RECOMMENDED SEISMIC DESIGN GROUND MOTIONS

Seismic Parameter	1,000-year return period "No Collapse" Criteria
Site Class	E
Rock Peak Ground Acceleration, PGA_{rock}	0.26g
Short Period Acceleration, S_s	0.60g
Long-Period Acceleration, S_1	0.23g
Zero-Period Site Factor, F_{pga}	1.41
Short-Period Site Factor, F_a	1.49
Long-Period Site Factor, F_v	3.06
Peak Design Acceleration Coefficient, A_s	0.36g
Short Period Design Acceleration, S_{DS}	0.90g
Long Period Design Acceleration, S_{D1}	0.71g

Notes:
1) g = gravity acceleration
2) Spectral values calculated assuming 5 percent structural damping

WSDOT GDM requires that all bridges be designed for 1,000-year return period ground motions under "No Collapse" criteria. Under this level of shaking, the bridge, bridge foundation, approach structures, and approach fills within 100 feet of the bridge must be able to withstand the forces and displacements without collapse of any portion of the structure.

7.2 Seismic Hazards Evaluation

Seismic hazards considered in the evaluation include ground shaking, liquefaction and associated effects (e.g., flow failure, lateral spreading, and settlement), slope instability, fault rupture, tsunami, and seiche. The following sections include a discussion of the relevant seismic hazards present at the project site. The primary hazards at this site are ground shaking, liquefaction, and liquefaction-related effects. In our opinion, the potential for fault rupture is low, given the large distance between the project site and the nearest potentially active fault. The risk of seismically induced tsunami and seiche is also very low at the site.

7.2.1 Liquefaction

Liquefaction is a phenomenon in which excess pore pressure of loose-to medium-dense, saturated, granular soils increases during ground shaking. The increase in excess pore pressure results in a reduction of soil shear strength and a potential quicksand-like condition. Liquefaction can result in differential ground settlement, foundation bearing capacity failure, lateral spreading, and flow failure.

Soil behavior under seismic loading is the primary factor in determining the susceptibility of a soil to liquefaction. Important factors in evaluating soil behavior are relative density, the fines content (percent of soil by weight smaller than 0.075 millimeter, passing the No. 200 sieve), and the plasticity characteristics of the fines.

Relative density is estimated based on SPT values. We used the provided grain size analyses and Atterberg limits test results to evaluate the index parameters of the soils at the site. Our liquefaction potential assessment for cohesive soils was performed using the recommendations presented in Boulanger and Idriss (2006). Boulanger and Idriss (2006) provided recommendations that fine-grained soils with plasticity indices greater than seven (7) would be susceptible to cyclic softening instead of liquefaction.

The Alluvium consisting of very soft/very loose to stiff/medium dense silt with varying amounts of sand, very loose to medium dense silty sand, sand and gravel with sand was encountered below the project site. The Alluvium is susceptible to liquefaction below the groundwater at approximate elevation 12.5 feet to a depth of approximate elevation -60 feet, approximately 80 feet bgs.

The Alluvium consisting of dense sand is not susceptible to liquefaction. According to Section 6.1.2.3 of Washington State Depertment of Transportation, 2015 Geotechnical Design Manual, the maximum liquefaction depth below ground surface should be limited to to 80 feet.

7.2.2 Liquefied Soil Residual Strength

We estimated the shear strength of the liquefied soil using methods recommended in the WSDOT GDM. These methods include Idriss and Boulanger (2007), Olson and Stark (2002), and Kramer (2008). These methods base the liquefied soil shear strength on $(N_1)_{60}$ or $(N_1)_{60\text{-cs}}$ values.

For our analysis, we estimated the residual shear strength by taking the average of the residual shear strengths determined using the three recommended methods. Our analysis indicates that the residual shear strength of the liquefied Alluvium may be characterized by residual friction angle ranging between 6 and 20 degrees.

7.2.3 Liquefaction-Induced Settlement

Settlement primarily occurs in the liquefiable soils. The settlement is related to densification and rearrangement of particles during ground shaking, as well as volume change as the excess pore pressure dissipates after ground shaking. Seismic ground settlement may not occur uniformly over an area, and differential settlement could impact the proposed structures supported by liquefied soil. Consequently, damage to the bridge approach embankments (pavement failures and embankment deformations) may occur, as a result of settlement.

Liquefaction-induced settlement magnitude was estimated using the methods presented in Tokimatsu and Seed (1987), and Ishihara and Yoshimine (1992). Our analyses indicate post-seismic settlements to range between 3 and 16 inches at the bridge and approach embankment locations. The differential settlement may range between 50 to 100 percent of total settlement.

WSDOT requires the bridge and approach embankments within 100 feet of the bridge be designed for "non-collapse criteria." WSDOT does not have post-seismic deformation criteria for the approach embankments. Discussions of deformation for the approach embankments are provided in a later section. In addition, the post-seismic settlement will develop negative skin friction (downdrag) along the proposed deep foundation elements supporting the bridge piers, which is addressed in a later section of this report.

7.2.4 Lateral Spreading and Flow Failure

Lateral spreading and flow failure (post-seismic slope instability) were considered as seismic impacts on the proposed bridge abutments and approach embankments. Potential lateral spreading and flow failure of the site ground surface towards the river is low because the site is in general leveled with fill in the vicinity of the golf course and levees along the river and located more than 1,000 feet from the river.

However, there is potential for flow failure and slope instability of the existing railroad embankment impacting the bridge piers 3 and 4. Based on our slope stability analysis, the bridge piers should be placed at least 20 feet away from the railroad embankment to minimize negative impacts to the East bridge abutment foundation from the railroad embankment failure.

8.0 PRELIMINARY ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

8.1 General

Design recommendations are based on the 30 percent plans, the additional information provided by HDR, and our field explorations. Geotechnical design recommendations are provided for the proposed bridge and the approach embankments and the retaining walls. Also, preliminary key construction considerations were developed associated with the geotechnical design recommendations for the bridge and the approach embankments and the retaining walls. Key geotechnical design concerns include static and seismic settlements, static and seismic stabilities, and potential downdrag on bridge foundations.

The recommendations for the bridge, approach embankments, and retaining walls are included in the following sections. If structure types, configurations, and locations change after this report, Shannon & Wilson should be contacted to provide updated recommendations.

We understand that the bridge foundations, approach embankments, and retaining walls will be designed considering the following manuals and specifications:

- Washington State Department of Transportation, Geotechnical Design Manual (WSDOT GDM), May 2015;
- Washington State Department of Transportation Design Manual (WSDOT DM), July 2018;
- Washington State Department of Transportation Bridge Design Manual (LRFD) (WSDOT BDM), June 2018;
- Washington State Department of Transportation Standard Specifications for Road, Bridge, and Municipal Construction (WSDOT SSRBMC), 2018;
- AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 (AASHTO); and
- Applicable FHWA geotechnical guidelines.

8.2 Retaining Wall Alternatives

Retaining walls are proposed to retain both approach embankments. The selection of an appropriate retaining wall system is dependent upon several factors, including tolerance to total and differential settlement, and construction considerations. Based on the explored subsurface conditions and the fill heights, we considered a Mechanically Stabilized Earth (MSE) wall, a soldier pile wall, a cast-in-place (CIP) concrete rigid gravity wall, an MSE wall support on stone column, and an MSE wall with Geofoam. A comparison of these three types of walls is presented in Table 5.

TABLE 5
COMPARISON OF RETAINING WALL ALTERNATIVES

Wall Type	Advantages	Disadvantages
Mechanically Stabilized Earth (MSE) with Conventional Fills	• Conventional construction. • Low mobilization/demobilization costs. • Can be constructed as a two-stage system to accommodate settlement (up to 12 inches, per WSDOT GDM).	• Restricts access to any existing utilities beneath reinforced zone. • Restricts future installation of facilities within or beneath reinforced zone. • Potential significant downdrag forces on bridge foundation due to settlement of conventional embankment fills. • Post-seismic global instability. • Large post-seismic settlement. • Large static settlement and therefore, two-stage construction will be required. • Significant deformation post-seismic condition.
CIP Concrete Rigid Gravity with Conventional Fills	• Conventional construction for wall. • Low mobilization/demobilization costs. • Local contractor may be available to construct this type of wall.	• Maximum tolerable settlement = 1 to 2.5 inches, per WSDOT GDM. • Will require enlarged footings to achieve bearing resistance. • Potential significant downdrag forces on bridge foundation due to settlement of conventional embankment fills. • Post-seismic global instability. • Large static and post-seismic settlement. • Significant deformation post-seismic condition. • Relatively more expensive than MSE wall.
Cantilever Soldier Pile with Conventional Fills	• Settlement-tolerant if embedded in dense sand alluvium below 80 feet.	• Vibrations generated by driving temporary casing may impact adjacent structures. • Soldier pile construction generates surface spoils. • Potential significant down drag forces on soldier piles and bridge foundation due to settlement of conventional embankment fills. • Large static and post-seismic settlement. • Significant deformation post-seismic condition. • Relatively more expensive to install piles, especially drilled-in, below 80 feet into dense sand alluvium. • Relatively more expensive than MSE wall and CIP wall.

Wall Type	Advantages	Disadvantages
Mechanically Stabilized Earth (MSE) and Approach Embankments Backfilled with Geofoam Fills	• Conventional construction for wall. • Low mobilization/demobilization costs for wall. • Local contractor may be available to construct this type of wall. • Reduce static and post-seismic settlement comparing to MSE, CIP, and Soldier pile wall with conventional fills. • Reduce downdrag forces on bridge foundation resulting from static and seismic settlement of embankment fills. • Post-seismic global stability.	• Restricts access to any existing utilities beneath reinforced zone. • Restricts future installation of facilities within or beneath reinforced zone. • Geofoam will require tie-down in flood plain area to reduce buoyancy. • Geofoam requires onsite storage location. • Geofoam susceptible to hydrocarbon degradation – needs to be encapsulated with a resistant membrane. • Large deformation post-seismic condition. • May require specialty contractor to install geofoam. • Relatively more expensive than MSE, CIP, and Soldier pile wall with conventional fills.
Mechanically Stabilized Earth (MSE) with Conventional Fills on Stone Columns to a depth of 40 feet	• Conventional construction for wall. • Low mobilization/demobilization costs for wall. • Local contractor may be available to construct this type of wall. • Reduce static and post-seismic settlement • Reduce downdrag forces on bridge foundation resulting from static and seismic settlement of embankment fills. • Post-seismic global stability.	• Restricts access to any existing utilities beneath reinforced zone. • Restricts future installation of facilities within or beneath reinforced zone. • Require specialty contractor to install stone column. • Ground improvement construction generates surface spoils. • Relatively more expensive than MSE, CIP, and Soldier pile wall with conventional fills and MSE with Geofoam.

Based upon the comparisons summarized in Table 5 and subsurface conditions, Mechanically Stabilized Earth (MSE) walls constructed with conventional fills on stone columns extending to a depth of approximately 40 feet, elevation -20 feet were selected as the preferred retaining wall type for all approach fills. In general, a stone column system is installed first to support the MSE wall. A MSE wall is internally stabilized by layers of steel or geogrid reinforcement and externally stabilized through gravity. The walls are constructed by placing lifts of compacted granular material between the reinforcement layers. The facing may be structural or purely aesthetic, depending on the wall type.

8.3 MSE Retaining Walls

8.3.1 General

MSE retaining walls are proposed to retain both approach embankments at the bridge piers 1 and 4. We understand that the proposed retaining walls will have a maximum exposed height of approximately 30 feet and taper to ground surface. The proposed length of the retaining wall is approximately 350 feet. The proposed MSE walls are a single wall system . For design purposes, we have assumed that subdrainage systems will be installed to prevent hydrostatic pressure from developing behind all retaining walls. Also, we assumed that the backfill behind the walls is level.

At the time of this report, we recommend that MSE walls with conventional fills on stone column improvements be used to construct the approach fills. Conceptual recommendations for stone column improvements are provided in Section 8.7.

8.3.2 MSE Wall Design Soil Parameters

As recommended by AASHTO LRFD, MSE wall minimum soil reinforcement length should be 70 percent of the wall height (0.7H) as measured from the leveling pad, or 8 feet, whichever is greater. The MSE wall should be constructed in accordance with WSDOT SSRBMC Division 6-13 Structural Earth Walls. MSE wall reinforced zone should meet the requirements provided in Division 9-03.14(4) Gravel Borrow for Structural Earth Wall. Embankment fill placed behind the reinforced zone should meet the specifications provided in WSDOT SSRBMC Division 2-03.3(14) – Rock Embankment. The estimated soil parameters are presented in Table 6.

TABLE 6
MSE WALL GEOTECHNICAL DESIGN PARAMETERS

Material Type:	MSE Reinforced Zone (Gravel Borrow for Structural Earth Wall)	Retained Fill (Rock Embankment)	Foundation Soil (Stone Columns)
Unit Weight (pcf)	130	130	120
Internal Friction Angle (degrees)	34	34	34
Cohesion (psf)	3000	0	0

MSE wall lateral pressures should be calculated using soil parameters of retained fill provided in Table 6. If Gravel Borrow for Structural Earth Wall is used as Retained Fill material, then the MSE Granular Backfill design parameters provided in Table 5 (above) should be used to calculate lateral earth pressures on retaining walls.

The MSE walls should be embedded in accordance with Section 15.4.5 of the WSDOT GDM, with a minimum embedment of 2 feet at the face of the wall.

8.3.3 MSE Wall Lateral Resistance

Lateral resistance to lateral movement for an MSE wall consists of sliding friction and passive resistance. We recommend that passive earth pressure be neglected when calculating the lateral resistance because potential soil disturbance or loss in front of the wall, and future excavation in front of the wall. The nominal friction resistance for sliding can be expressed as the vertical load (on the footing) multiplied by a coefficient of 0.6 for MSE-reinforced soil mass on an approved subgrade. A resistance factor of 1.0 should be used in calculation of friction sliding resistance.

8.3.4 MSE Wall Bearing Resistance

The bearing resistance analysis was performed in accordance with the WSDOT GDM and AASHTO LRFD. The factored bearing resistance analysis was based on the assumption that the MSE walls will be supported on stone columns. Figure 5, Factored Bearing Resistance versus MSE Reinforcement Length, presents the nominal bearing resistance versus effective footing width curves under strength limit state and extreme event limit state.

We assume MSE walls will be constructed as a part of new approach fills, and all approach fill settlements will be allowed to occur prior to installation of permanent MSE wall facing. Therefore, the service limit state bearing resistance was not estimated. We provided static and post-seismic bearing resistance with stone coulumn improvement. The bearing resistance of the wall was evaluated as a rectangular foundation with a length to width (L/B) ratio of 10, where the width is the width of the reinforced backfill. A resistance factor of 0.65 should be used for the strength limit state, and a factor of 0.9 should be used for the extreme event limit state designs.

8.3.5 MSE Wall Static Settlement with Stone Column Improvements

Settlement will result from the construction of the proposed MSE walls and placement of approach fills. We recommend that the stone comlumn be installed to a depth of approximately 40 feet, approximately elevation -20 feet. We estimate that the static settlement within stone column improvement zone is approximately three inches, and static settlement of the unimproved soils below the stone column zone is approximately three inches for a total static settlement of six inches. We assume permanent wall facing will not be installed, until settlement is complete. According to the WSDOT GDM, MSE walls constructed with flexible facing will tolerate up to six inches of settlement.

Due to settlement, we recommend the MSE walls be constructed in two stages. The first phase would consist of constructing the wall with flexible facing, welded wire facing, and would allow the wall to settle. The second phase would consist of installing permanent fascia consisting of precast concrete panels or cast-in-place panels after settlement is completed. A settlement monitoring program is recommended to be implemented during construction to monitor static settlement. Recommendations for settlement monitoring is provided in Section 11.3.2.

8.3.6 MSE Wall Drainage

Suitable drainage for walls can be provided by granular backfill material and a wall base subdrain system consisting of a 6-inch-diameter perforated or slotted drain pipe wrapped in an envelope of filter material at least 12 inches thick and confined by a separation geotextile. The filter material should meet the requirements for Gravel Backfill for Drains specified in Division 9-03.12(4) of the WSDOT SSRBMC.

The separation geotextile fabric should meet the requirements for Geotextile for Underground Drainage Filtration Property specified in Table 2 in the Division 9-3.2(1) of the WSDOT SSRBMC. The subdrain should be above the typical groundwater level, convey any collected seepage to the end of the wall, and daylight at low spots below the wall elevation. In addition, the subdrain should be daylight to face of wall or tie-in to drainage system every 300 feet.

8.3.7 Global Stability Analysis

We conducted global stability analyses for the existing MSE walls using the computer program SLOPE/W, Version 9.1 (Geo-Slope International, 2018R2). The Morgenstern-Price slope stability analysis method was used for irregular surface failure mechanisms. The analyses was performed for static, seismic, and post-seismic conditions. A live load of 250 psf was assumed for the static condition. For seismic slope stability analyses, pseudo-static and post-seismic procedures described in the WSDOT GDM Chapter 6 were followed. Horizontal acceleration coefficients equal to one-half of the site-adjusted peak ground acceleration (0.5 x

F_{pga} x PGA) were used. For our seismic slope stability analyses, we used horizontal seismic coefficients equal to 0.225 for the 1,000- year ground motion level.

The WSDOT GDM requires that highway retaining walls be designed with a maximum resistance factor for global stability of 0.65 (equivalent to a FS of 1.5) for the static conditions. For seismic and post-seismic analyses, a maximum resistance factor of 0.9, or an FS of 1.1, is required.

We developed two cross sections in longitudinal directions of the approach embankments for global stability analysis, based on the existing MSE wall information and anticipated subsurface conditions. We assumed the wall width is 0.7H. We evaluated global stability analyses for conventional fills, geofoam fills for MSE walls and approach embankments, and stone coumlun improvement extending to a depth of 40 feet in our analyses. Based on our analyses, the East and the West walls and embankments satisfy the minimum global stability FS requirements for static and seismic conditions. However, the East and West walls and embankments do not satisfy the minimum global stability FS requirements for post-seismic condition. Geofoam fills and stone column improvement were modelled into these cross sections; as a result the East and West walls and embankments satisfy the minimum global stability FS requirements for post-seismic condition. The results of our global stability analyses for the walls and approach embankment are presented in Appendix D and are summarized in Table 7.

TABLE 7
GLOBAL STABILITY ANALYSIS RESULTS FOR MSE WALLS

MSE Wall Locations	Factor of Safety				
	Static	Seismic	Post-Seismic	Post-Seismic and Stone Column Improvement	Post-Seismic and Geofoam Fills
Wall 1, Pier 1 (longitudinal direction)	1.6	1.2	0.3	1.5	5.5
Wall 2, Pier 4 (longitudinal direction)	1.6	1.2	0.3	1.4	5.6
Railway Track Embankment	N/E[1]	N/E[1]	0.5	N/A[2]	N/A[2]

Notes:
1) Not consider in the design to evaluate for static and seismic conditions.
2) Not applicable to perform improvements.

8.4 Approach Embankments

We understand that the proposed embankments may be up to 30 feet in height. We recommend that the embankment be constructed 2 horizontal (H):1 vertical (V) or flatter side slopes. Proposed roadway embankment fills will be subject to static and post-seismic settlement. Discussion of the settlement analysis and results is included in.

8.5 Bridge Foundation Alternatives

The selection of an appropriate foundation system for the proposed bridge structure is dependent upon several factors, including foundation capacities, tolerance to total and differential settlement resulting from static loads, and construction considerations. Based on the explored subsurface conditions, we considered driven pile, drilled shaft, and spread footing foundations. Bridge foundation loads are not available at this time. A comparison of these three types of foundations is presented in Table 8.

TABLE 8
COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES

Foundation	Description	Advantages	Disadvantages
Driven Steel Closed-End Pipe Piles (16- to 24-inch diameter)	Piles driven into dense to sand alluvium with a nominal compressive resistance on the order of 80 to 150 tons	▪ Generates relatively high bearing resistance. ▪ Relatively fast construction. ▪ Feasible for staged construction. ▪ Lower cost than drilled-in foundations.	▪ Relatively low lateral resistance; ▪ Significant risk of damaging existing adjacent structures due to vibrations generated by pile driving. ▪ Vibration may cause additional settlement of the adjacent structures. ▪ Noise impact to neighbor residents and commercial.
Small- to Medium-Diameter Drilled Shafts (2- to 6-foot diameter)	Multiple small- to medium-diameter shafts located at each pier, nominal compressive resistance on the order of 400 to 1,500 tons	▪ Higher level of control of construction variability compared to driven piles. ▪ Feasible for staged construction. ▪ Can be constructed using non-vibratory methods.	▪ Relatively more expensive than driven piles. ▪ Require a specialty contractor. ▪ Higher construction QA/QC requirements. ▪ Relatively longer construction duration compared to driven piles. ▪ High mobilization/demobilization costs compared to other foundation types.
Spread Footings	Spread footings founded on loose to medium sand alluvium, nominal bearing resistance of about 5 ksf	▪ Conventional construction. ▪ Least expensive mobilization /demobilization costs.	▪ Significant to static and post-seismic settlement. ▪ Significant deformation post-seismic condition. ▪ Will require significant enlarged footings to achieve bearing resistance. ▪ May require dewatering and shoring for the footing excavation. ▪ Post-seismic instability.

Based upon the comparisons summarized in Table 8 and subsurface conditions, drilled shafts were selected as the recommended foundation type for bridge foundations.

8.6 Drilled Shaft Foundations

8.6.1 General

As discussed in section 8.5, a drilled shaft foundation system was selected as the preferred foundation system for the proposed piers 1 to 4 (abutment and interior pier locations. The following sections provide our recommendations for axial and lateral resistance of 2- to 6-foot-diameter drilled shafts at the proposedpier locations (1 and 4) and the lateral resistance of 6- and 7-foot-diameter drilled shafts at the proposed interior pier locations (2 and 3) . We also understand that the drilled shafts will be located outside and in front of the MSE walls retaining approach embankments, but within the stone column improvement zone.

We designed the drilled shafts to resist axial loads by both side and end resistances; therefore, we recommend that the drilled shafts be constructed using fully temporary cased excavations. In addition, there is a potential for cave-in of loose sand under groundwater at the drilled shaft locations. Further, due to concerns over the potential impact of construction vibration on the MSE retaining walls, settlement of loose sand, and the adjacent commercial and residential structures on the east abutment; we recommend that temporary casing should be installed using a non-vibratory drilling method, such as rotary or oscillator method. Boring B-2 was used to estimate the soil properties for foundation design at piers 1 and 2. Boring B-1 was used to estimate the soil properties for foundation design at piers 3 and 4. Additional geotechnical explorations should be performed to characterize the soils at the north abutment.

We also recommend that the center of the drilled shafts be located at least 10 feet from the face of the MSE walls to reduce potential construction impacts on the MSE walls.

8.6.2 Drilled Shaft Axial Resistance

We performed axial resistance evaluation in general accordance with AASHTO LRFD. We evaluated axial resistance for service, strength, and extreme event limit states. The analyses were based on the subsurface conditions encountered in the project borings and our experience with similar soil and project conditions. We estimated unit side and tip resistance values based on the average SPT values (N-values) within each unit, laboratory tests, load tests in similar soil conditions from other projects, and our experience.

Our axial resistance analyses results for axial resistance of 2- to 6-foot-diameter drilled shafts for piers 1 and 4 are presented in Figures E1 through E10 in Appendix E and axial resistance of 7-foot-diameter drilled shaft for piers 2 and 3 and in Figures E11 and E12. These results are presented as plots of nominal and factored axial resistance versus depth for service, strength, and extreme event limit states. Recommended resistance factors for each limit state are provided in the notes section of each figure.

Recommended resistance factor values could be increased if a load test program is implemented for the project. Estimated drilled shaft lengths and tip elevations (based on the provided factored design loads per foundation element at each pier) are summarized in Table 9.

TABLE 9
ESTIMATED DRILLED SHAFT LENGTH AND TIP ELEVATION

Pier Location	Shaft Diameter (feet)	Estimated Shaft Length[2] (Strengh) (feet)	Estimated Shaft Length (Service) (feet)	Estimated Shaft Length (feet)	Estimated Shaft Tip Elevation (feet)	Factored Axial Compressive Resistance Per Shaft (kips)		
						Strength	Service[3]	Extreme[4] Event
1	2	NA	81	NA	NA	1000	700	N/P
	3	99	60	99	-76	1000	700	N/P
	4	83	48	83	-60	1000	700	N/P
	5	69	36	69	-46	1000	700	N/P
	6	55	26	55	-32	1000	700	N/P
2 and 3	6	120	83	120	-100	2700	1900	N/A
	7	107	77	107	-87	2700	1900	N/A
4	2	126	80	126	-106	1000	700	N/P
	3	99	60	99	-79	1000	700	N/P
	4	83	48	83	-63	1000	700	N/P
	5	69	36	69	-49	1000	700	N/P
	6	55	26	55	-35	1000	700	N/P

Notes:
1) The ground surface is assumed at elevation 20 feet for the pier 4 23 feet for the pier 1, and 20 feet for piers 2 and 3.
2) Estimated shaft length is taken as the distance between the ground surface and estimated shaft tip elevation.
3) Service Resistance based on 1.0-inch allowable settlement.
4) Not provided.

8.6.3 Shaft Static and Post-Seismic Downdrag Load

To reduce potential static on downdrag forces on the bridge foundation, we recommend the MSE walls and approach embankments be installed first and allow static settlement to cease, and then install the drilled shafts.

There is potential of post-seismic downdrag forces acting on brigde foundations due to post-seimic settlement of unimproved soils below the stone column improvement zone. Seismic downdrag force are for each shaft diameter is included in Figures E1 through E12 in Appendix E.

8.6.4 Axial Group Efficiency

The estimated nominal axial resistance assumes the shafts are oriented in a single row and spaced at least three shaft diameters apart (2.5D), measured center-to-center. Based on this assumption, the shaft group effects are not considered. If, during final design, the shaft spacing is changed, the appropriate shaft efficiency factor must be established and applied, as recommended by the AASHTO LRFD and described above.

8.6.5 Bridge Foundation Setback from BNSF Rail Embankment

As metioned in Section 7.4.2, There is potential for BNSF rail embankment slope instability and flow failure towards the east and west bridge abutment foundations. We recommend that the east and west bridge abutment foundations should be placed at least 20 feet away from the railroad embankment to minimize negative impacts to the bridge abutment foundations from the railroad embankment failure. The slope stability analysis results are shown on Figure D11 in Appendix D. If bridge piers are located within the 20 feet setback, the bridge deep foundation would need to be designed to accommodate additional lateral loads from slope instability of the railroad embankment. The additional lateral loads will be provided in the final design once the bridge pier locations are determined.

8.6.6 Lateral Resistance

The bridge foundations will be subjected to lateral loads resulting from live and seismic loading. We understand that the laterally loaded shaft analyses will be performed with the aid of the computer program LPILE developed by Ensoft, Inc. Geotechnical input parameters for the LPILE computer model are provided in Tables 10 and 11 for static and post-seimic conditions, respectively. For design, we have assumed that the groundwater level is at elevation 13 feet.

TABLE 10
STATIC LPILE GEOTECHNICAL INPUT PARAMETERS FOR BRIDGE FOUNDATIONS

Pier	Elevation		Soil Type (p-y Curve)	Effective Unit Weight[1] (pci)[2]	Cohesion (psi)[2]	Static Friction Angle (deg)[2]	Static Modulus (pci)[2]
	Layer Top (feet)	Layer Bottom (feet)					
1	20	13	Sand	0.069	--	34	225
	13	-20	Sand	0.033	--	34	125
	-20	-30	Sand	0.039	--	36	60
	-30	-60	sand	0.033	--	36	60
	-60	-95	Sand	0.036	--	38	125
4	20	13	Sand	0.069	--	34	225
	13	-20	Sand	0.033	--	34	125
	-20	-30	Sand	0.039	--	36	60
	-30	-60	Sand	0.033	--	36	60
	-60	-120	Sand	0.036	--	38	125
2 , 3	20	13	Sand	0.066	--	32	25
	13	-10	Sand	0.030	--	32	20
	-10	-20	Sand	0.030	--	34	20
	-20	-30	Sand	0.039	--	36	60
	-30	-60	Sand	0.033	--	36	60
	-60	-120	Sand	0.036	--	38	125

Notes:

1) Effective unit weight = Total unit weight – Unit weight of water (62.4 pcf = 0.036 pci)
2) deg = degrees, pci = pounds per cubic inch, and psi = pounds per square inch.

TABLE 11
POST-SEISMIC LPILE GEOTECHNICAL INPUT PARAMETERS FOR BRIDGE FOUNDATIONS

Pier	Elevation		Soil Type (p-y Curve)	Effective Unit Weight[1] (pci)[2]	Cohesion (psi)[2]	Post-Seismic Friction Angle (deg)[2]	Post-Seismic Modulus (pci)[2]
	Layer Top (feet)	Layer Bottom (feet)					
1	23	13	Sand	0.069	--	34	225
	13	-17	Sand	0.033	--	34	125
	-17	-40	Sand	0.033	--	20	20
	-40	-95	Sand	0.036	--	36	125
4	20	13	Sand	0.069	--	34	225
	13	-20	Sand	?	--	34	125
	-20	-60	Sand	0.033	--	20	20
	-60	-120	Sand	0.036	--	38	125
2	20	13	Sand	0.066	--	6	25
	13	-10	Sand	0.030	--	6	20
	-10	-40	Sand	0.033	--	20	20
	-40	-80	Sand	0.036	--	38	125
3	20	13	Sand	0.066	--	6	25
	13	0	Sand	0.030	--	6	20
	0	-20	Sand	0.030	--	14	20
	-20	-60	Sand	0.033	--	20	20
	-60	-80	Sand	0.036	--	38	125

Notes:

1) Effective unit weight = Total unit weight – Unit weight of water (62.4 pcf = 0.036 pci)
2) deg = degrees, pci = pounds per cubic inch, and psi = pounds per square inch.

The post-seismic Lpile input parameters presented in Table 11 are for liquefiable soil conditions, and they will be improved with stone column improvement. The Lpile input parameters with stone column improvement will be provided during final design when stone column design is completed.

8.6.7 Lateral Group Efficiency

The estimated lateral resistance parameters presented in Tables 9 and 10 are recommended for drilled shafts in a single row with center-to-center spacing greater than five shaft diameters (5D). For shaft spacing less than 5D, the appropriate P-Multiplier must be established and applied. If, during final design, the shaft spacing is changed, the appropriate shaft efficiency factor must be established and applied, as recommended by the AASHTO LRFD and described above.

8.6.8 Drilled Shaft Construction Considerations

8.6.8.1 General

The drilled shaft installation procedures should follow the WSDOT SSRBMC, Division 6-19 Shafts, with appropriate project-specific provisions. The selection of equipment and procedures for constructing drilled shafts should consider shaft diameter and length and subsurface conditions. The design and performance of drilled shafts can be significantly influenced by the equipment and construction procedures used to install the shafts.

Generally, drilled shafts are constructed by excavating a cylindrical bore to the prescribed embedment with a large-diameter auger or other drilling tool. Temporary or permanent casing is often used, depending on site conditions. Upon completion of drilling and inspection of the shaft, a steel rebar cage is placed, and concrete is pumped into the hole to complete the drilled shaft.

In our opinion, due to the possibility of instability drilling in loose sand under groundwater, we recommend that the drilled shafts be constructed using fully-cased excavations using a non-vibratory and non-driving method. The drilled shafts should be constructed in the wet, if groundwater is encountered during drilling. The temporary casing should be advanced ahead of the auger.

Further, due to concerns over the potential impact of construction vibration on the adjacent structures, we recommend that the temporary casing be installed using a non-vibratory method. Due to the potential hydrostatic imbalances, drilling slurry may be required to avoid soil loss around the casing. Drilled shaft contractors who participate on this project should be required to demonstrate that they have suitable equipment for this project and adequate experience in the construction of shafts with similar subsurface conditions.

8.6.8.2 Potential Obstructions

Although fill material debris and boulders, etc., were not encountered in the borings, there is potential that fill material debris may be encountered in localized fill areas that may cause obstruction.

8.6.8.3 Shaft Quality Control

We recommend full-time observation of the drilled shafts by a qualified representative from our firm to observe the contractor’s means, methods, and equipment; and to assist the drilled-shaft inspector with an understanding of the critical issues for drilled shaft construction. In addition, the design geotechnical engineer and structural engineer should make periodic visits. We recommend either Thermal Integrity Testing or cross-hole sonic log (CSL). Shaft testing and access tubes should be installed and performed in accordance with WSDOT SSRBMC, Division 6-19.3(6) for nondestructive quality assurance testing and the project special provisions.

8.7 Stone Column Ground Improvement Conceptual Design

The stone column ground improvement was evaluated according to FHWA stone column design guidelines including FHWA Ground Improvement Technical Summaries (FHWA-SA-98-086). The Stone Column treatment area is approximately within 100 feet of the proposed bridge.. We recommend that the stone column treatment area be extended at least 20 feet outside the MSE wall and approach embankment footprints. The stone column treatment depth will extend to 40 feet depth below existing ground surface at approximate elevation of 23 feet in the west approach area and 20 feet in the east approach. The stone column diameter is approximately 3.5 feet and maximum spacing is approximately eight feet. Minimum area replacement ratio should be greater than 15 percent.

Post Improvement Static Settlement: Based upon the above conceptual design of the stone column ground improvement, the ground improvement should reduce the estimated unimproved ground settlement by approximately 30 percent: from about 10 inches unimproved to six inches improved near bridge abutments (piers 1 and 4). Differential settlement beneath the MSE walls should then be less than one percent over most practical distances.

Post Improvement Seismic Settlement: The magnitude of seismic induced settlement for the improved zone is dependent on the column spacing and diameter. Closer spacing and larger column diameter will result in smaller post improvement settlement and increase the cost. Based upon our current conceptual design, the stone column improvement may result in post improvement seismic settlement of three inches below the improvement zone. This estimated settlement could manifest itself as differential settlement localized between adjacent stone columns or might occur over large areas within the improvement zone. This magnitude of differential settlement mostly will meet "no collapse" requirement by WSDOT.

9.0 DESIGN RECOMMENDATIONS FOR BRIDGE OVERCROSSING WETLAND

9.1 General

As discussed in Section 7.2, the Alluvium is susceptible to liquefaction below the groundwater at approximate elevation 12.5 feet to a depth of approximate elevation -60 feet: approximately 80 feet below ground surface.

We understand that the overcrossing may consist of a bridge or culvert. Struture type has not been determined yet.

If a new single span bridge is selected, the bridge will require to be designed for seismic hazards; and a deep foundation system consisting of drilled shafts or driven pile foundation will also be required. We anticipate drilled shafts or driven piles will need to extend into non-liquefiable layer below elevation -60 feet: approximately 80 feet bgs. Additionally, post-seismic settlement will result in downdrag forces on the drilled shafts or driven piles

According to Section 10.3.1 of the American Association of State Highway and Transportation Officials Load and Resistance Factor Design, 8th Edition, 2017 (AASHTO LRFD), a box culvert and buried structures may not need to be designed for seismic conditions. Based on WSDOT BDM, seismic design will be required for buried structures with spans equal to or greater than 20 feet.

10.0 PAVEMENT DESIGN RECOMMENDATIONS

10.1 General

We understand that the new pavement for the extension of Hazel Street over BNSF rails and South Pacific Avenue to South River Road, Douglas Street, and 3rd Avenue extension to Hazel Street, will consist of Hot Mix Aspahlt (HMA). Pavement designs were performed in accordance with the recommended procedures and guidelines in the 2017 City of Kelso's Engineering Design Manual (KEDM) and Standard Plans and Specifications; 2018 WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (WSSC); and 1993 AASHTO Guide for Design of Pavement Structures (AASHTO).

The results, conclusions, and design alternative recommendations in this report are based on our understanding and synthesis of City of Kelso requirements, field data, laboratory testing, structural pavement analysis, and our engineering judgment.

We evaluated new HMA pavement for a design life of 50 years. Results from the pavement design provide a quantitative basis for evaluating the design alternatives and selecting the final pavement rehabilitation approach. The recommended pavement sections meet the minimum structural requirements. However, there may be additional project considerations, such as cost effectiveness, that may influence final selection of pavement sections. All pavement construction should be performed in accordance with the 2018 WSSC.

10.1.1 Traffic Data

Specific traffic data, including a projected growth rate of two percent, was provided by HDR/City of Kelso. The highest number of trucks, based on a total traffic count of 242 with 11 percent truck volume, was assumed for the deisgn. Truck types consisting of five-axle semi-truck trailers were assumed for design.

10.1.2 New Pavement and Subgrade Parameters for Design

We developed input values for the pavement design based primarily on correlation of resilient modulus to Dynamic Cone Penetrometer (DCP) test results. The design subgrade resilient modulus (M_r) value of 6,000 pounds per square inch (psi) corresponds to the average of all the corrected DCP tests.

10.1.3 Other AC Pavement Design Parameters

The following parameters are for the new pavement analyses:

- Standard Deviation = 0.5
- Initial Serviceability = 4.2
- Terminal Serviceability = 2.7
- Reliability = 75%
- New Asphalt Layer Coefficient = 0.42
- New Aggregate Base Layer Coefficient = 0.1
- Drainage Coefficient = 1.0 (good)

10.2 Hazel Street Pavement Recommendations

We evaluated the proposed extension of Hazel Street to South River Road for the new HMA pavement using the data, procedures, and assumptions discussed in the preceding Section 10.1. We recommend that the new pavement section consist of 6 inches of HMA and 12 inches of Crushed Rock Base Course (CSBC), assuming the required subgrade reinforcement geotextile per the 2018 KEDM.

10.3 Douglas St. and Douglas-Hazel St. Connection Pavement Recommendations

Based on traffic data provided by HDR, we recommend the asphalt and base course thickness of 4 inches and 6 inches, respectively, per Table 3.7 in the 2018 KEDM. Subgrade reinforcement geotextile is required per the 2018 KEDM.

10.4 AC Pavement Material Recommendations

We recommend HMA mix design gradation is ½-inch dense-graded HMA. The required HMA binder grade is PG 64-22 per 2018 KEDM. Asphalt should be in accordance with Section 5-04 of the WSDOT SS 2018. CSBC should be in accordance with Section 9-03.9(3) of the WSDOT SS 2018.

11.0 PRELIMINARY GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

11.1 Site Preparation and Earthwork

11.1.1 General

This section contains construction considerations including wall excavation and foundation subgrade preparation. Earthwork should be performed in accordance with WSDOT SSRBMC.

11.1.2 Site Preparation and Excavation

Site preparation will include (1) clearing, grubbing, and roadside cleanup; (2) removal of existing structures and underground utilities; and (3) subgrade preparation and excavation. These construction activities should generally be accomplished in accordance with the WSDOT SSRBMC, Division 2. If temporary shoring dewatering is needed, the design of such shoring is traditionally the responsibility of the contractor.

After site stripping and preparation activities are completed, the exposed subgrade to receive structure, pavement, and fill should be proof-rolled with a fully loaded 10- to 12-yard dump truck or similar heavy rubber-tired construction equipment to identify soft, loose, or unsuitable areas. The proof-roll should be conducted prior to fill placement.

The site stripping and proof-roll should be observed by a qualified geotechnical engineer or representative, who should determine stripping depth, evaluate the suitability of the subgrade, and identify areas of yielding. If loose and/or wet, soft soil zones are identified during proof-rolling, the soils should be removed and replaced with compacted structural fill in accordance with WSDOT SSRBMC, Division 2-09.3(1)C, Removal of Unstable Base Material.

Disturbance of subgrade soil due to construction equipment and activities could affect support of the proposed walls and embankment. The contractor should take necessary steps to protect the subgrade from becoming disturbed.

11.1.3 Temporary Cut-and-Fill Slopes

Temporary cut slopes are typically the responsibility of the contractor and should comply with applicable local, state, and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards. For general guidance, we suggest that temporary construction slopes be made at 1H:1V or flatter above groundwater. In areas of loose fills, very soft soil, or groundwater seepage, flatter slopes are likely to be required.

11.2 Approach Embankments

We understand the south side of the proposed approach embankment on the east side will be contained by a retaining wall near the bridge abutment and will transition to a conventional embankment away from the bridge, where feasible. We also understand that the proposed embankments may be up to 38 feet in height.

We recommend that embankment fill be placed behind the reinforced zone, which should meet the specifications provided in WSDOT SSRBMC Division 2-03.3(14) – Rock Embankment. We recommend that the embankment be constructed 2H:1V or flatter side slopes. Proposed roadway embankment fills will be subject to static and post-seismic settlement.

11.3 MSE Wall Construction Considerations

11.3.1 MSE Wall Leveling Pad

A leveling pad is an unreinforced concrete pad generally used to begin the facing construction, if concrete fascia panels are used; this allows a uniform, level starting point to place the fascia panels on which to build upward. The surface of the leveling pad should be smooth and horizontal, both side-to-side and front-to-back, to ensure the fascia panel courses are level.

11.3.2 MSE Wall Settlement Monitoring Program

We have recommended that field instrumentation be implemented to monitor settlement along the MSE Wall and approach embankments. The monitoring program should be maintained during and following embankment and wall construction.

In regard to wall construction, we recommend monitoring the settlement to help determine the timing of concrete wall panel installation, as well as construction of roadway pavement.

The settlement monitoring program for the wall and approach embankment should consist of the installation of a pair of settlement plates: one settlement plate along the centerline of the approach embankment, and one settlement plate under the face of wall. The settlement plate under the wall should be installed a few feet behind the face of the wall. Plate pairs should be placed every 100 feet along the length of the wall.

We recommend periodic level surveying of the plate elevation and fill surface elevation be performed two to three times a week and for two to three months following wall/embankment construction. The results of the wall survey may allow installation of concrete fascia panels and pavement sooner.

The settlement monitoring data should be provided to Shannon & Wilson for evaluation as the information is collected. Based on our review of the initial monitoring data, the schedule for reading the settlement plates may be revised.

11.4 Pavement Subgrade Preparation

11.4.1 Subgrade Preparation During Dry Weather (Summer)

For fill areas, the assumed subgrade soil for new and reconstructed pavement is compacted embankment fill composed of any combination of non-plastic to low-plasticity silts, sand, and gravel. For cut areas, the assumed subgrade soil is compacted on-site silt, gravel, and lean clay.

We recommend that the prepared subgrade be checked to identify any soft or weak spots prior to the placement of pavement material. At a minimum, the subgrade check should consist of proof-rolling the subgrade with a fully loaded dump truck. Soft or weak spots should be over-excavated and replaced with compacted aggregate base, in accordance with WSDOT SSRBMC, Division 2-09.3(1)C, Removal of Unstable Base Material.

Provisions should be made under this contract for a quantity of subgrade stabilization equivalent to 20 percent of the total area of new pavement construction, that will be performed during the summer at a depth of 24 inches.

The subgrade should be compacted to a minimum density of 95 percent of the maximum dry density as determined by ASTM D698 for the upper 12 inches of subgrade soil. Where the exposed subgrade consists of fine-grained soils, we recommend that a non-woven separation geotextile be used between the approved soil subgrade and aggregate base to separate and reduce the potential for fines to migrate into the aggregate base. The non-woven separation geotextile should meet the requirements in Table 3 in Division 9-33.2(1) Geotextile for Separation and Soil Stabilization.

11.4.1 Subgrade Preparation During Wet Weather (Winter)

We anticipate that much of the subgrade soils within cut areas are fine-grained and will be sensitive to moisture during handling and compaction. Proceeding with site earthwork operations using these soils during wet weather could add significant costs to the project. Therefore, we recommend that, if possible, site stripping, preparation, and earthwork be completed during periods of warm, dry weather when soil moisture can be controlled by aeration.

During or subsequent to wet weather, drying or compaction of the fine-grained on-site subgrade soils will be difficult or impossible. Therefore, it will be necessary to amend the on-site soils with cement, or perform subgrade stabilization to a minimum depth of 24 inches.

Subgrade stabilization should be performed in accordance with WSDOT SSRBMC, Division 2-09.3(1)C, Removal of Unstable Base Material.

Provisions should be made under this contract for a quantity of subgrade stabilization equivalent to 60 percent of the total area of new or reconstructed pavement construction that will be performed during the winter at a depth of 24 inches. A non-woven separation geotextile should be placed between the fine-grained subgrade soil and aggregate base.

Delays in site earthwork activities should be anticipated during periods of heavy rain. In addition, site clearing and stripping activities will expose fine-grained subgrades that are subject to disturbance (severe pumping and loss of equipment support) if construction traffic is allowed on the subgrade while wet conditions exist.

12.0 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and further assume that the explorations are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

If subsurface conditions different from those encountered in the explorations are encountered or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary.

If there is a substantial lapse of time between the submission of this report and the start of construction at the site, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that we review our report to determine the applicability of the conclusions and recommendations.

Within the limitations of scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

These conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as observed at the time of our explorations.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples from test borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

This report was prepared for the exclusive use of City of Kelso and HDR Engineering, Inc. in the design of the South Kelso Grade Separation Project. The data and report should be provided to the contractors for their information, but our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions included in this report.

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

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

SHANNON & WILSON, INC.

TRAVIS THUAN NGUYEN
35390

9/7/2018

Travis Nguyen, PE
Associate | Geotechnical Engineer

Risheng "Park" Piao, PE
Vice President | Geotechnical Engineer

TTN:HXS:RPP/mmm

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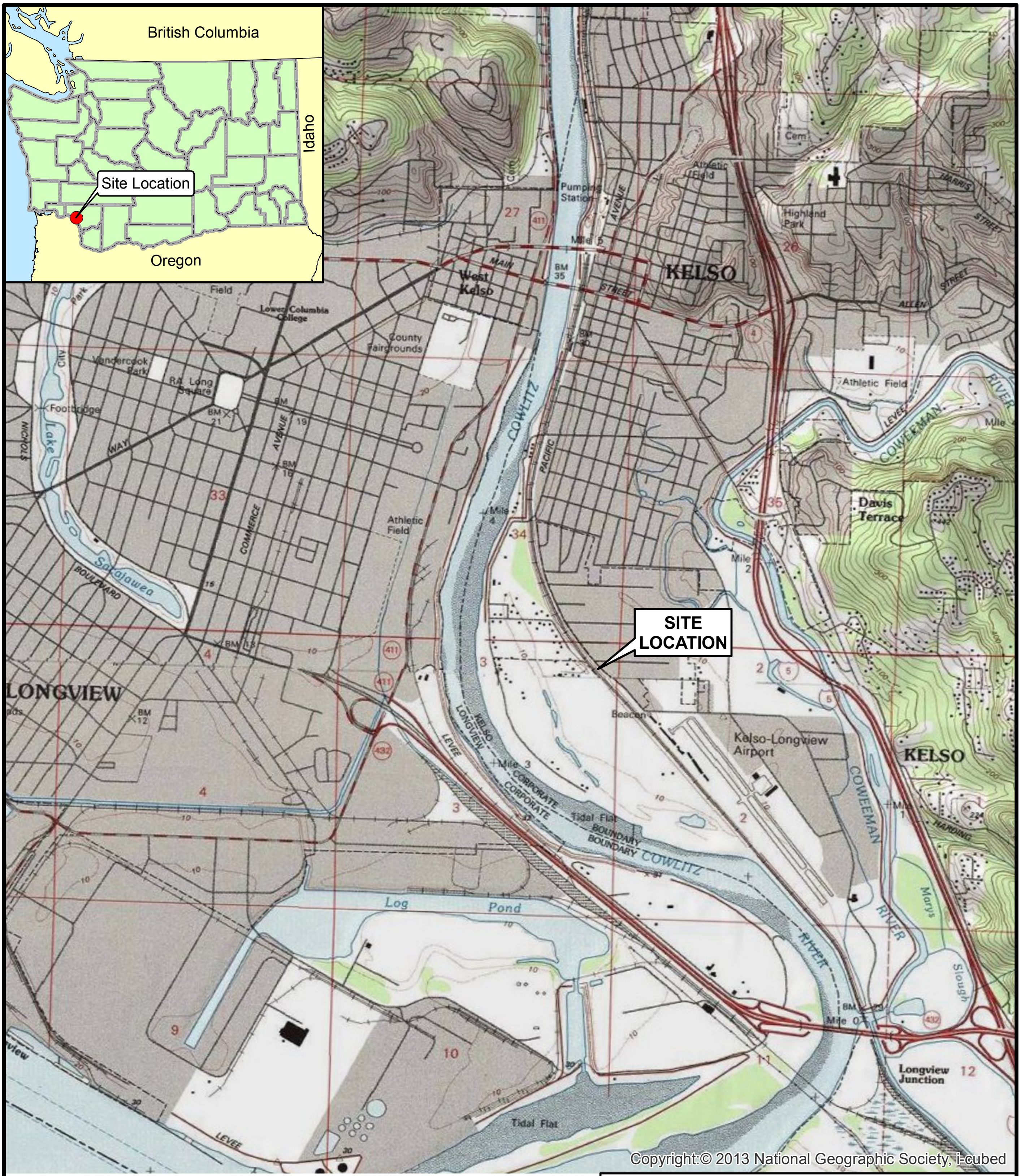


South Kelso Railroad Grade Separation
Kelso, Washington

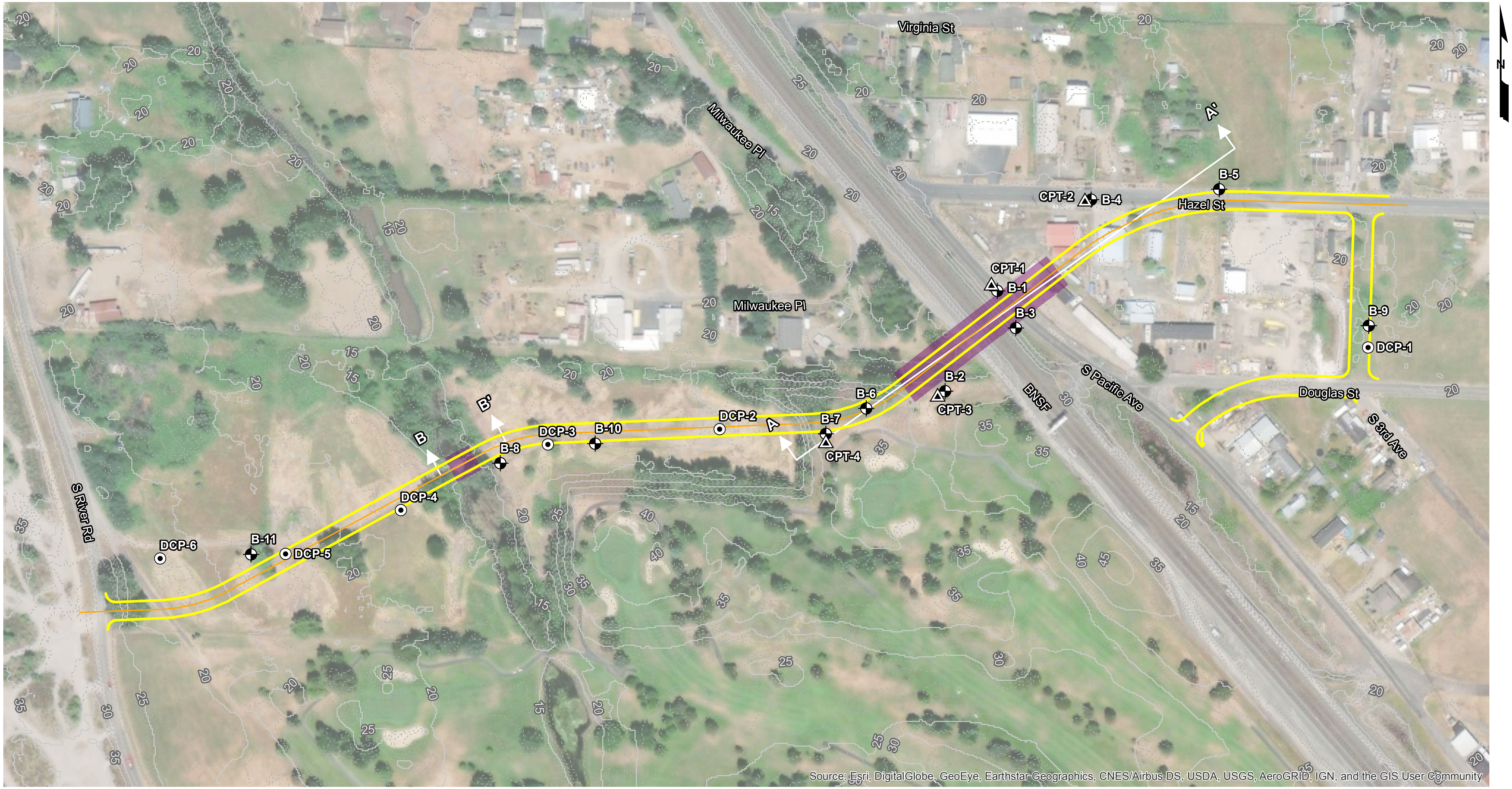
VICINITY MAP

September 2018 24-1-04201-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

Filename: T:\Projects\24-1\4201_S Kelso Grade S\Avmxd\VicinityMap.mxd Date: 5/10/2018 Login: aeh



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

LEGEND

B-1 ⊕	Designation and Approximate Location of Boring
CPT-1 △	Designation and Approximate Location of Cone Penetrometer Test (CPT)
DCP-1 ⊙	Designation and Approximate Location of Dynamic Cone Penetrometer Test (DCP)
🟨	Proposed Project Corridor
🟧	Proposed Alignment
🟪	Proposed Bridge
A A'	Location and Designation of Interpretive Subsurface Profile

0 100 200 400

Scale in Feet

NOTES

1. Contours were derived from 2010 LiDAR data and may not reflect existing current grade in all locations.
2. Proposed features adapted from file C17913x500.dwg, downloaded from ProjectWise on August 21, 2018.

South Kelso Railroad Grade Separation
Kelso, Washington

SITE AND EXPLORATION PLAN

September 2018 24-1-04201-001

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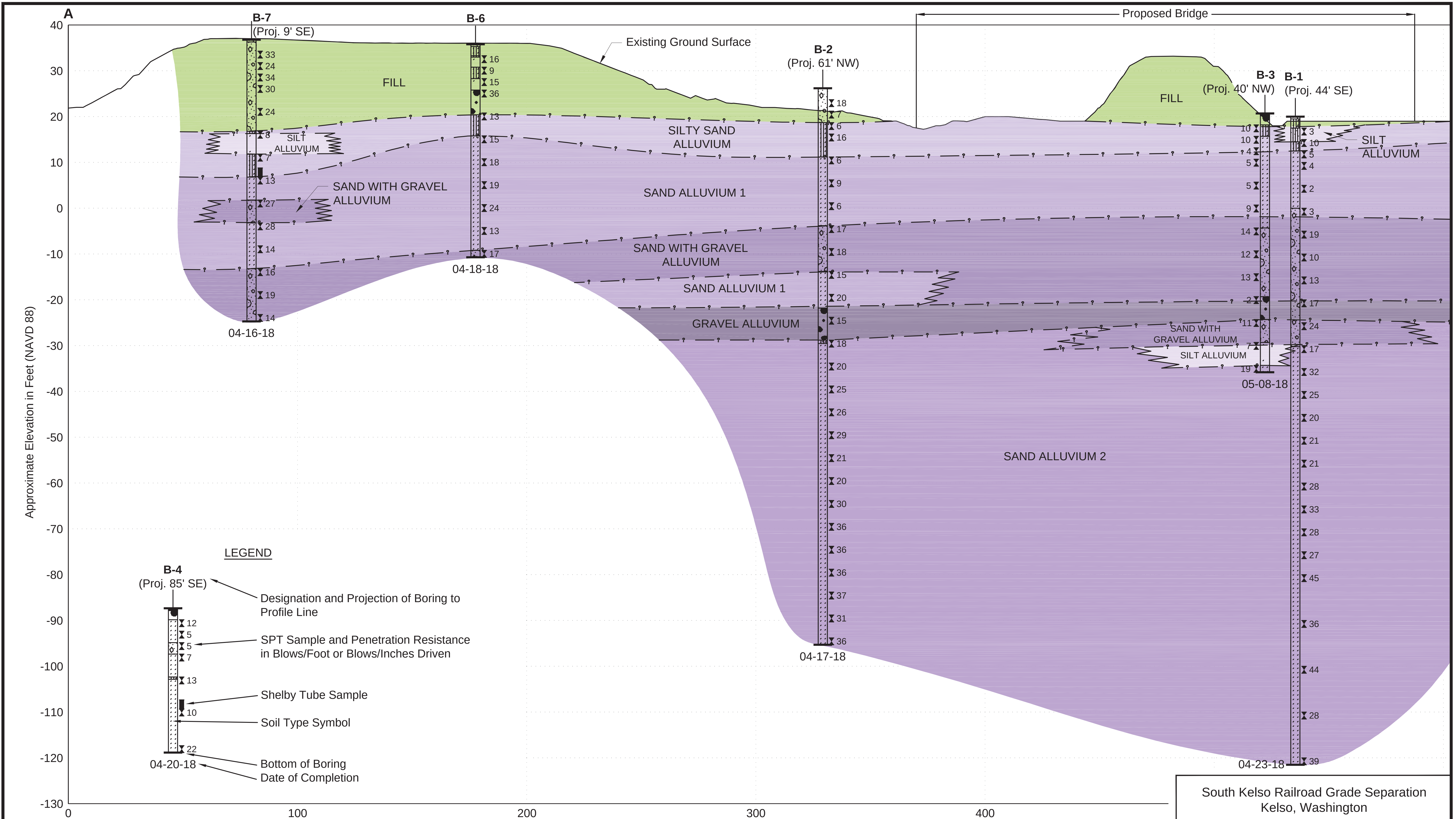
FIG. 2

Filename: T:\Projects\24-14201_S Kelso Grade S\Avmxd\SitePlan.mxd Date: 8/22/2018 Login: ath



0 20 40

Vertical Scale in Feet

0 40 80

Horizontal Scale in Feet

Vertical Exaggeration = 2x

NOTES

1. The ground surface was derived from 2010 LiDAR data and may not reflect current existing grade in all locations.
2. Profile generalized from materials observed in borings. Variations may exist between profile and actual conditions. See Appendix A for complete boring logs and explanations of symbols.
3. See Figure 2 for profile location.
4. Boring locations and elevations are approximate.

File: I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\DRAFTING\Profiles.dwg Date: 09-07-2018 Author: ath

South Kelso Railroad Grade Separation
Kelso, Washington

INTERPRETIVE SUBSURFACE PROFILE A-A'

June 2018 24-1-04201-001

SHANNON & WILSON, INC.
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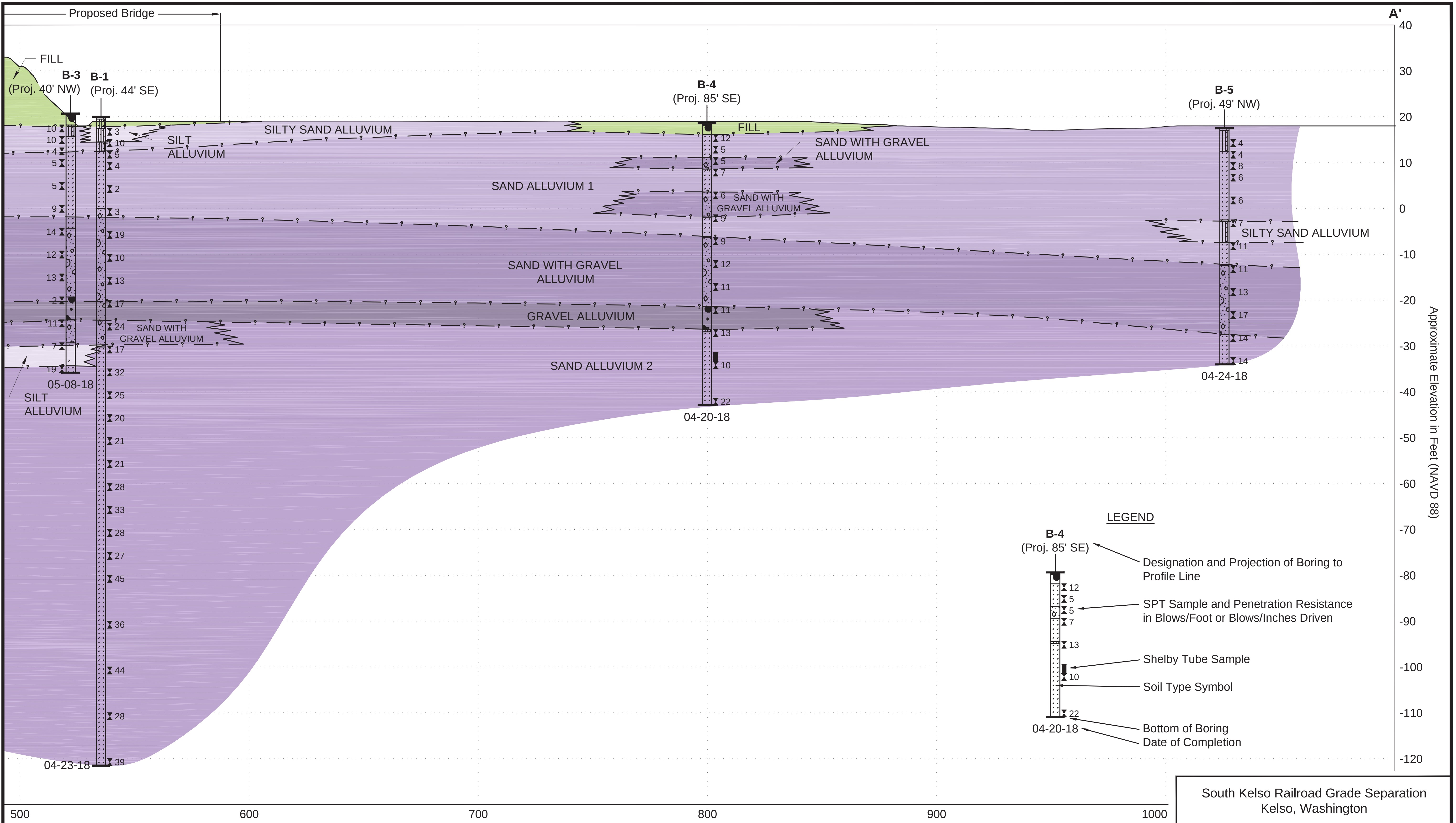
FIG. 3
Sheet 1 of 2



Approximate Distance in Feet

0 20 40 0 40 80

Vertical Scale in Feet

Horizontal Scale in Feet

Vertical Exaggeration = 2x

NOTES

1. The ground surface was derived from 2010 LiDAR data and may not reflect current existing grade in all locations.
2. Profile generalized from materials observed in borings. Variations may exist between profile and actual conditions. See Appendix A for complete boring logs and explanations of symbols.
3. See Figure 2 for profile location.
4. Boring locations and elevations are approximate.

South Kelso Railroad Grade Separation
Kelso, Washington

INTERPRETIVE SUBSURFACE PROFILE A-A'

June 2018
24-1-04201-001

SHANNON & WILSON, INC.
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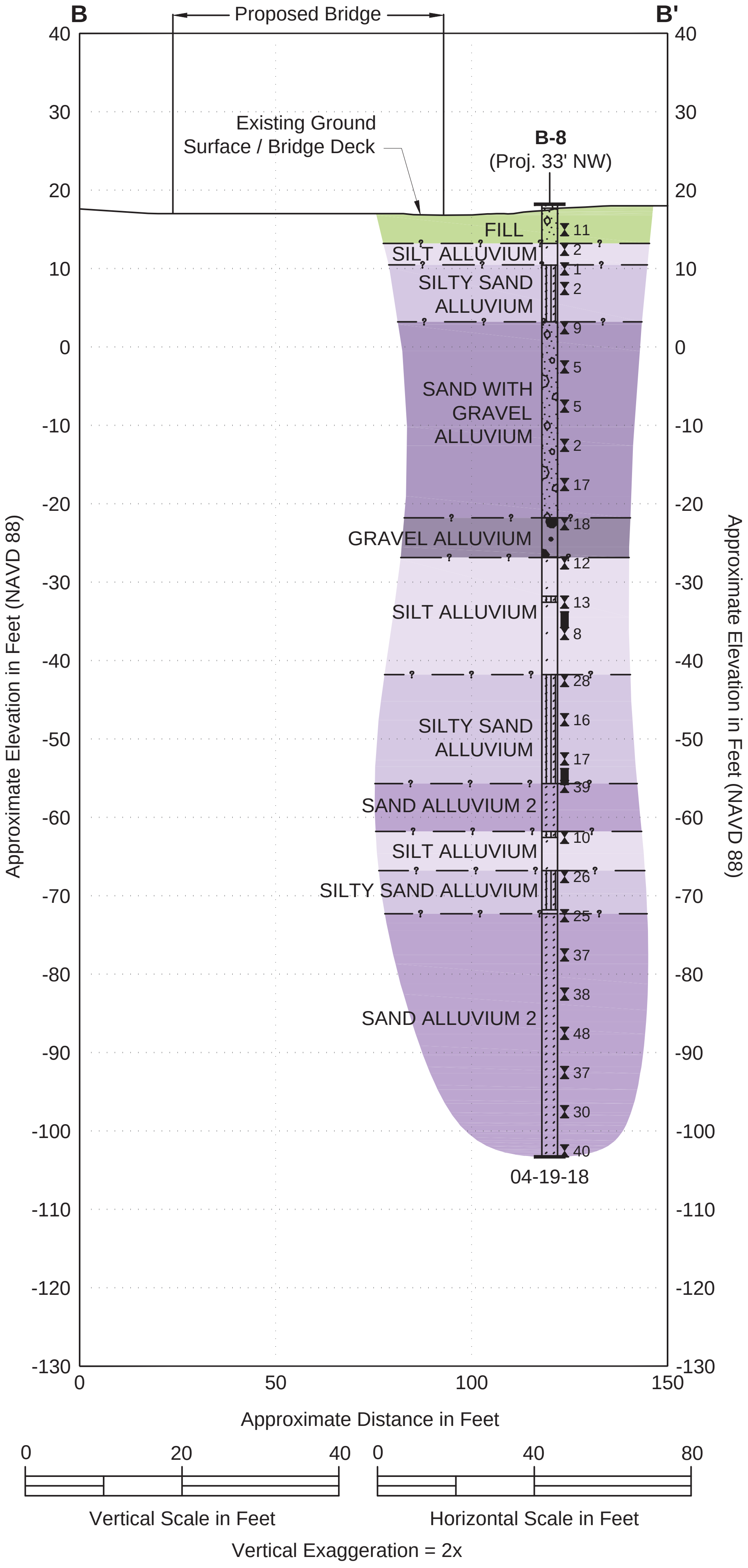
FIG. 3
Sheet 2 of 2

File: I:\EFI24-1 PDX\04200s\04201 S Kelso Grade S\DRAFTING\Profiles.dwg Date: 09-07-2018 Author: ath



File: I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\DRAFTING\Profiles.dwg Date: 09-07-2018 Author: ath

NOTES

1. The ground surface was derived from 2010 LiDAR data and may not reflect current existing grade in all locations.
2. Profile generalized from materials observed in borings. Variations may exist between profile and actual conditions. See Appendix A for complete boring logs and explanations of symbols.
3. See Figure 2 for profile location.
4. Boring locations and elevations are approximate.

South Kelso Railroad Grade Separation
Kelso, Washington

INTERPRETIVE SUBSURFACE PROFILE B-B'

June 2018
24-1-04201-001

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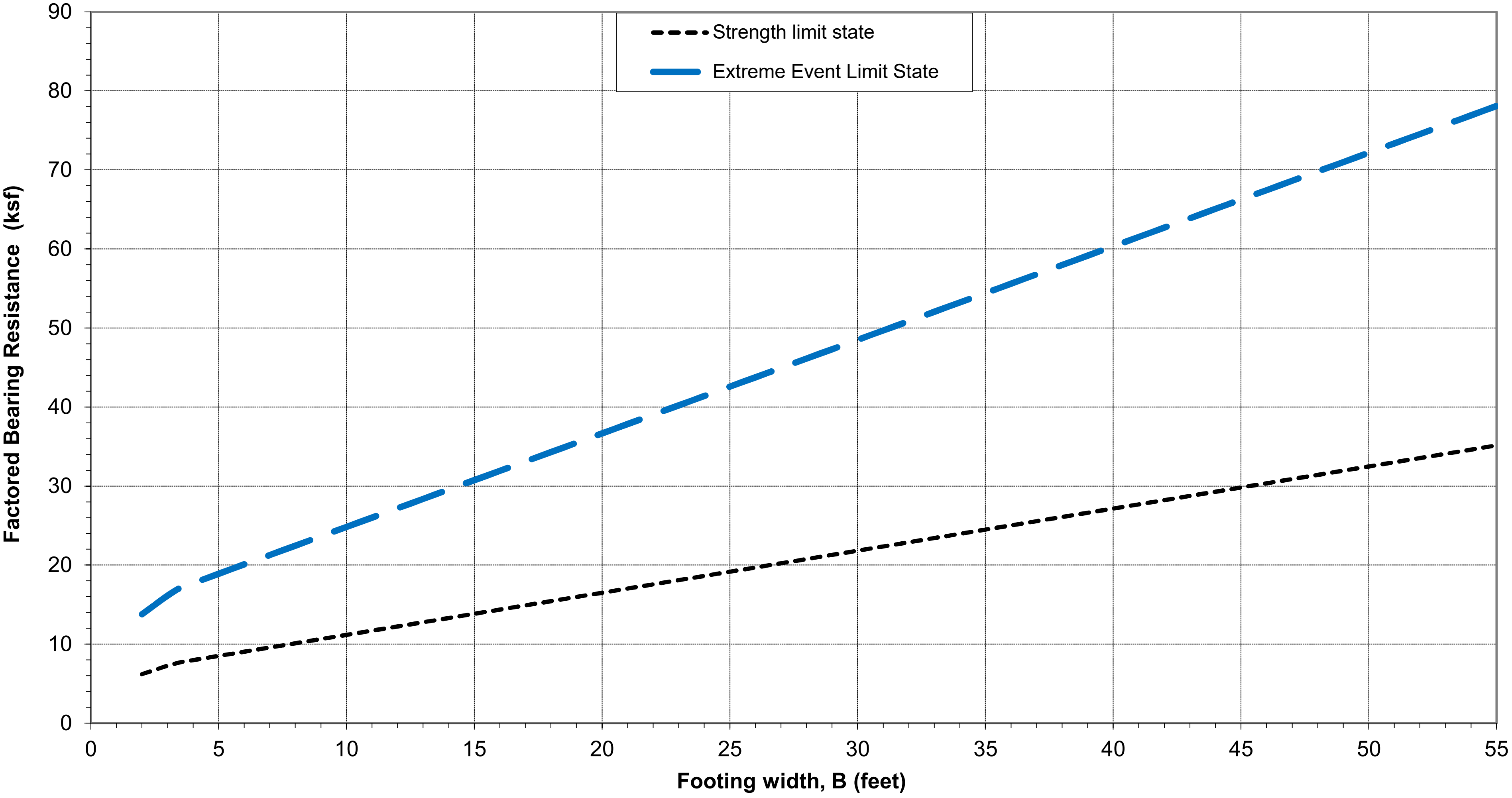
FIG. 4



NOTES

1. We recommend using the following resistance factors for footing LRFD design; the plotted bearing capacities use the bearing capacity resistance factors.

Limit State	Sliding Shear	Passive Press.	Bearing Capacity
Service	N/A	N/A	1.0
Strength	0.8	0.5	0.45
Extreme Event	1.0	1.0	1

2. The factored bearing capacities are based on a soil friction angle of 34 degrees, a soil cohesion of 0 psf, a total unit weight of 120 pcf, a Poisson's ratio of 0.2, and a soil elastic modulus of 570 ksf. We assumed that the bottom of the footing was 2 feet below the ground surface.
3. **psf** - pounds per square foot; **pcf** - pounds per cubic foot; **ksf** - kips per square foot (1 kip = 1000 pounds)

S. Kelso Railroad Grade Separation
South Kelso, Washington

FACTORED BEARING RESISTANCE VERSUS FOOTING WIDTH RECTANGULAR FOOTING, L/B = 10

September 2018 24-1-04201-001

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FIG. 5

APPENDIX A

FIELD EXPLORATIONS

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APPENDIX A

FIELD EXPLORATIONS

A.1 GENERAL

The field exploration program included eleven geotechnical borings, designated B-1 through B-11, and six Dynamic Cone Penetrometer tests (DCPs), designated DCP-1 through DCP-6, which were performed along the proposed alignment across South Pacific Avenue and BNSF rails extending from Hazel Street to South River Road. The boring, and DCP locations were not surveyed. The boring and DCP locations were referenced to nearby existing structures and should be considered approximate. Boring, and DCP test locations are shown on the Site and Exploration Plan, Figure 2. This appendix describes the techniques used to advance and sample the borings and presents logs of the materials encountered during drilling.

A.2 BORINGS

With the exception of boring B-3, the geotechnical borings were drilled between April 16 and 24, 2018, using a track-mounted CME 850 drill rig provided and operated by Western States Drilling, Inc. (Western States), of Hubbard, Oregon. Boring B-3 was drilled on May 8, 2018, using a truck-mounted CME 75 drill rig provided and operated by Western States. The borings were drilled to depths ranging from 11.5 to 141.5 feet. Borings B-1 through B-8, B-10, and B-11 were advanced using open-hole mud rotary drilling techniques. Hollow-stem auger drilling technique was used to advance boring B-9 to estimate of the groundwater table depth. A Shannon & Wilson field representative was on site during drilling to locate the borings, observe drilling, collect samples, and maintain logs of the materials encountered.

A.2.1 Disturbed Sampling

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

SPT N-values can be significantly affected by several factors, including the efficiency of the hammer used. Automatic hammers generally have higher energy transfer efficiencies than cathead (manual) hammers. Based on information we received from Western States Drilling, the energy transfer efficiency of the hammer used on the track-mounted CME 850 averaged 88.3 percent, and the hammer used on the truck-mounted CME 75 averaged 86 percent, when measured in January of 2017. For reference, cathead hammers are typically assumed to have an average energy efficiency of 60 percent. All N-values presented in this report are in blows per foot, as counted in the field. No corrections of any kind have been applied. N-values of zero indicate that the sampler advanced the last 12 inches of the 18-inch sampling interval without a single hammer strike. That is, the weight of the drilling rods or the weight of the drilling rods plus the weight of the hammer (not in motion) was sufficient to advance the sampler.

A.2.2 Undisturbed Sampling

Undisturbed samples were collected at selected depths using 3-inch O.D. thin-wall Shelby tubes. The tubes were pushed into undisturbed soil within the boreholes using down-pressure from the drill rig. The soils exposed at the ends of the tubes were identified and described in the field. After examination, the ends of the tubes were sealed with to preserve the natural moisture content of the samples. The sealed tubes were stored in the upright position and care was taken to avoid shock and vibration during their transport to and storage in the Shannon & Wilson laboratory.

A.3 BOREHOLE INSTALLATIONS AND ABANDONMENT

A.3.1 Vibrating Wire Pressure Transducer

A vibrating wire pressure transducer was installed in boring B-1 to a depth of approximately 38.75 feet below ground surface to measure groundwater level. A vibrating wire pressure transducer measures pressure using a pressure-sensitive diaphragm with a vibrating wire element attached to it. A cable runs from the transducer to the ground surface, where a readout device can be attached. The wire element vibrates when current is applied through the cable using the electronic readout device. Pressure acting on the outside face of the diaphragm causes it to deflect, which changes the tension of the wire element and the frequency of its vibration. The readout device measures the frequency of the induced vibration, which can be converted into a pressure, or height of groundwater above the transducer.

The vibrating wire pressure transducer was installed in the open borehole and taped to the outside of a 1-inch diameter PVC pipe to control the depth of installation. Prior to insertion, initial readings were taken with the transducers in a shallow bucket of water to determine field zero-head readings. With the transducer in place, the hole was backfilled with bentonite-cement grout. The cable leading up from the transducer was protected at the surface with flush-mount monuments set in concrete.

A.3.2 Borehole Abandonment

Borings that did not receive installations were backfilled in accordance with Washington Department of Ecology regulations, using bentonite-cement grout or bentonite chips Pavement sections penetrated by the borings were repaired with matching sections of nominally compacted gravel and asphalt cold patch or matching surface material.

A.4 MATERIAL DESCRIPTIONS

Soil samples were described and identified visually in the field, in general accordance with ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) and revised in accordance with Soil and Rock Classification and Logging from Chapter 4 of the WSDOT Geotechnical Design Manual (May 2015). Consistency, color, relative moisture, degree of plasticity, peculiar odors, and other distinguishing characteristics of the samples were noted. Once transported to our laboratory, the samples were re-examined, various classification tests were performed, and the field descriptions and identifications were modified where necessary.

A.5 LOGS OF TEST BORINGS

Summary logs of test borings are presented in the Log of Test Borings, Figures A1 through A11. Material descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The right-hand portion of the logs provides our description, identification, fines contents, moisture contents, sample recovery, and geotechnical unit designation for the materials encountered in the boring. The left-hand portion of the boring logs shows a graphic log, sample locations and designations, SPT blow counts and N-values, and a graphical representation of N-values, and natural water contents.

A.6 DYNAMIC CONE PENETROMETER TESTS

Pavement subgrade testing was conducted at the surface at six locations along the proposed alignment using a Dynamic Cone Penetrometer (DCP). The DCP is a device widely used to determine in-situ strength properties of base materials and subgrade soils. The four main components of the DCP include the cone, rod, anvil, and hammer. The cone is attached to one end of the DCP rod while the anvil and hammer are attached to the other end. Energy is applied to the cone tip through the rod by dropping the 17.64-pound hammer a distance of 22.6 inches against the anvil. The diameter of the cone is 0.16 inches larger than the rod to ensure that only tip resistance is measured. The number of blows required to advance the cone into the subsurface materials is recorded. The DCP index is the ratio of the depth of penetration to the number of blows of the hammer. This can be correlated to a variety of material properties, including California Bearing Ratio (CBR) and Resilient Modulus. DCP testing was performed and documented by Shannon & Wilson field personnel. DCP Test Data is presented in Figures A12 through A17.

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-1

Job No. 24-1-04201-001 SR

Elevation 20.0 ft

Figure A1 Sheet 1 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Site Address Right-of-way, S. Pacific Ave

Inspector Hoda Soltani

Start April 23, 2018 Completion April 23, 2018 Well ID#

Equipment CME 850 Track Rig #7

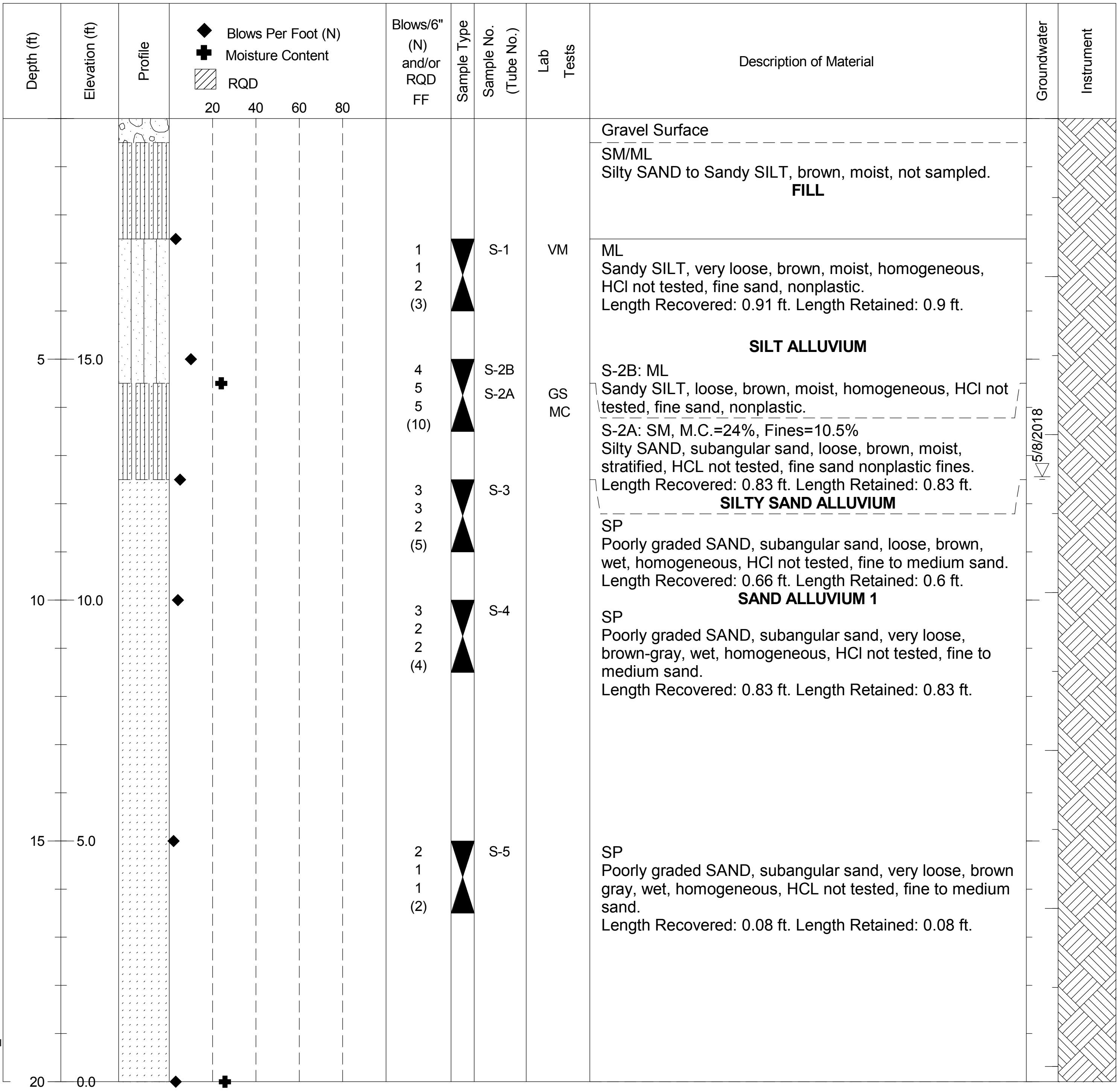
Station Offset Hole Dia (inches) 5

Method Mud Rotary

Northing 298129.4394 Easting 1029385.1556 Collected by

Datum WA83-SF, NAVD88

County Cowlitz Subsection NE1/4 of NE1/4 Section 3 Range 2W Township 7N

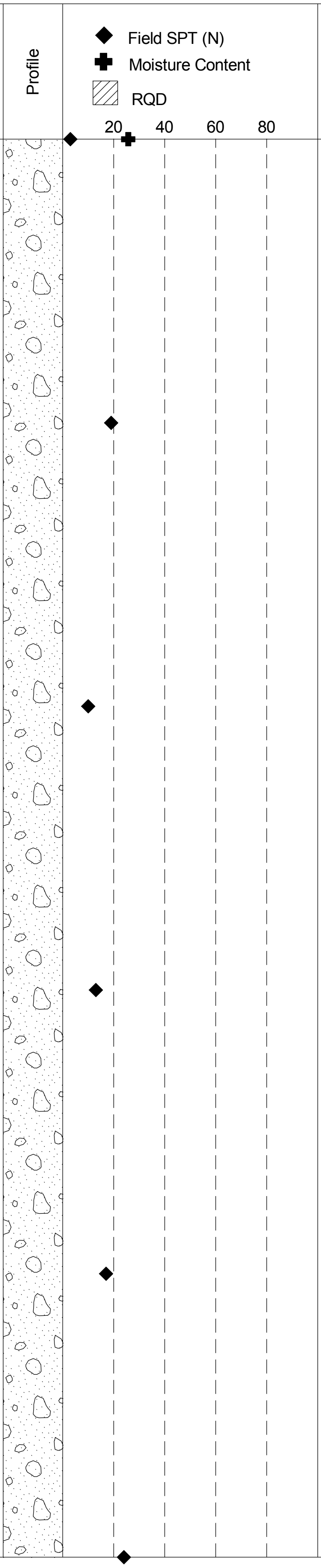


Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
					Gravel Surface
					SM/ML Silty SAND to Sandy SILT, brown, moist, not sampled. **FILL**
		1 1 2 (3)	S-1	VM	ML Sandy SILT, very loose, brown, moist, homogeneous, HCl not tested, fine sand, nonplastic. Length Recovered: 0.91 ft. Length Retained: 0.9 ft. **SILT ALLUVIUM**
5	15.0	4 5 5 (10)	S-2B S-2A	GS MC	S-2B: ML Sandy SILT, loose, brown, moist, homogeneous, HCl not tested, fine sand, nonplastic. S-2A: SM, M.C.=24%, Fines=10.5% Silty SAND, subangular sand, loose, brown, moist, stratified, HCL not tested, fine sand nonplastic fines. Length Recovered: 0.83 ft. Length Retained: 0.83 ft. **SILTY SAND ALLUVIUM**
		3 3 2 (5)	S-3		SP Poorly graded SAND, subangular sand, loose, brown, wet, homogeneous, HCl not tested, fine to medium sand. Length Recovered: 0.66 ft. Length Retained: 0.6 ft. **SAND ALLUVIUM 1**
10	10.0	3 2 2 (4)	S-4		SP Poorly graded SAND, subangular sand, very loose, brown-gray, wet, homogeneous, HCl not tested, fine to medium sand. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.
15	5.0	2 1 1 (2)	S-5		SP Poorly graded SAND, subangular sand, very loose, brown gray, wet, homogeneous, HCL not tested, fine to medium sand. Length Recovered: 0.08 ft. Length Retained: 0.08 ft.
20	0.0				

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card
HOLE No. B-1
Job No. 24-1-04201-001 SR
Elevation 20.0 ft
Figure A1 Sheet 2 of 6
Project South Kelso Railroad Grade Separation Project
Driller Western States Lic#



Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
		2 1 2 (3)	S-6	MC GS	SP, M.C.=26%, Gravel=10.5%, Sand=88.3%, Fines=1.2% Poorly graded SAND with few to little gravel, subangular gravel, subangular sand, very loose, brown-gray, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.
					SAND WITH GRAVEL ALLUVIUM
25	-5	6 10 9 (19)	S-7		SP Poorly graded SAND with gravel, subangular gravel, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.
30	-10	5 5 5 (10)	S-8		SP Poorly graded SAND with trace gravel, subangular gravel, subangular sand, loose, dark gray, wet, homogeneous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.5 ft. Length Retained: 0.5 ft.
35	-15	5 5 8 (13)	S-9		SP Poorly graded SAND with trace gravel, subangular, gravel, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.
					Geokon 4500-350kPa SN# 1809349 Vibrating Wire Piezometer installed at 38.75 feet.
40	-20	6 9 8 (17)	S-10		SP Poorly graded SAND with trace gravel, subangular gravel, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.
45	-25				

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-1

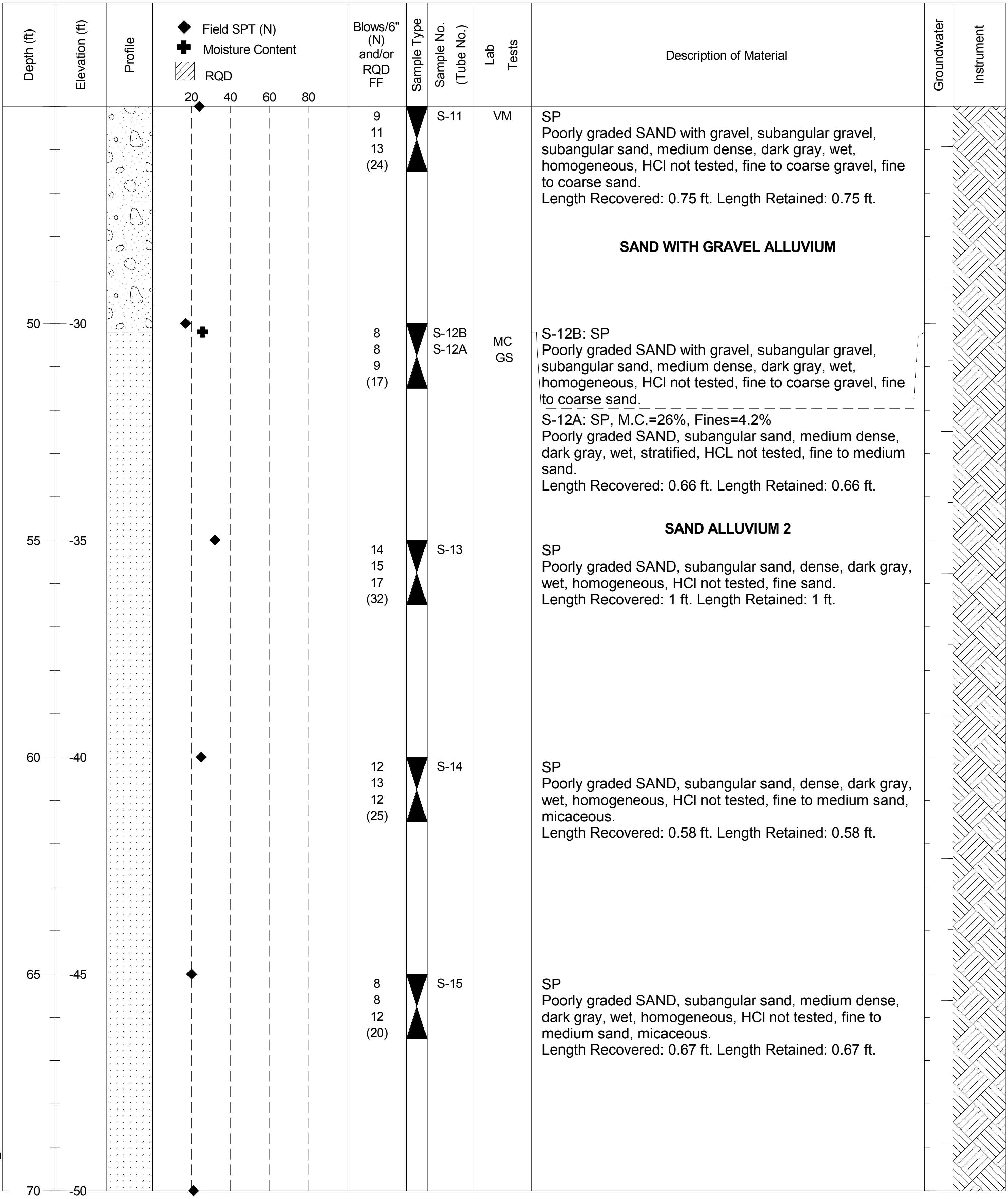
Job No. 24-1-04201-001 SR

Elevation 20.0 ft

Figure A1 Sheet 3 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#



Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
		9 11 13 (24)	S-11	VM	SP Poorly graded SAND with gravel, subangular gravel, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft. **SAND WITH GRAVEL ALLUVIUM**
50	-30	8 8 9 (17)	S-12B S-12A	MC GS	S-12B: SP Poorly graded SAND with gravel, subangular gravel, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. S-12A: SP, M.C.=26%, Fines=4.2% Poorly graded SAND, subangular sand, medium dense, dark gray, wet, stratified, HCL not tested, fine to medium sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft. **SAND ALLUVIUM 2**
55	-35	14 15 17 (32)	S-13		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine sand. Length Recovered: 1 ft. Length Retained: 1 ft.
60	-40	12 13 12 (25)	S-14		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.58 ft. Length Retained: 0.58 ft.
65	-45	8 8 12 (20)	S-15		SP Poorly graded SAND, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.67 ft. Length Retained: 0.67 ft.
70	-50				

LOG OF TEST BORING

Start Card

HOLE No. B-1

Job No. 24-1-04201-001

SR

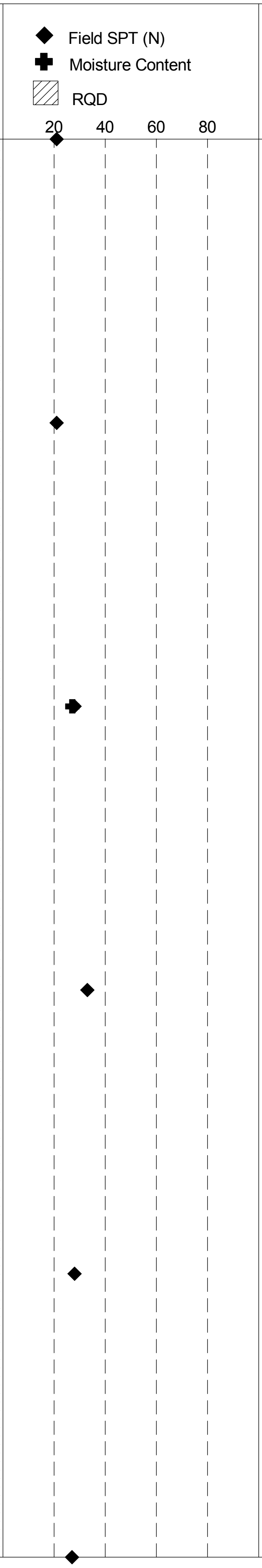
Elevation 20.0 ft

Figure A1 Sheet 4 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States

Lic#



Depth (ft)	Elevation (ft)	Profile	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
			9 11 10 (21)		S-16		SP Poorly graded SAND, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.67 ft. Length Retained: 0.67 ft.		
							SAND ALLUVIUM 2		
75	-55		10 10 11 (21)		S-17		SP Poorly graded SAND, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.		
80	-60		11 13 15 (28)		S-18	MC GS	SP, M.C.=27%, Fines=4.8% Poorly graded SAND, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.5 ft. Length Retained: 0.5 ft.		
85	-65		13 15 18 (33)		S-19		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.		
90	-70		15 13 15 (28)		S-20		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
95	-75								

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

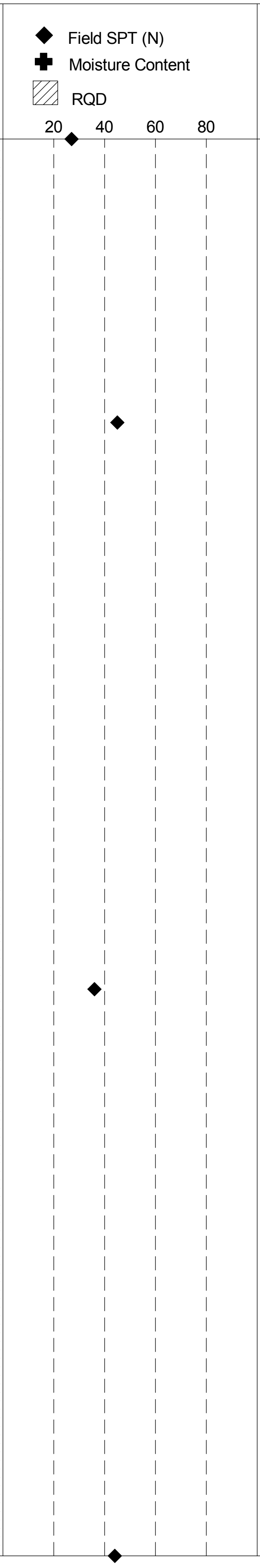
HOLE No. B-1

Job No. 24-1-04201-001 SR Elevation 20.0 ft

Figure A1 Sheet 5 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#



Depth (ft)	Elevation (ft)	Profile	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
			15 14 13 (27)		S-21		SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, trace coarse sand, trace to few coarse sand-sized pumice fragments. Length Recovered: 1 ft. Length Retained: 1 ft.		
							SAND ALLUVIUM 2		
100	-80		20 22 23 (45)		S-22		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine sand. Length Recovered: 0.91 ft. Length Retained: 0.91 ft.		
105	-85								
110	-90		15 14 22 (36)		S-23		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.91 ft. Length Retained: 0.91 ft.		
115	-95								
120	-100								

LOG OF TEST BORING

Start Card

HOLE No. B-1

Job No. 24-1-04201-001 SR

Elevation 20.0 ft

Figure A1 Sheet 6 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Depth (ft)	Elevation (ft)	Profile	◆ Field SPT (N) ✚ Moisture Content ▨ RQD 20 40 60 80	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				18 20 24 (44)		S-24		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 1.08 ft. Length Retained: 1.08 ft.		
125	-105							**SAND ALLUVIUM 2**		
130	-110			13 12 16 (28)		S-25		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
135	-115									
140	-120			15 19 20 (39)		S-26		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 1.25 ft. Length Retained: 1.25 ft.		
								End of test hole boring at 141.5 ft below ground elevation. This is a summary Log of Test Boring.		
145	-125									

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-10

Job No. 24-1-04201-001 SR

Elevation 18.7 ft

Figure A10 Sheet 1 of 1

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Site Address Grass Field, 2002 S. River Rd

Inspector Hoda Soltani

Start April 18, 2018 Completion April 18, 2018 Well ID#

Equipment CME 850 Track Rig #7

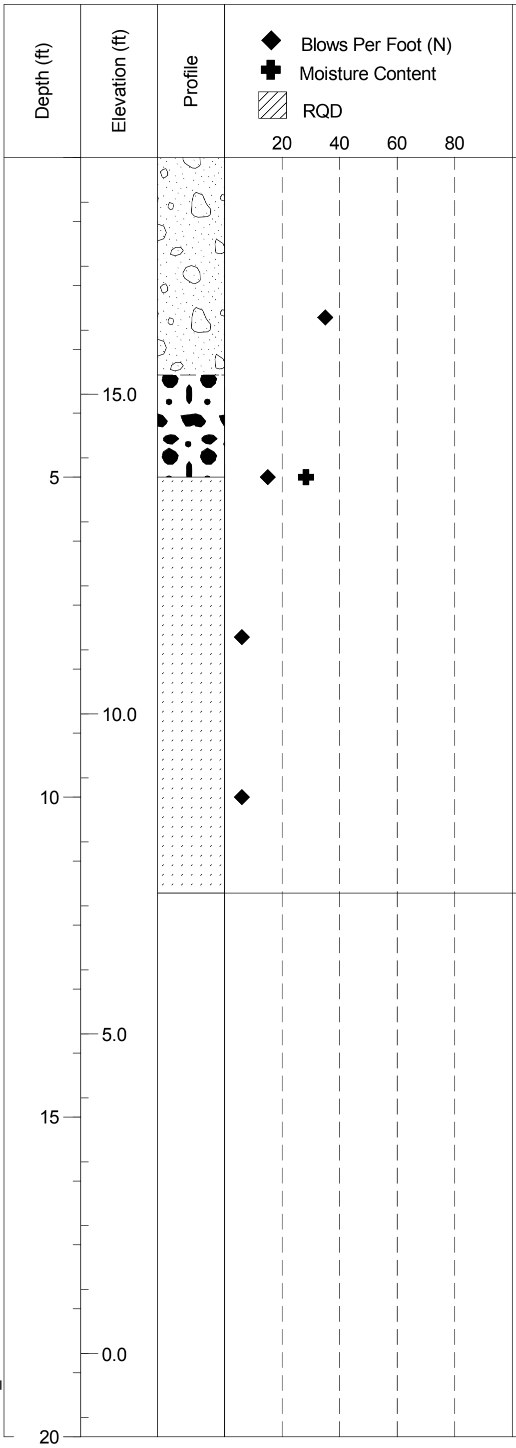
Station Offset Hole Dia (inches) 5

Method Mud Rotary

Northing 297816.5878 Easting 1028562.4905 Collected by

Datum WA83-SF, NAVD88

County Cowlitz Subsection NE1/4 of NE1/4 Section 3 Range 2W Township 7N



Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
			SP Poorly graded SAND with gravel, dark gray, moist, not sampled. **FILL**
7 18 17 (35)	S-1B S-1A	VM	S-1B: SP Poorly graded SAND with gravel, subangular gravel, subangular sand, dense, dark gray, moist, stratified, HCl not tested, fine gravel, fine to coarse sand. S-1A: GP Poorly graded GRAVEL with sand, subangular gravel, subangular sand, dense, dark gray, moist, stratified, fine to coarse gravel, fine to coarse sand. Length Recovered: 1 ft. Length Retained: 1 ft.
9 7 8 (15)	S-2	MC GS	SP, M.C.=28%, Fines=4.8% Poorly graded SAND, subangular sand, medium dense, brown, moist, homogenous, HCl not tested, fine to medium sand. Length Recovered: 1.08 ft. Length Retained: 1.08 ft. **SAND ALLUVIUM 1**
4 2 4 (6)	S-3		SP Poorly graded SAND, subangular sand, loose, brown, moist, homogenous, HCl not tested, fine to medium sand. Length Recovered: 0.5 ft. Length Retained: 0.5 ft.
3 3 3 (6)	S-4		SP Poorly graded SAND, subangular sand, loose, brown, moist, homogenous, HCl not tested, fine to coarse sand. Length Recovered: 1 ft. Length Retained: 1 ft.
			End of test hole boring at 11.5 ft below ground elevation. This is a summary Log of Test Boring.

Groundwater Instrument

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-11

Job No. 24-1-04201-001 SR Elevation 21.6 ft

Figure A11 Sheet 1 of 1

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Site Address Grass Field, 2002 S. River Rd

Inspector Hoda Soltani

Start April 18, 2018 Completion April 18, 2018 Well ID#

Equipment CME 850 Track Rig #7

Station Offset Hole Dia (inches) 5

Method Mud Rotary

Northing 297589.9725 Easting 1027858.224 Collected by

Datum WA83-SF, NAVD88

County Cowlitz Subsection NE1/4 of NW1/4 Section 3 Range 2W Township 7N

Depth (ft)	Elevation (ft)	Profile	◆ Blows Per Foot (N) ✚ Moisture Content ▨ RQD 20 40 60 80	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
								Sod		
								SM Silty SAND, brown to red-brown, moist, not sampled. **SILTY SAND ALLUVIUM**		
	20.0									
			◆	3 3 2 (5)		S-1	VM	SM Silty SAND, subangular to subrounded sand, loose, red-brown, wet, homogenous, HCl not tested, fine sand, nonplastic fines. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
5			◆	2 1 2 (3)		S-2B S-2A		S-2B: ML SILT with Sand, very loose, brown, moist, stratified, HCl not tested, fine sand, nonplastic. S-2A: SM Silty SAND, subangular sand, very loose, brown, wet, stratified, HCL not tested, fine sand, nonplastic fines. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
	15.0									
			◆	2 2 2 (4)		S-3		SM Silty SAND, subangular sand, very loose, brown, wet, stratified, HCl not tested, fine sand, nonplastic fines. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
10			◆	2 3 4 (7)		S-4		SP Poorly graded SAND, subangular sand, loose, brown, wet, homogenous, HCl not tested, fine to medium sand. Length Recovered: 1 ft. Length Retained: 1 ft. **SAND ALLUVIUM 1**		
	10.0							End of test hole boring at 11.5 ft below ground elevation. This is a summary Log of Test Boring.		
15										
	5.0									
20										

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-2

Job No. 24-1-04201-001 SR

Elevation 26.2 ft

Figure A2 Sheet 1 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Site Address Golf Course, 2222 S. River Rd

Inspector Hoda Soltani

Start April 17, 2018 Completion April 17, 2018 Well ID#

Equipment CME 850 Track Rig #7

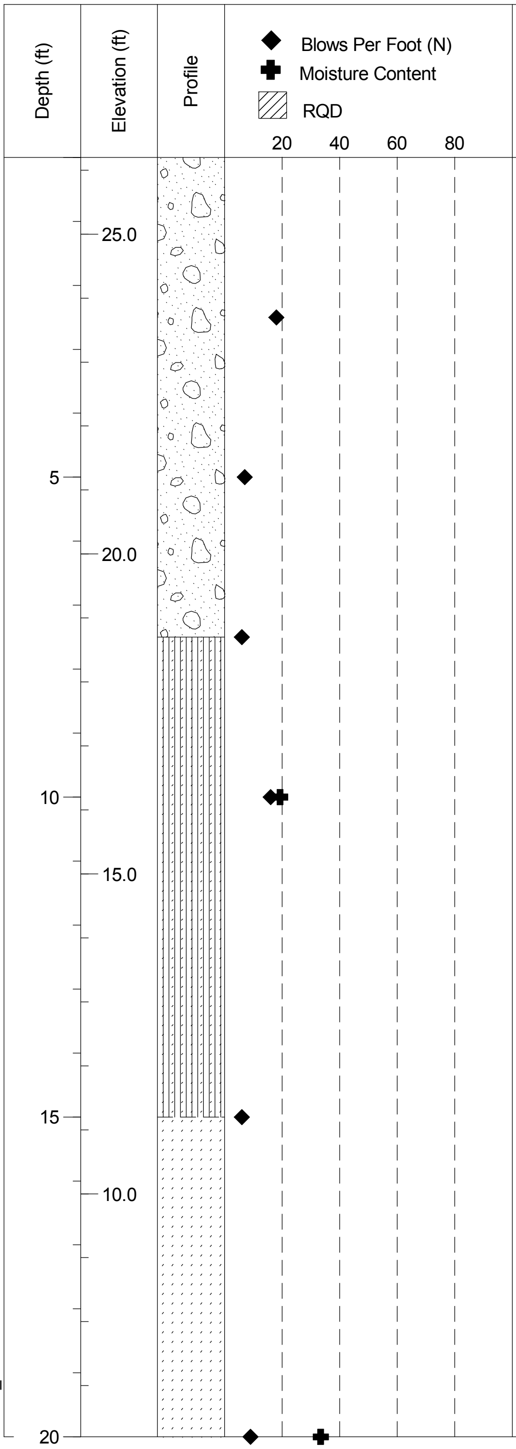
Station Offset Hole Dia (inches) 5

Method Mud Rotary

Northing 297923.7643 Easting 1029278.3398 Collected by

Datum WA83-SF, NAVD88

County Cowlitz Subsection NE1/4 of NE1/4 Section 3 Range 2W Township 7N



Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				SP Poorly Graded SAND with gravel, gray-brown, moist, not sampled. **FILL**		
9 9 9 (18)	⧗	S-1	VM	SP Poorly graded SAND with gravel, subangular gravel, subangular sand, medium dense, gray-brown, moist, stratified, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 1.25 ft. Length Retained: 1 ft.		
4 4 3 (7)	⧗	S-2		SP Poorly graded SAND with gravel, subangular gravel, subangular sand, medium dense, gray-brown, moist, stratified, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.6 ft. Length Retained: 0.6 ft.		
4 3 3 (6)	⧗	S-3	VM	SM Silty SAND, subangular sand, loose, gray-brown, moist, homogenous, HCl not tested, fine sand, nonplastic fines. Length Recovered: 0.83 ft. Length Retained: 0.8 ft. **SILTY SAND ALLUVIUM**		
5 8 8 (16)	⧗	S-4	MC GS	SM, M.C.=19%, Gravel=21.6%, Sand=57.1%, Fines=21.3% Silty SAND with gravel, subangular to subrounded gravel, subangular sand, medium dense, dark brown, moist, homogenous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.		
4 3 3 (6)	⧗	S-5		SP Poorly graded SAND, subangular sand, loose, dark gray, moist, homogenous, HCl not tested, fine to medium sand. Length Recovered: 0.58 ft. Length Retained: 0.58 ft. **SAND ALLUVIUM 1**		

LOG OF TEST BORING

Start Card

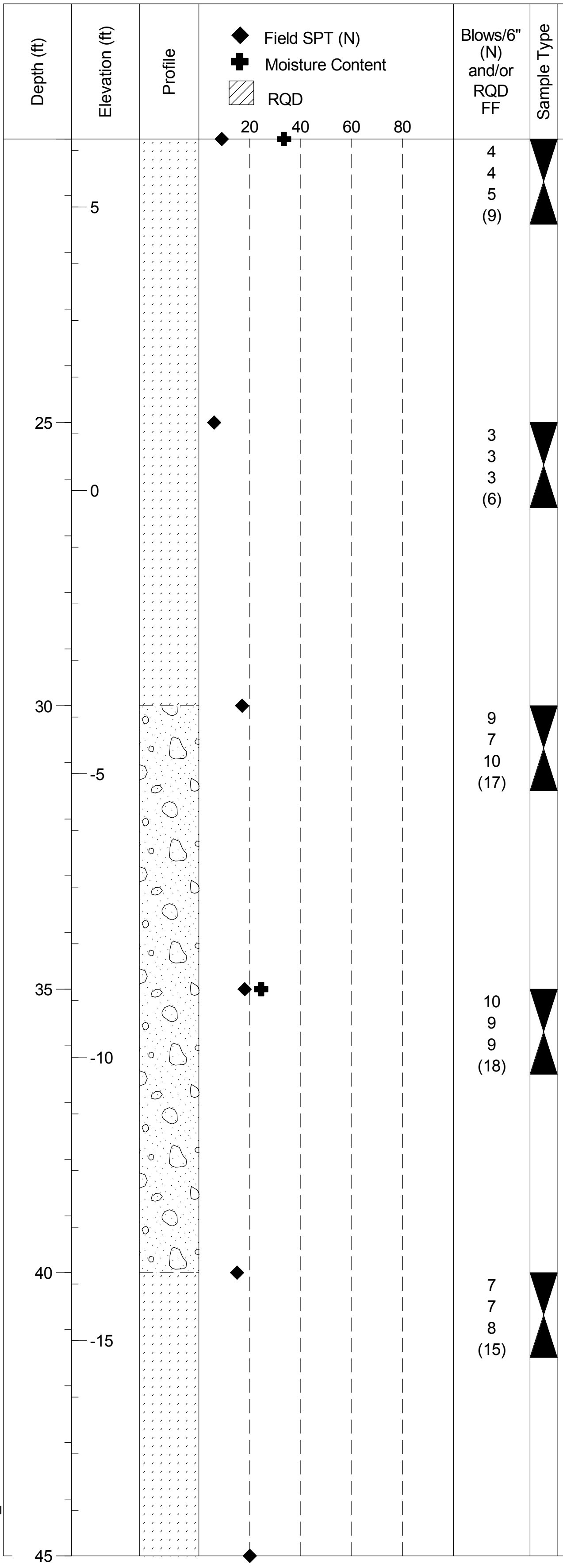
HOLE No. B-2

Job No. 24-1-04201-001 SR Elevation 26.2 ft

Figure A2 Sheet 2 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#



Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
		4 4 5 (9)	S-6	MC GS	SP, M.C.=33%, Fines=3.1% Poorly graded SAND, subangular sand, loose, dark gray, wet, homogenous, HCl not tested, fine to medium sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.
	5				**SAND ALLUVIUM 1**
25		3 3 3 (6)	S-7		SP Poorly graded SAND, subangular sand, loose, dark gray, wet, homogenous, HCl not tested, fine to medium sand. Length Recovered: 0.58 ft. Length Retained: 0.58 ft.
	0				
30		9 7 10 (17)	S-8		SP Poorly graded SAND with gravel, subangular to rounded gravel, subangular sand, medium dense, dark gray to yellow-brown, wet, homogenous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.
	-5				**SAND WITH GRAVEL ALLUVIUM**
35		10 9 9 (18)	S-9	MC GS	SP, M.C.=25%, Fines=4.9% Poorly graded SAND with trace gravel, subangular to subrounded gravel, subangular sand, medium dense, dark gray, wet, homogenous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.58 ft. Length Retained: 0.58 ft.
	-10				
40		7 7 8 (15)	S-10		SP Poorly graded SAND, subangular sand, medium dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand. Length Recovered: 0.58 ft. Length Retained: 0.58 ft.
	-15				**SAND ALLUVIUM 1**
45					

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-2

Job No. 24-1-04201-001

SR

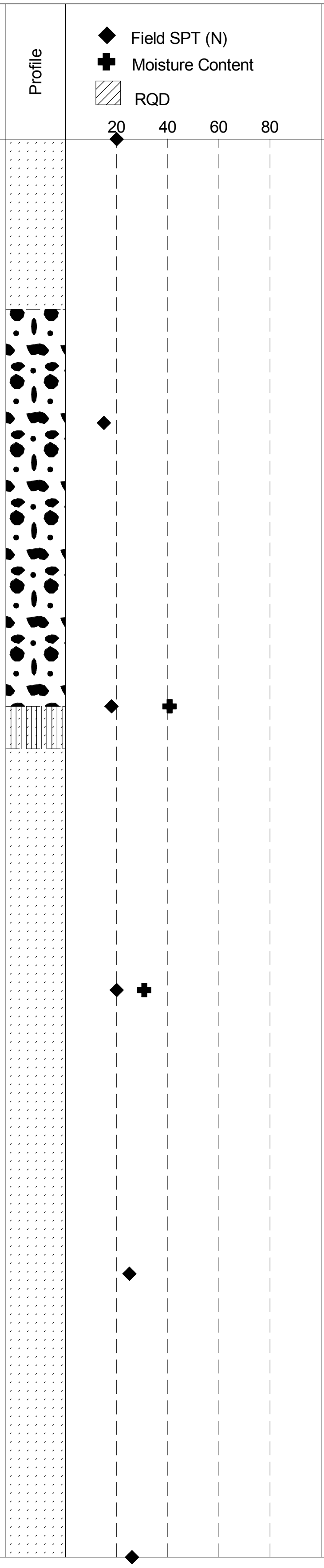
Elevation 26.2 ft

Figure A2 Sheet 3 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States

Lic#



Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
		10 9 11 (20)	S-11		SP Poorly graded SAND, subangular sand, medium dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand. Length Recovered: 0.83 ft. Length Retained: 0.83 ft. **SAND ALLUVIUM 1**
	-20				
					Driller indicates harder drilling at 48 feet.
50		14 8 7 (15)	S-12	VM	GP Poorly graded GRAVEL with sand, subangular to subrounded gravel, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand, trace organics. Length Recovered: 0.83 ft. Length Retained: 0.83 ft. **GRAVEL ALLUVIUM**
	-25				
55		3 8 10 (18)	S-13B S-13A	MC GS	S-13B: SM. M.C.=41%, Fines=36.1% Silty SAND, subangular to subrounded sand, medium dense, gray, wet, stratified, HCl not tested, fine sand, nonplastic fines. **SILTY SAND ALLUVIUM** S-13A: SP Poorly graded SAND, subangular sand, medium dense, olive gray to gray, wet, stratified, HCl not tested, fine to medium sand. Length Recovered: 1.08 ft. Length Retained: 1 ft. **SAND ALLUVIUM 2**
	-30				
60		9 9 11 (20)	S-14B S-14A	MC GS	S-14B: SP, M.C.=31%, Fines=7.3% Poorly graded SAND, subangular sand, medium dense, dark gray, wet, stratified, HCl not tested, fine to medium sand. S-14A: SP Poorly graded SAND, subangular sand, medium dense, dark gray, wet, stratified, HCl not tested, fine sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.
	-35				
65		12 12 13 (25)	S-15		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogenous, HCl not tested, fine sand, micaceous. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.
	-40				
70					

Sample Type

Groundwater

Instrument

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-2

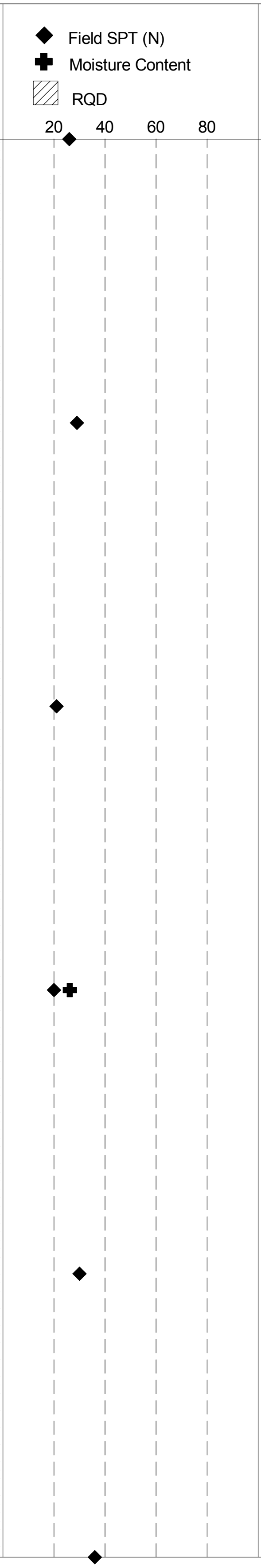
Job No. 24-1-04201-001 SR

Elevation 26.2 ft

Figure A2 Sheet 4 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#



Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
		15 11 14 (26)	S-16		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.
	-45				
					SAND ALLUVIUM 2
75		12 14 15 (29)	S-17		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 1 ft. Length Retained: 1 ft.
	-50				
80		11 9 12 (21)	S-18		SP Poorly graded SAND, subangular sand, medium dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.
	-55				
85		15 10 10 (20)	S-19	MC GS	SP, M.C.=26%, Fines=3.6% Poorly graded SAND, subangular sand, medium dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.
	-60				
90		15 14 16 (30)	S-20		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.
	-65				
95					

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

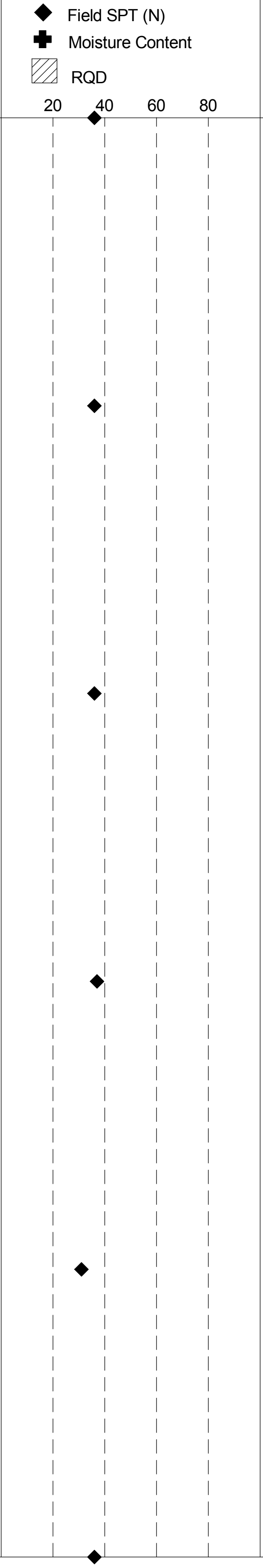
HOLE No. B-2

Job No. 24-1-04201-001 SR Elevation 26.2 ft

Figure A2 Sheet 5 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#



Depth (ft)	Elevation (ft)	Profile	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	-70		10 16 20 (36)		S-21		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.91 ft. Length Retained: 0.91 ft.		
							SAND ALLUVIUM 2		
100	-75		15 13 23 (36)		S-22		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.		
105	-80		16 18 18 (36)		S-23		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogenous, HCl not tested, fine sand, micaceous. Length Recovered: 0.92 ft. Length Retained: 0.92 ft.		
110	-85		15 17 20 (37)		S-24		SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, wet, homogenous, HCl not tested, fine to coarse sand, micaceous. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.		
115	-90		15 15 16 (31)		S-25		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogenous, HCl not tested, fine to coarse sand, micaceous, trace coarse sand-sized pumice fragments. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.		
120									

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card ______

HOLE No. B-2

Job No. 24-1-04201-001 SR ______ Elevation 26.2 ft

Figure A2 Sheet 6 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic# ______

◆ Field SPT (N)
✚ Moisture Content
▨ RQD

Depth (ft)	Elevation (ft)	Profile	20 40 60 80	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	-95			18 19 17 (36)		S-26		SP Poorly graded SAND, subangular sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.91 ft. Length Retained: 0.91 ft. **SAND ALLUVIUM 2** End of test hole boring at 121.5 ft below ground elevation. This is a summary Log of Test Boring.		
125	-100									
130	-105									
135	-110									
140	-115									
145										

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-3

Job No. 24-1-04201-001
SR
Elevation 20.7 ft
Figure A3 Sheet 1 of 3

Project South Kelso Railroad Grade Separation Project
Driller Western States Lic#

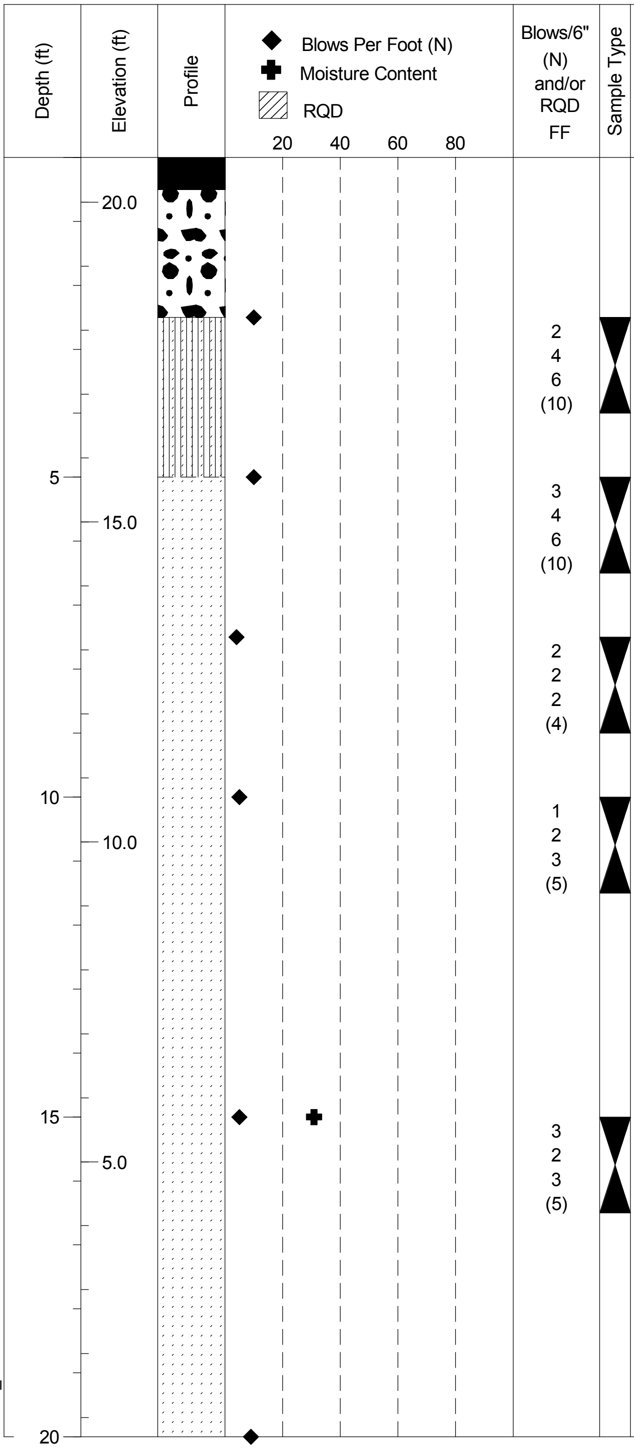
Site Address Right-of-way, S. Pacific Ave
Inspector Hoda Soltani

Start May 8, 2018
Completion May 8, 2018
Well ID#
Equipment CME 75 Truck Rig #5

Station
Offset
Hole Dia (inches) 5
Method Mud

Northing 298052.9527
Easting 1029423.1821
Collected by
Datum WA83-SF, NAVD88

County Cowlitz
Subsection NE1/4 of NE1/4
Section 3
Range 2W
Township 7N



Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
			Asphalt Concrete
			GP Poorly graded GRAVEL with sand, gray-brown, moist, not sampled. **FILL**
2 4 6 (10)	S-1		SM Silty SAND, subangular sand, loose, gray-brown, moist, homogeneous, HCl not tested, fine to medium sand, nonplastic fines. Length Recovered: 0.92 ft. Length Retained: 0.92 ft. **SILTY SAND ALLUVIUM**
3 4 6 (10)	S-2		SP Poorly graded SAND, subangular sand, loose, gray, moist, homogeneous, HCl not tested, fine to medium sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft. **SAND ALLUVIUM 1**
2 2 2 (4)	S-3		SP Poorly graded SAND, subangular sand, very loose, gray, moist, homogeneous, HCl not tested, fine to medium sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.
1 2 3 (5)	S-4		SP Poorly graded SAND, subangular sand, loose, gray, moist, homogeneous, HCl not tested, fine sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.
3 2 3 (5)	S-5	GS MC	SP, M.C.=31%, Fines=2.3% Poorly graded SAND, subangular sand, loose, gray, moist, homogeneous, HCl not tested, fine to medium sand. Length Recovered: 0.58 ft. Length Retained: 0.58 ft.

Groundwater
Instrument

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card ____________

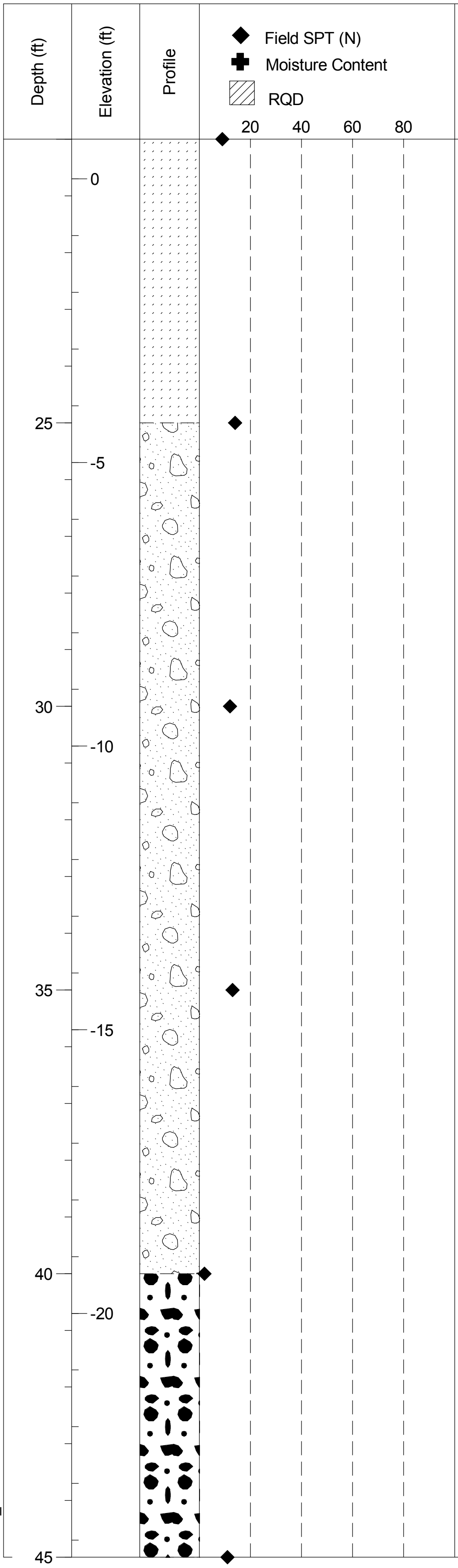
Job No. 24-1-04201-001 SR ________ Elevation 20.7 ft

HOLE No. B-3

Figure A3 Sheet 2 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic# ________



Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
3 5 4 (9)	⧗	S-6		SP Poorly graded SAND, subangular sand, loose, gray, moist, homogeneous, HCl not tested, fine to coarse sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
				SAND ALLUVIUM 1		
6 6 8 (14)	⧗	S-7	VM	SP Poorly graded SAND with gravel, subangular to subrounded gravel, subangular sand, medium dense, gray, moist, stratified, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
				SAND WITH GRAVEL ALLUVIUM		
4 4 8 (12)	⧗	S-8		SP Poorly graded SAND with trace gravel, subangular to subrounded gravel, subangular sand, medium dense, gray, moist, stratified, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
6 7 6 (13)	⧗	S-9	VM	SP Poorly graded SAND with gravel, subangular to subrounded gravel, subangular sand, medium dense, gray, moist, stratified, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.		
2 1 1 (2)	⧗	S-10		GP Poorly graded GRAVEL with sand, subangular to subrounded gravel, subangular sand, very loose, gray, moist, stratified, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.2 ft. Length Retained: 0.2 ft.		
				GRAVEL ALLUVIUM		

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-3

Job No. 24-1-04201-001

SR

Elevation 20.7 ft

Figure A3 Sheet 3 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States

Lic#

Depth (ft)	Elevation (ft)	Profile	◆ Field SPT (N) ✚ Moisture Content ▨ RQD 20 40 60 80	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	-25			5 6 5 (11)		S-11		SP Poorly graded SAND with trace gravel, subangular to subrounded gravel, subangular sand, medium dense, gray, wet, stratified, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.2 ft. Length Retained: 0.2 ft. **SAND WITH GRAVEL ALLUVIUM**		
50	-30			1 2 5 (7)		S-12	GS MC	ML, M.C.=45%, Fines=82.1% SILT with Sand, medium stiff, gray, wet, stratified, HCl not tested, fine sand, nonplastic to low plasticity. Length Recovered: 1.4 ft. Length Retained: 1.4 ft. **SILT ALLUVIUM**		
55	-35			6 9 10 (19)		S-13		SP Poorly graded SAND, subangular sand, medium dense, gray, wet, homogeneous, HCL not tested, fine to medium sand, micaceous. Length Recovered: 1 ft. Length Retained: 1 ft. **SAND ALLUVIUM 2**		
								End of test hole boring at 56.5 ft below ground elevation. This is a summary Log of Test Boring.		
60	-40									
65	-45									
70										

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-4

Figure A4 Sheet 1 of 3

Job No. 24-1-04201-001 SR Elevation 18.6 ft

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Site Address Right-of-way, Hazel St

Inspector Hoda Soltani

Start April 20, 2018 Completion April 20, 2018 Well ID#

Equipment CME 850 Track Rig #7

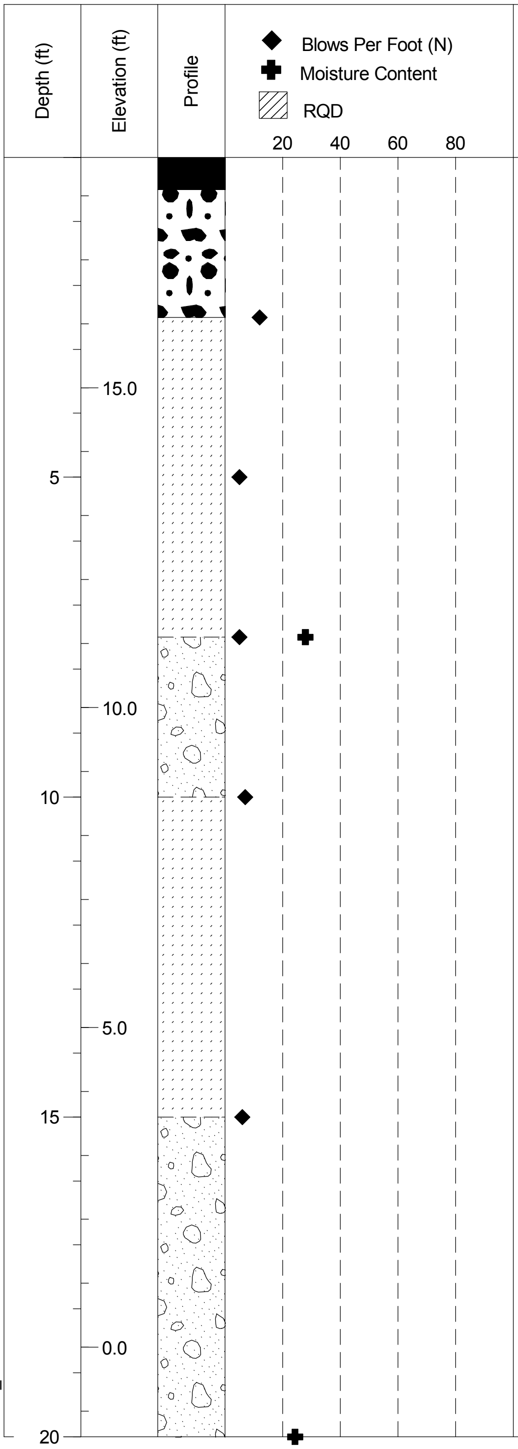
Station Offset Hole Dia (inches) 5

Method Mud Rotary

Northing 298315.9471 Easting 1029576.9793 Collected by

Datum WA83-SF, NAVD88

County Cowlitz Subsection NE1/4 of NE1/4 Section 3 Range 2W Township 7N



Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				Asphalt Concrete		
				GP Poorly graded GRAVEL with sand, gray-brown, moist, not sampled. **FILL**		
7 7 5 (12)		S-1	VM	SP Poorly graded SAND, subangular sand, medium dense, brown, moist, homogenous, HCl not tested, fine to medium sand. Length Recovered: 1 ft. Length Retained: 1 ft. **SAND ALLUVIUM 1**		
2 2 3 (5)		S-2		SP Poorly graded SAND, subangular sand, loose, brown, moist, homogenous, HCl not tested, fine sand. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.		
3 3 2 (5)		S-3	MC GS	SP, M.C.=28%, Fines=3.0% Poorly graded SAND trace gravel, subangular gravel, subangular sand, loose, brown, moist, stratified, HCl not tested, fine gravel, fine to medium sand. Length Recovered: 0.83 ft. Length Retained: 0.83 ft. **SAND WITH GRAVEL ALLUVIUM**		
3 3 4 (7)		S-4		SP Poorly graded SAND, subangular to subrounded sand, loose, brown, moist, homogenous, HCl not tested, fine to coarse sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft. **SAND ALLUVIUM 1**		
3 2 4 (6)		S-5		SP Poorly graded SAND with gravel, subrounded gravel, subangular to subrounded sand, loose, gray, wet, homogenous, HCl not tested, fine to coarse gravel, fine to medium sand. Length Recovered: 0.16 ft. Length Retained: 0.16 ft. **SAND WITH GRAVEL ALLUVIUM**		

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-4

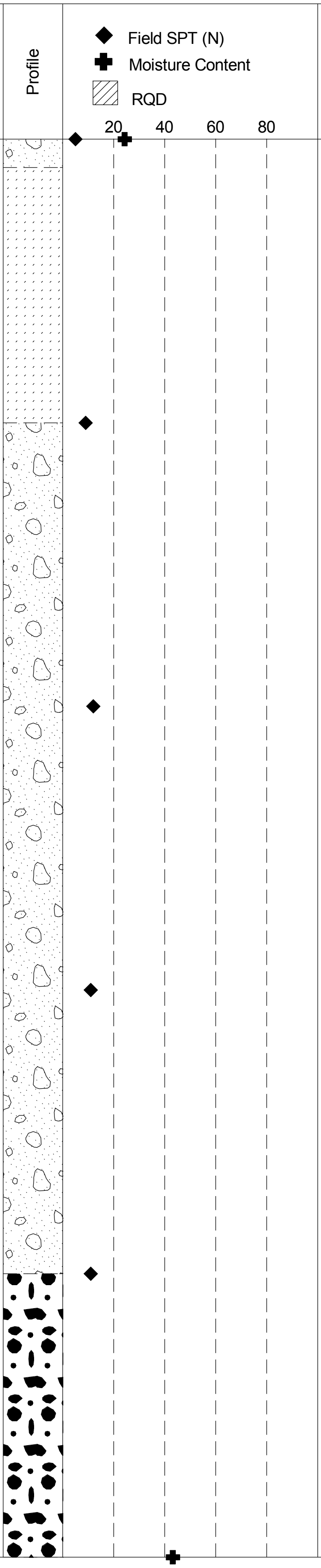
Job No. 24-1-04201-001 SR

Elevation 18.6 ft

Figure A4 Sheet 2 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#



Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
		3 2 3 (5)		S-6B S-6A	MC GS	S-6B: SP, M.C.=24%, Gravel=11.8%, Sand=84.3%, Fines=3.9% Poorly graded SAND with few to little gravel, subangular gravel, subangular sand, loose, gray, wet, stratified, HCl not tested, fine gravel, fine to coarse sand. S-6A: SP Poorly graded SAND, subangular sand, loose, gray, wet, stratified, HCl not tested, fine to medium sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
	-5					**SAND ALLUVIUM 1**		
25		1 2 7 (9)		S-7B S-7A		S-7B and S-7A: SP Poorly graded SAND with gravel, subangular to subrounded gravel, subangular sand, medium dense, gray, wet, stratified, HCl not tested, fine to coarse gravel, fine to coarse sand, trace to few organics and wood debris. Length Recovered: 0.5 ft. Length Retained: 0.5 ft.		
	-10					**SAND WITH GRAVEL ALLUVIUM**		
30		8 5 7 (12)		S-8	VM	SP Poorly graded SAND with gravel, subangular to subrounded gravel, subangular sand, medium dense, gray, wet, stratified, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.58 ft. Length Retained: 0.58 ft.		
	-15							
35		6 5 6 (11)		S-9		SP Poorly graded SAND with gravel, subangular to subrounded gravel, subangular sand, medium dense, gray, wet, stratified, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.5 ft. Length Retained: 0.5 ft.		
	-20							
40		8 6 5 (11)		S-10		GP Poorly graded GRAVEL wtih sand, subangular to subrounded gravel, subangular sand, medium dense, gray, wet, stratified, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.5 ft. Length Retained: 0.5 ft.		
	-25					**GRAVEL ALLUVIUM**		
45								

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card ____

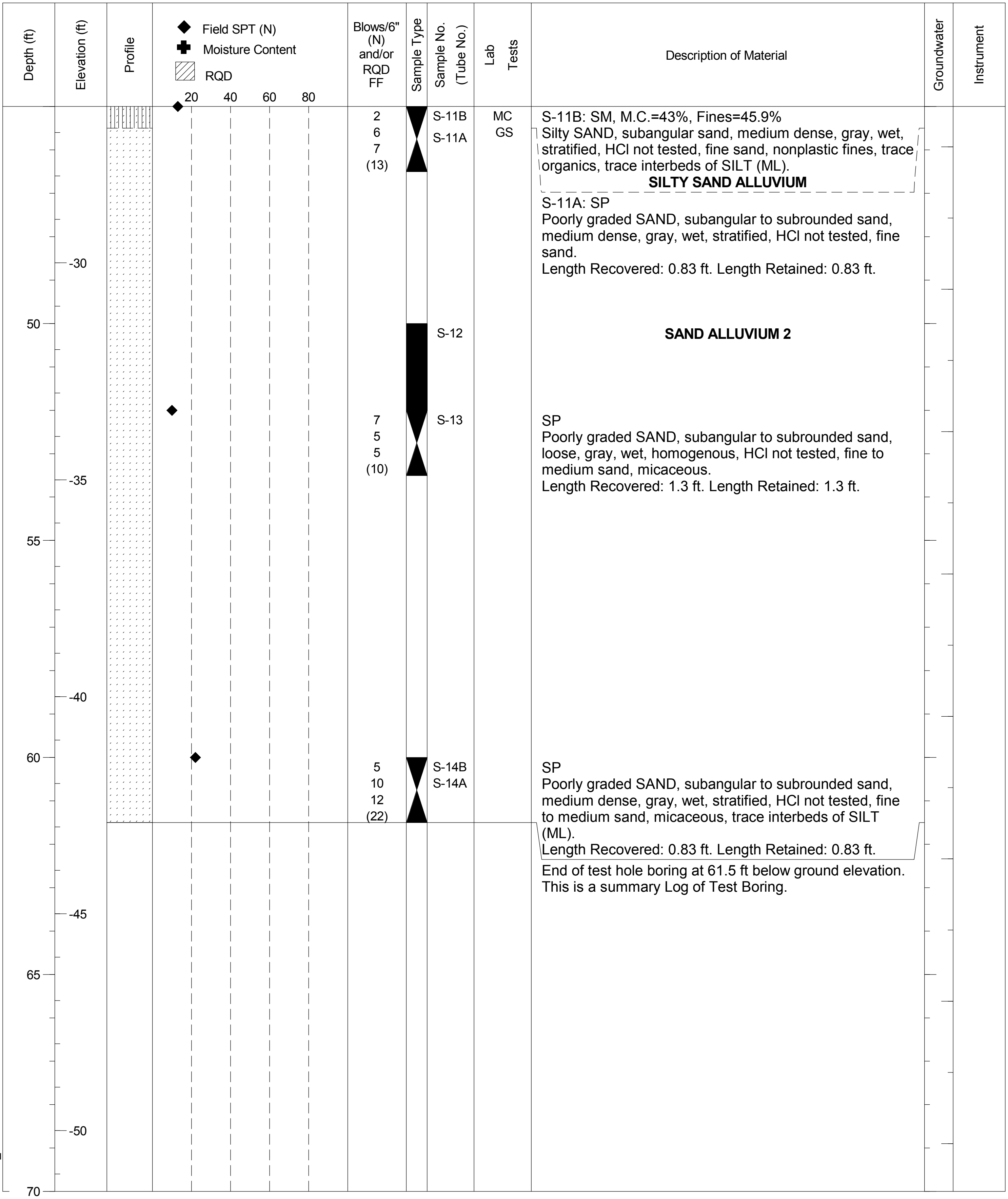
Job No. 24-1-04201-001 SR ____ Elevation 18.6 ft

HOLE No. B-4

Figure A4 Sheet 3 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic# ____



Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
2 6 7 (13)	S-11B S-11A	MC GS	S-11B: SM, M.C.=43%, Fines=45.9% Silty SAND, subangular sand, medium dense, gray, wet, stratified, HCl not tested, fine sand, nonplastic fines, trace organics, trace interbeds of SILT (ML). **SILTY SAND ALLUVIUM** S-11A: SP Poorly graded SAND, subangular to subrounded sand, medium dense, gray, wet, stratified, HCl not tested, fine sand. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.
	S-12		**SAND ALLUVIUM 2**
7 5 5 (10)	S-13		SP Poorly graded SAND, subangular to subrounded sand, loose, gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 1.3 ft. Length Retained: 1.3 ft.
5 10 12 (22)	S-14B S-14A		SP Poorly graded SAND, subangular to subrounded sand, medium dense, gray, wet, stratified, HCl not tested, fine to medium sand, micaceous, trace interbeds of SILT (ML). Length Recovered: 0.83 ft. Length Retained: 0.83 ft.

End of test hole boring at 61.5 ft below ground elevation.
This is a summary Log of Test Boring.

SOILA_FIG#SW 24-1-04201WSDOT.GPJ SOIL.GDT 5/29/18

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-5

Job No. 24-1-04201-001 SR Elevation 17.5 ft

Figure A5 Sheet 1 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Site Address Right-of-way, Hazel St

Inspector Hoda Soltani

Start April 24, 2018 Completion April 24, 2018 Well ID#

Equipment CME 850 Track Rig #7

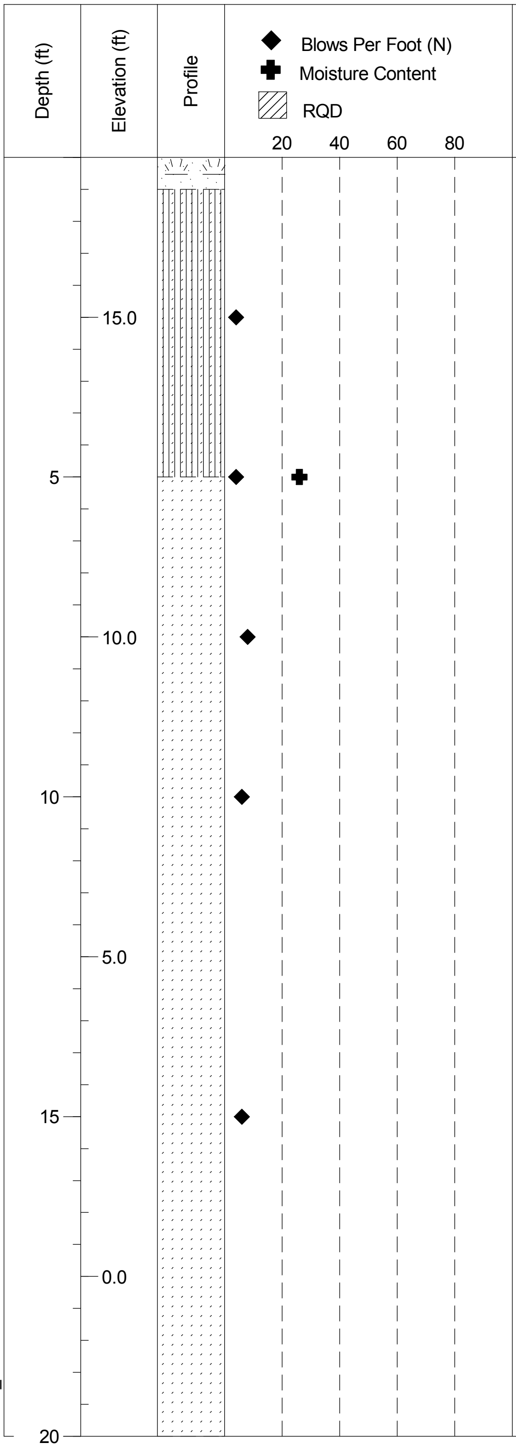
Station Offset Hole Dia (inches) 5

Method Mud Rotary

Northing 298337.1312 Easting 1029838.9087 Collected by

Datum WA83-SF, NAVD88

County Cowlitz Subsection NW1/4 of NW1/4 Section 2 Range 2W Township 7N



Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
0	17.5					Sod		
	15.0	1 2 2 (4)		S-1	VM	SM Silty SAND, subangular sand, very loose, gray-brown, wet, homogeneous, HCl not tested, fine sand, nonplastic fines. Length Recovered: 1 ft. Length Retained: 1 ft. **SILTY SAND ALLUVIUM**		
5		2 2 2 (4)		S-2	MC GS	SP, M.C.=26%, Fines=2.7% Poorly graded SAND, subangular sand, very loose, gray-brown, wet, homogeneous, HCl not tested, fine to medium sand. Length Recovered: 0.58 ft. Length Retained: 0.58 ft. **SAND ALLUVIUM 1**		
	10.0	3 3 5 (8)		S-3		SP Poorly graded SAND, subangular to subrounded sand, loose, gray-brown, wet, homogeneous, HCl not tested, fine to coarse sand. Length Recovered: 0.58 ft. Length Retained: 0.58 ft.		
10		2 2 4 (6)		S-4		SP Poorly graded SAND, subangular sand, loose, gray-brown, wet, homogeneous, HCl not tested, fine to medium sand. Length Recovered: 0.5 ft. Length Retained: 0.5 ft.		
	5.0							
15		2 3 3 (6)		S-5		SP Poorly graded SAND, subangular sand, loose, gray-brown, wet, homogeneous, HCl not tested, fine to coarse sand. Length Recovered: 1 ft. Length Retained: 1 ft.		
	0.0							
20								

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card ________

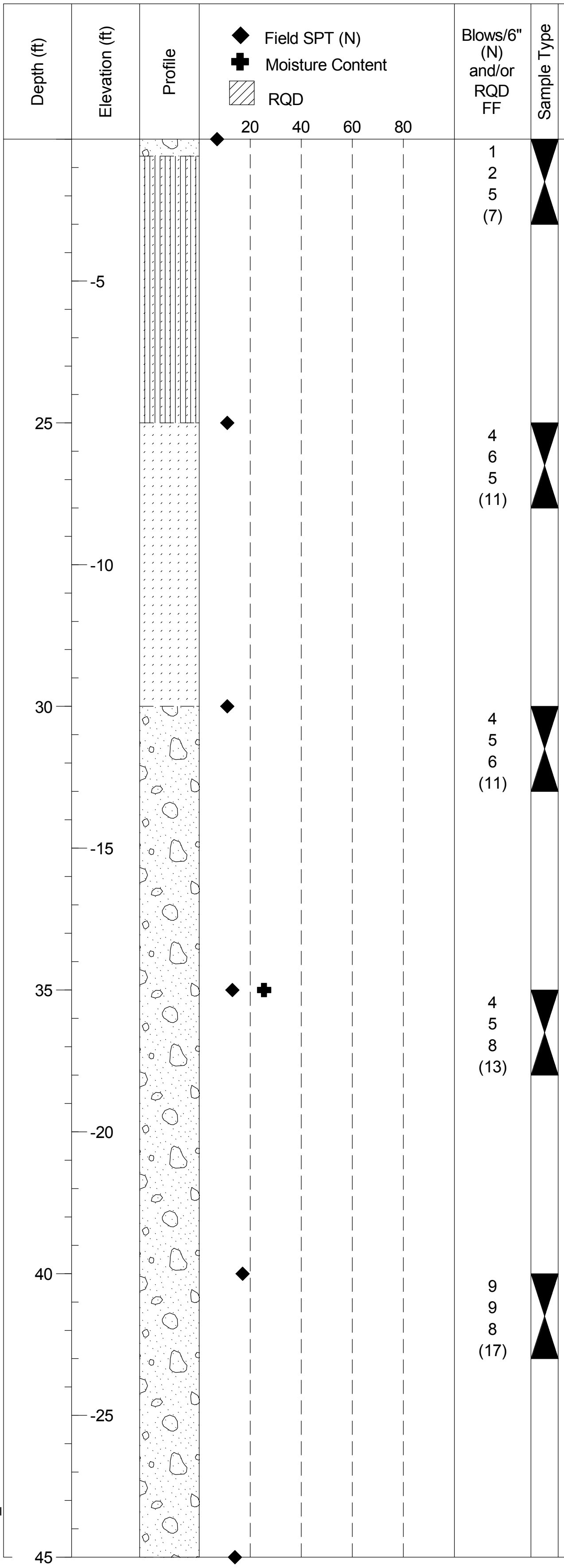
HOLE No. B-5

Job No. 24-1-04201-001 SR ________ Elevation 17.5 ft

Figure A5 Sheet 2 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic# ________



Depth (ft)	Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
	1 2 5 (7)	S-6B S-6A	VM VM	S-6B: SP Poorly graded SAND with gravel, subangular gravel, subangular sand, loose, gray, wet, HCl not tested, fine to coarse gravel, fine to coarse sand, thin interbed. S-6A: SM Silty SAND, subangular sand, loose, gray-brown, wet, stratified, HCl not tested, fine sand, nonplastic fines. Length Recovered: 0.58 ft. Length Retained: 0.58 ft. **SILTY SAND ALLUVIUM**
25	4 6 5 (11)	S-7	VM	SP Poorly graded SAND, subangular sand, medium dense, gray-brown, wet, homogeneous, HCl not tested, fine to medium sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft. **SAND ALLUVIUM 1**
30	4 5 6 (11)	S-8	VM	SP Poorly graded SAND with gravel, subangular gravel, subangular sand, medium dense, gray-brown, wet, homogeneous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.5 ft. Length Retained: 0.5 ft. **SAND WITH GRAVEL ALLUVIUM**
35	4 5 8 (13)	S-9	MC GS	SP, M.C.=25%, Fines=3.9% Poorly graded SAND with gravel, subangular to subrounded gravel, subangular sand, medium dense, gray-brown, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.91 ft. Length Retained: 0.91 ft.
40	9 9 8 (17)	S-10	VM	SP Poorly graded SAND with trace to few gravel, subangular gravel, subangular sand, dense, gray-brown, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.
45				

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-5

Job No. 24-1-04201-001 SR

Elevation 17.5 ft

Figure A5 Sheet 3 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

◆ Field SPT (N)
✚ Moisture Content
▨ RQD

Depth (ft)	Elevation (ft)	Profile	Field SPT (N) 20 40 60 80	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
			◆	6 7 7 (14)	⧗	S-11		SP Poorly graded SAND, subangular sand, medium dense, gray-brown, wet, stratified, HCl not tested, fine to coarse sand, trace to few coarse sand-sized pumice fragments. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
	-30							**SAND ALLUVIUM 2**		
50			◆	4 5 9 (14)	⧗	S-12		SP Poorly graded SAND, subangular sand, medium dense, gray-brown, wet, stratified, HCl not tested, fine to medium sand, trace interbeds of SILT (ML). Length Recovered: 1.5 ft. Length Retained: 1.5 ft.		
								End of test hole boring at 51.5 ft below ground elevation. This is a summary Log of Test Boring.		
	-35									
55										
	-40									
60										
	-45									
65										
	-50									
70										

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-6

Job No. 24-1-04201-001 SR Elevation 35.8 ft

Figure A6 Sheet 1 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Site Address Golf Course, 2222 S. River Rd

Inspector Hoda Soltani

Start April 16, 2018 Completion April 18, 2018 Well ID#

Equipment CME 850 Track Rig #7

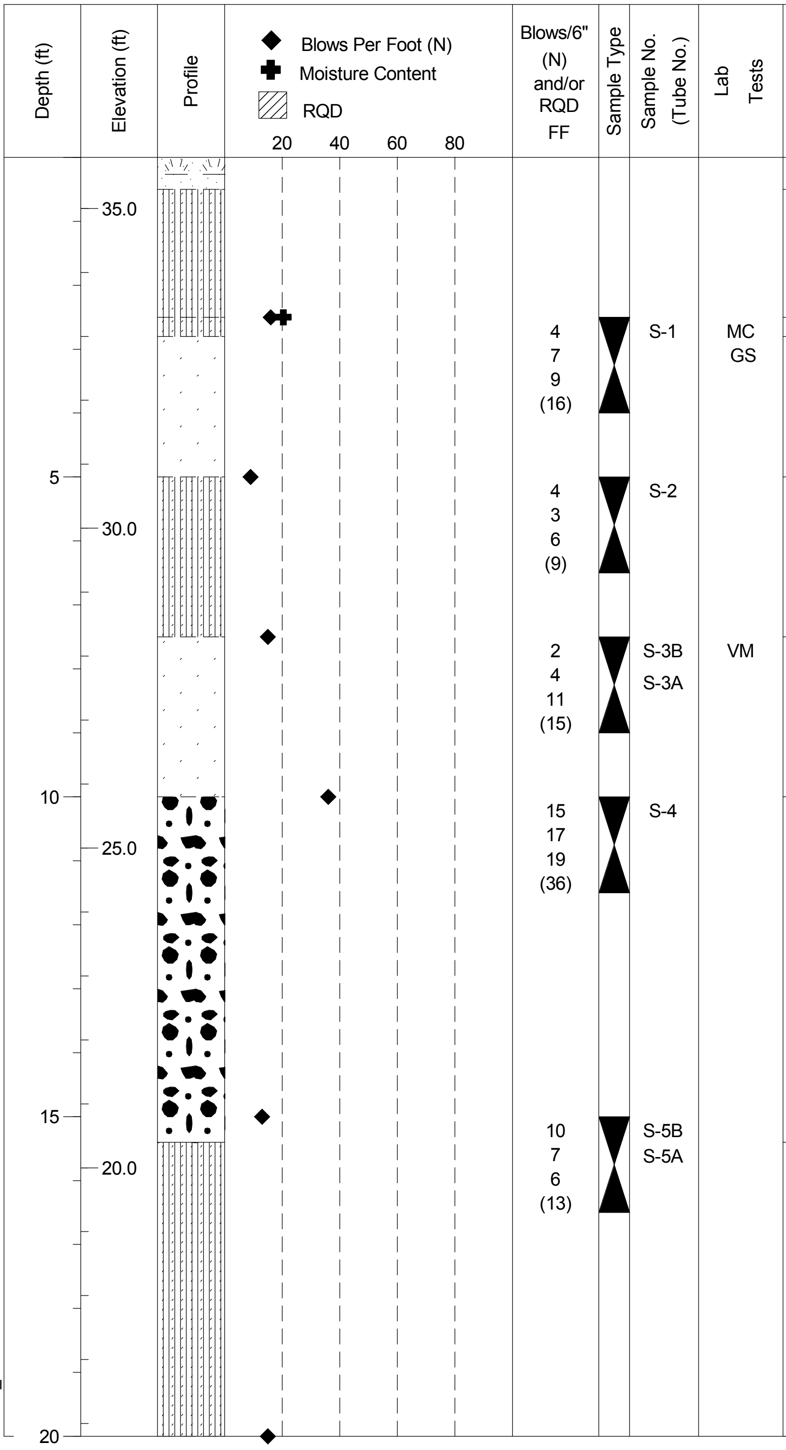
Station Offset Hole Dia (inches) 5

Method Mud Rotary

Northing 297888.5526 Easting 1029117.4462 Collected by

Datum WA83-SF, NAVD88

County Cowlitz Subsection NE1/4 of NE1/4 Section 3 Range 2W Township 7N



Description of Material | Groundwater | Instrument

Sod

SM
Silty SAND, dark gray, moist to wet, not sampled.

FILL

S-1B: SM, M.C.=20%, Fines=43.8%
Silty SAND, medium dense, dark gray, wet, disturbed, HCl not tested, fine to medium sand, low plasticity fines, trace organics and rootlets.

S-1A: ML
SILT, very stiff, dark gray mottled orange-brown, moist, disturbed, HCl not tested, fine sand, nonplastic to low plasticity, trace organic debris.
Length Recovered: 0.67 ft. Length Retained: 0.67 ft.

SM
Silty SAND, loose, dark gray, moist to wet, disturbed, HCl not tested, fine to medium sand, low plasticity fines, trace organics and rootlets.
Length Recovered: 1.33 ft. Length Retained: 1.33 ft.

S-3B: ML
SILT with trace sand and gravel, stiff, dark brown-gray, moist to wet, disturbed, HCl not tested, fine sand, low plasticity, trace organics and rootlets.
S-3A: Recovered coarse gravel fragment at bottom of SPT.
Length Recovered: 1 ft. Length Retained: 1 ft.

GP
Poorly graded GRAVEL with sand, subangular to subrounded gravel, subangular sand, dense, gray, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand, trace organics and roots.
Length Recovered: 2 ft. Length Retained: 1.5 ft.

S-5B: GP
Poorly graded GRAVEL with sand, subangular to subrounded gravel, subangular sand, dense, gray, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand, trace organics and roots.

S-5A: SM
Silty SAND, subangular sand, medium dense, brown, wet, stratified, HCl not tested, fine sand, nonplastic fines.
Length Recovered: 0.75 ft. Length Retained: 0.75 ft.

SILTY SAND ALLUVIUM

LOG OF TEST BORING

Start Card ______

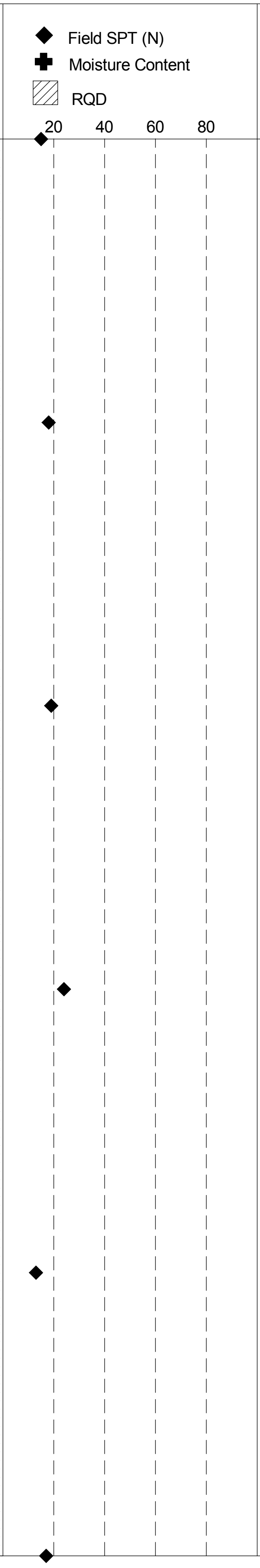
HOLE No. B-6

Job No. 24-1-04201-001 SR ______ Elevation 35.8 ft

Figure A6 Sheet 2 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic# ______



Depth (ft)	Elevation (ft)	Profile	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	15		7 8 7 (15)	⧗	S-6		SP Poorly graded SAND, subangular sand, medium dense, gray, wet, homogenous, HCl not tested, fine sand. Length Recovered: 1.16 ft. Length Retained: 1.16 ft.		
							SAND ALLUVIUM 1		
25	10		9 9 9 (18)	⧗	S-7		SP Poorly graded SAND, subangular sand, medium dense, gray, wet, homogenous, HCl not tested, fine to medium sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
30	5		8 9 10 (19)	⧗	S-8		SP Poorly graded SAND, subangular sand, medium dense, gray, wet, homogenous, HCl not tested, fine to medium sand. Length Recovered: 0.91 ft. Length Retained: 0.91 ft.		
35	0		9 11 13 (24)	⧗	S-9		SP Poorly graded SAND, subangular sand, medium dense, gray, wet, homogenous, HCl not tested, fine to coarse sand. Length Recovered: 1 ft. Length Retained: 1 ft.		
40	-5		6 6 7 (13)	⧗	S-10		SP Poorly graded SAND, subangular sand, medium dense, gray, wet, homogenous, HCl not tested, fine to medium sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.		
45									

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-6

Job No. 24-1-04201-001

SR

Elevation 35.8 ft

Figure A6 Sheet 3 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Depth (ft)	Elevation (ft)	Profile	◆ Field SPT (N) ✚ Moisture Content ▨ RQD 20 40 60 80	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	-10			9 8 9 (17)	⧗	S-11		SP Poorly graded SAND with trace gravel, subangular to subrounded gravel, subangular sand, medium dense, gray, wet, homogenous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft. **SAND WITH GRAVEL ALLUVIUM**		
								End of test hole boring at 46.5 ft below ground elevation. This is a summary Log of Test Boring.		
50	-15									
55	-20									
60	-25									
65	-30									
70										

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-7

Job No. 24-1-04201-001 SR

Elevation 36.8 ft

Figure A7 Sheet 1 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Site Address Golf Course, 2222 S. River Rd

Inspector Hoda Soltani

Start April 16, 2018 Completion April 16, 2018 Well ID#

Equipment CME 850 Track Rig #7

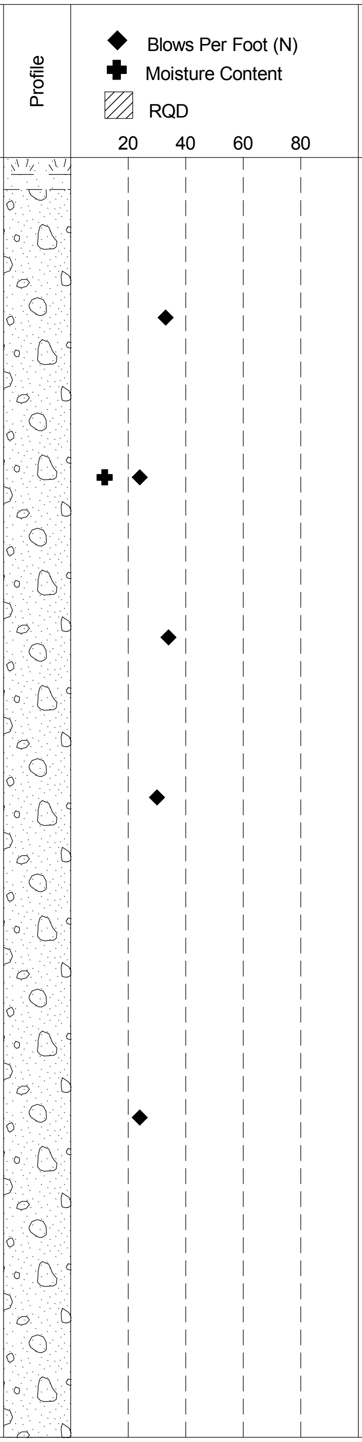
Station Offset Hole Dia (inches) 5

Method Mud Rotary

Northing 297836.3597 Easting 1029034.673 Collected by

Datum WA83-SF, NAVD88

County Cowlitz Subsection NE1/4 of NE1/4 Section 3 Range 2W Township 7N



Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
						Sod		
	35.0					SP Poorly graded SAND with gravel, gray-brown, moist, not sampled. **FILL**		
		16 16 17 (33)		S-1	VM	SP Poorly graded SAND with gravel, subangular gravel, subangular sand, dense, gray-brown, moist, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 1 ft. Length Retained: 1 ft.		
5		10 11 13 (24)		S-2	MC GS	SP, M.C.=12%, Fines=4.9% Poorly graded SAND with gravel, subangular to subrounded gravel, subangular sand, medium dense, dark gray and white, moist, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.9 ft. Length Retained: 0.9 ft.		
	30.0	12 15 19 (34)		S-3		SP Poorly graded SAND with gravel, subangular to subrounded gravel, subangular sand, dense, gray, moist, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.		
10		17 15 15 (30)		S-4		SP Poorly graded SAND with gravel, subangular to subrounded gravel, subangular sand, dense, gray, moist, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 1 ft. Length Retained: 1 ft.		
	25.0							
15		15 11 13 (24)		S-5		SP Poorly graded SAND with gravel, subangular to rounded gravel, subangular sand, medium dense, gray, moist, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
	20.0							
20								

SOILA_FIG#SW 24-1-04201WSDOT.GPJ SOIL.GDT 5/29/18

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-7

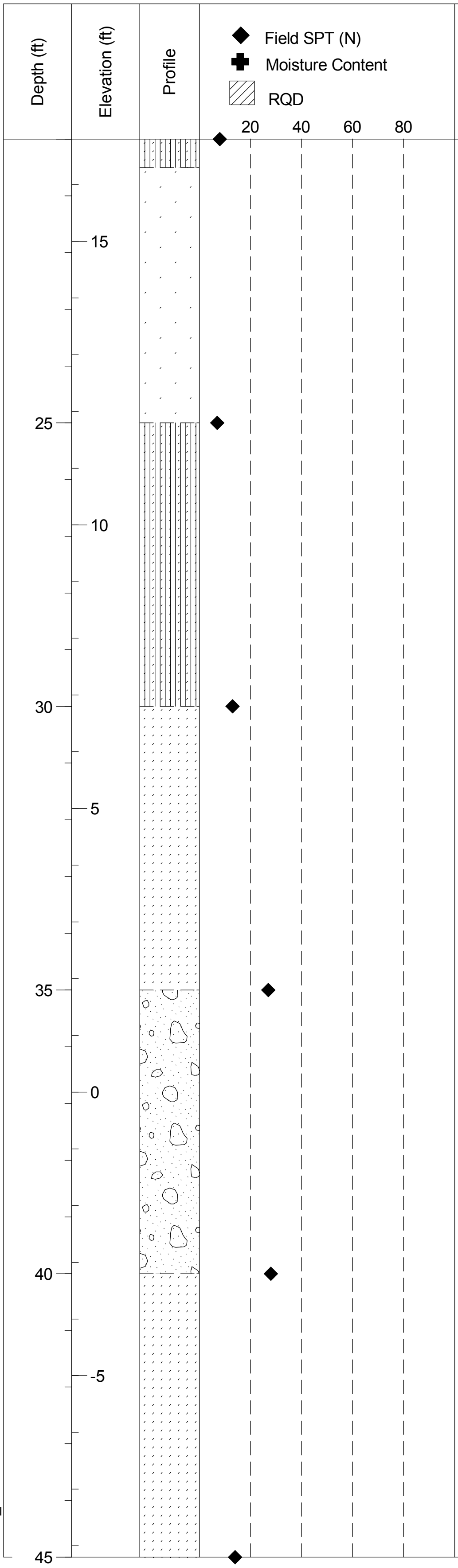
Job No. 24-1-04201-001 SR

Elevation 36.8 ft

Figure A7 Sheet 2 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#



Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
6 3 5 (8)	S-6B S-6A	VM VM	S-6B: SM Silty SAND, subangular sand, loose, dark gray, wet, stratified, HCl not tested, fine sand, nonplastic fines, slight iron oxidation and staining, few interbeds of SILT (ML). S-6A: ML SILT with trace sand, loose, dark gray, wet, stratified, HCl not tested, fine sand, nonplastic, slight iron oxidation and staining. Length Recovered: 1 ft. Length Retained: 1 ft. **SILT ALLUVIUM**
3 3 4 (7)	S-7B S-7A	VM VM	S-7B and S-7A: SM Silty SAND, subangular sand, loose, gray and red-yellow, wet, stratified, HCl not tested, fine sand, nonplastic fines. Length Recovered: 1.3 ft. Length Retained: 1.3 ft.
	S-8		**SILTY SAND ALLUVIUM**
6 6 7 (13)	S-9		SP Poorly graded SAND, subangular to subrounded sand, medium dense, gray, wet, homogeneous, HCl not tested, fine to coarse sand, trace coarse sand-sized pumice fragments. Length Recovered: 0.83 ft. Length Retained: 0.83 ft. **SAND ALLUVIUM 1**
7 11 16 (27)	S-10		SP Poorly graded SAND with trace gravel, subangular to subrounded gravel, subangular sand, dense, dark gray, wet, homogeneous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 1 ft. Length Retained: 1 ft. **SAND WITH GRAVEL ALLUVIUM**
11 15 13 (28)	S-11		SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, wet, homogeneous, HCl not tested, fine to coarse sand. Length Recovered: 1 ft. Length Retained: 1 ft. **SAND ALLUVIUM 1**

Groundwater

Instrument

LOG OF TEST BORING

Start Card

HOLE No. B-7

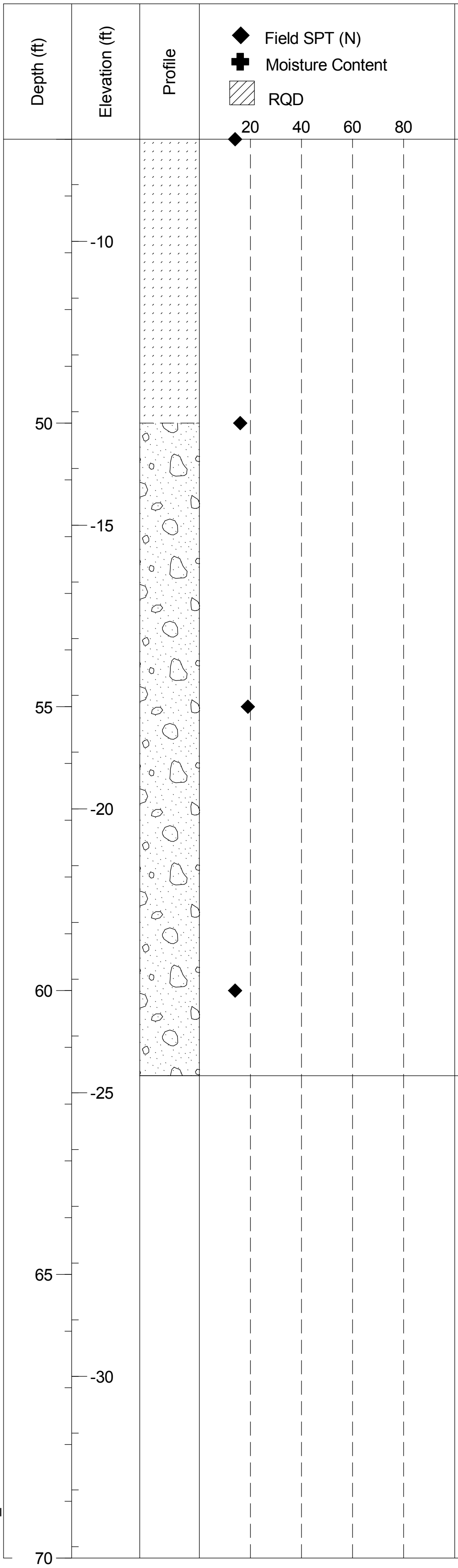
Job No. 24-1-04201-001 SR

Elevation 36.8 ft

Figure A7 Sheet 3 of 3

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

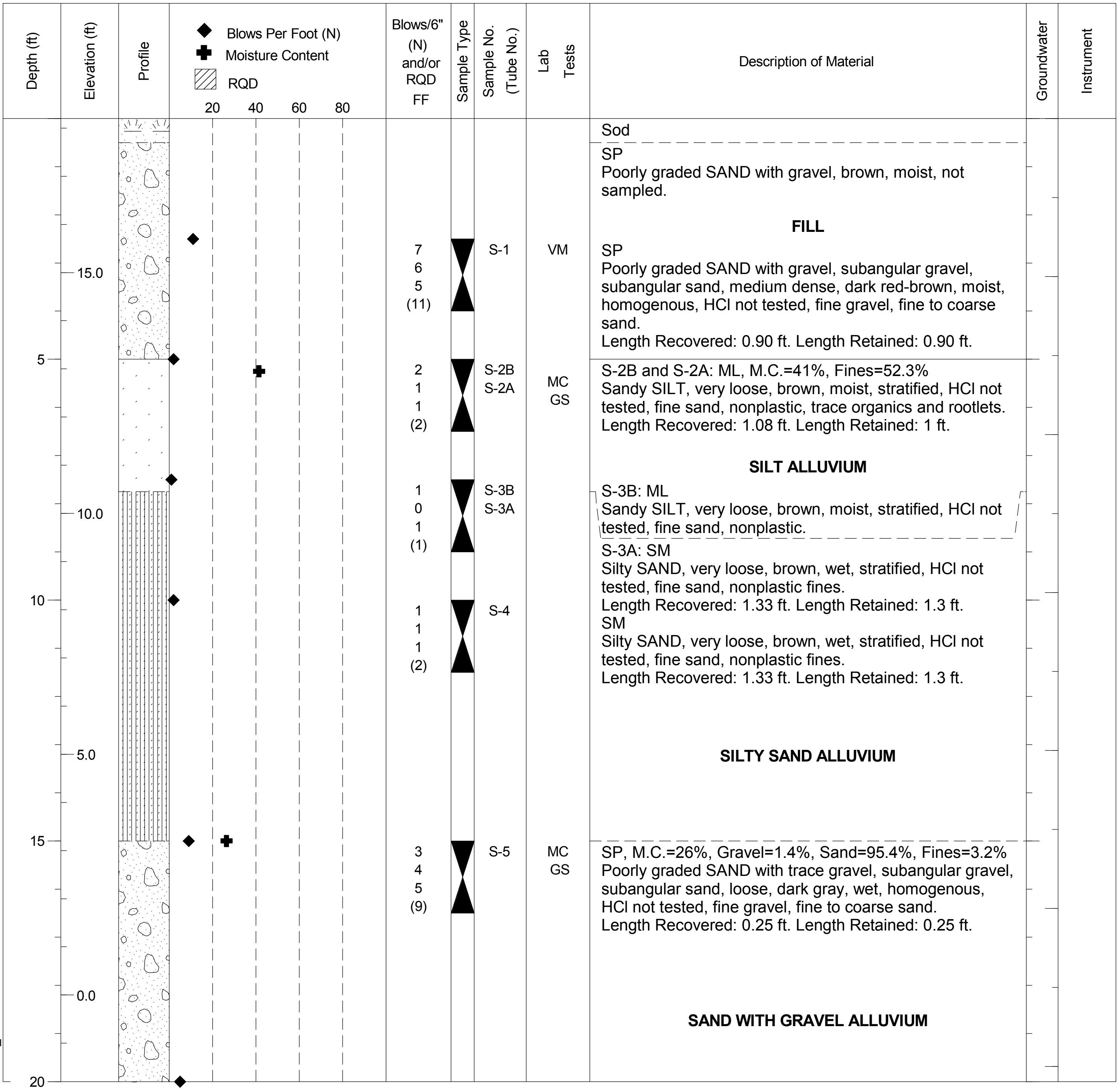


Depth (ft)	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	7 7 7 (14)		S-12		SP Poorly graded SAND, subangular to subrounded sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to coarse sand. Length Recovered: 0.67 ft. Length Retained: 0.67 ft.		
					SAND ALLUVIUM 1		
50	7 7 9 (16)		S-13		SP Poorly graded SAND with trace gravel, subangular to subrounded gravel, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.83 ft. Length Retained: 0.83 ft.		
					SAND WITH GRAVEL ALLUVIUM		
55	12 11 8 (19)		S-14		SP Poorly graded SAND with trace gravel, subangular to rounded gravel, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 1 ft. Length Retained: 1 ft.		
60	13 6 8 (14)		S-15		SP Poorly graded SAND with trace gravel, subangular to rounded gravel, subangular sand, medium dense, dark gray, wet, homogeneous, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.66 ft. Length Retained: 0.66 ft.		
					End of test hole boring at 61.5 ft below ground elevation. This is a summary Log of Test Boring.		

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card
HOLE No. B-8
Job No. 24-1-04201-001
SR
Elevation 18.2 ft
Figure A8 Sheet 1 of 6
Project South Kelso Railroad Grade Separation Project
Driller Western States
Lic#
Site Address Grass Field, 2002 S. River Rd
Inspector Hoda Soltani
Start April 18, 2018
Completion April 19, 2018
Well ID#
Equipment CME 850 Track Rig #7
Station
Offset
Hole Dia (inches) 5
Method Mud Rotary
Northing 297776.4917
Easting 1028368.2008
Collected by
Datum WA83-SF, NAVD88
County Cowlitz
Subsection NE1/4 of NE1/4
Section 3
Range 2W
Township 7N



Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
			Sod
			SP Poorly graded SAND with gravel, brown, moist, not sampled.
			FILL
7 6 5 (11)	S-1	VM	SP Poorly graded SAND with gravel, subangular gravel, subangular sand, medium dense, dark red-brown, moist, homogenous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.90 ft. Length Retained: 0.90 ft.
2 1 1 (2)	S-2B S-2A	MC GS	S-2B and S-2A: ML, M.C.=41%, Fines=52.3% Sandy SILT, very loose, brown, moist, stratified, HCl not tested, fine sand, nonplastic, trace organics and rootlets. Length Recovered: 1.08 ft. Length Retained: 1 ft.
			SILT ALLUVIUM
1 0 1 (1)	S-3B S-3A		S-3B: ML Sandy SILT, very loose, brown, moist, stratified, HCl not tested, fine sand, nonplastic. S-3A: SM Silty SAND, very loose, brown, wet, stratified, HCl not tested, fine sand, nonplastic fines. Length Recovered: 1.33 ft. Length Retained: 1.3 ft.
1 1 1 (2)	S-4		SM Silty SAND, very loose, brown, wet, stratified, HCl not tested, fine sand, nonplastic fines. Length Recovered: 1.33 ft. Length Retained: 1.3 ft.
			SILTY SAND ALLUVIUM
3 4 5 (9)	S-5	MC GS	SP, M.C.=26%, Gravel=1.4%, Sand=95.4%, Fines=3.2% Poorly graded SAND with trace gravel, subangular gravel, subangular sand, loose, dark gray, wet, homogenous, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.25 ft. Length Retained: 0.25 ft.
			SAND WITH GRAVEL ALLUVIUM

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

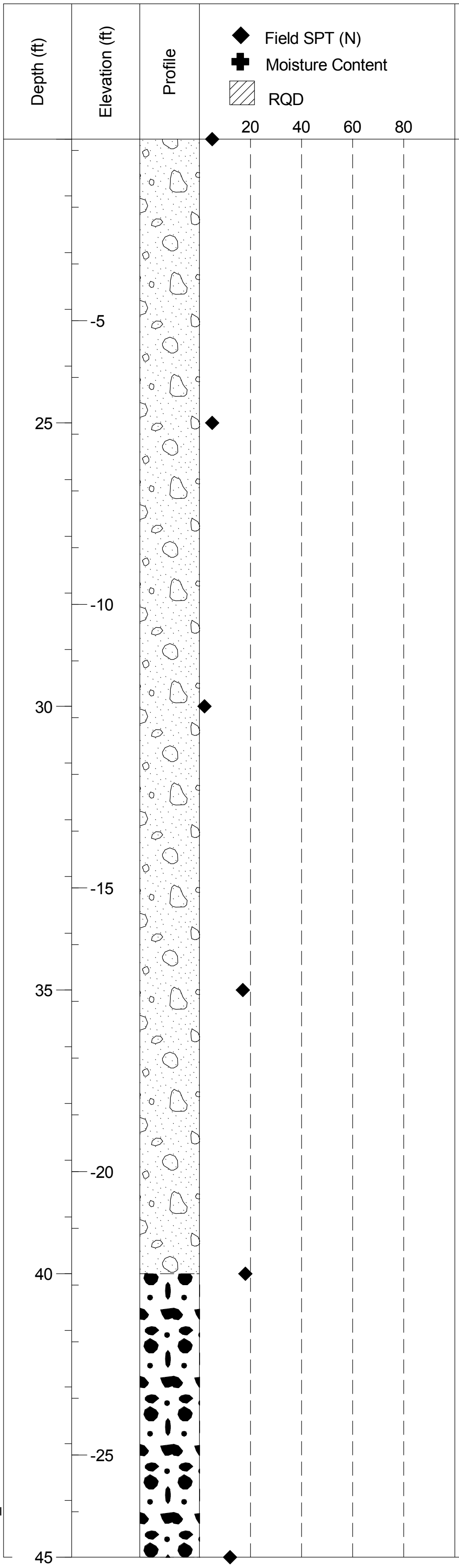
HOLE No. B-8

Job No. 24-1-04201-001 SR Elevation 18.2 ft

Figure A8 Sheet 2 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#



Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
2 2 3 (5)		S-6		SP Poorly graded SAND with trace gravel, subangular to subrounded gravel, subangular sand, loose, dark gray, wet, stratified, HCl not tested, fine gravel, fine to coarse sand, trace organics and wood debris. Length Recovered: 0.83 ft. Length Retained: 0.80 ft.		
				SAND WITH GRAVEL ALLUVIUM		
4 3 2 (5)		S-7		SP Poorly graded SAND with gravel, subangular to rounded gravel, subangular sand, loose, dark gray, wet, stratified, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.25 ft. Length Retained: 0.25 ft.		
0 1 1 (2)		S-8		SP Poorly graded SAND with trace gravel, subangular to subrounded gravel, subangular sand, very loose, dark gray, wet, stratified, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.25 ft. Length Retained: 0.25 ft.		
9 9 8 (17)		S-9	VM	SP Poorly graded SAND with gravel, subangular gravel, subangular sand, medium dense, dark gray, wet, stratified, HCl not tested, fine gravel, fine to coarse sand. Length Recovered: 0.25 ft. Length Retained: 0.25 ft.		
9 10 8 (18)		S-10	VM	GP Poorly graded GRAVEL with sand, subangular gravel, subangular sand, medium dense, dark gray, wet, stratified, HCl not tested, fine to coarse gravel, fine to coarse sand. Length Recovered: 0.25 ft. Length Retained: 0.25 ft.		
				GRAVEL ALLUVIUM		

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card ______

Job No. 24-1-04201-001 SR ______ Elevation 18.2 ft

HOLE No. B-8

Figure A8 Sheet 3 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic# ______

Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
		0 5 7 (12)	S-11		ML Sandy SILT, subangular to subrounded sand, medium dense, olive-gray, wet, stratified, HCl not tested, fine to medium sand, nonplastic, few organics, trace to few interbeds of Silty SAND (SM). Length Recovered: 1.6 ft. Length Retained: 1.6 ft.
	-30				**SILT ALLUVIUM**
50		7 6 7 (13)	S-12B S-12A		S-12B: SM Silty SAND, subangular sand, medium dense, brown, wet, stratified, HCl not tested, fine sand, nonplastic fines. S-12A: ML Sandy SILT, subangular to subrounded sand, medium dense, olive-gray, wet, stratified, HCl not tested, fine sand, nonplastic fines. Length Recovered: 1.58 ft. Length Retained: 1.5 ft.
			S-13		
	-35				
55		5 4 4 (8)	S-14B S-14A		S-14B: ML SILT with sand, subangular to subrounded sand, loose, light gray, wet, stratified, HCl not tested, fine to coarse sand, nonplastic, trace organics. S-14A: ML SILT, loose, light gray, wet, stratified, HCl not tested, trace fine sand, nonplastic. Length Recovered: 1.5 ft. Length Retained: 1.5 ft.
	-40				
60		11 11 17 (22)	S-15		SM Silty SAND, subangular sand, medium dense, light gray, wet, stratified, HCl not tested, fine sand, nonplastic fines, ash layer. Length Recovered: 1 ft. Length Retained: 1 ft.
	-45				**SILTY SAND ALLUVIUM**
65		7 7 9 (16)	S-16		SM Silty SAND, subangular to subrounded sand, medium dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, nonplastic fines. Length Recovered: 1 ft. Length Retained: 1 ft.
	-50				
70					

◆ Field SPT (N) ✚ Moisture Content ▨ RQD

LOG OF TEST BORING

Start Card ______

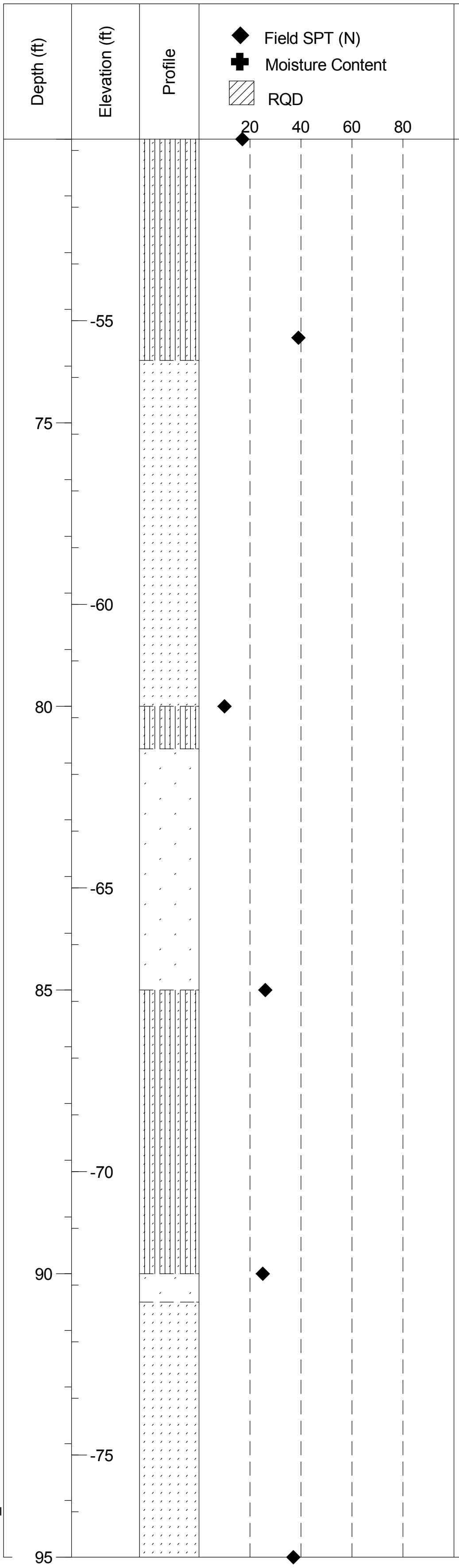
HOLE No. B-8

Job No. 24-1-04201-001 SR ______ Elevation 18.2 ft

Figure A8 Sheet 4 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic# ______



Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
4 6 11 (17)	S-17B S-17A		S-17B and S-17A: SM Silty SAND, subangular sand, medium dense, light gray, wet, homogeneous, HCl not tested, fine sand, nonplastic fines. Length Recovered: 1 ft. Length Retained: 1 ft.
	S-18		**SILTY SAND ALLUVIUM**
12 17 22 (39)	S-19C S-19B S-19A	VM VM	S-19C: SM Silty SAND, subangular to subrounded sand, dense, dark gray, wet, stratified, fine to medium sand, nonplastic fines. S-19B and S-19A: SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, moist to wet, stratified, HCl not tested, fine sand. Length Recovered: 1.5 ft. Length Retained: 1.5 ft. **SAND ALLUVIUM 2**
0 3 7 (10)	S-20B S-20A		S-20B: SM Silty SAND, subangular sand, loose, dark gray, wet, stratified, HCl not tested, fine sand, nonplastic fines, micaceous. S-20A: ML Sandy SILT, subangular sand, loose, dark gray, wet, stratified, HCl not tested, fine sand, nonplastic, micaceous. Length Recovered: 1.6 ft. Length Retained: 1.5 ft. **SILT ALLUVIUM**
9 13 13 (26)	S-21		SM Silty SAND, subangular sand, dense, dark gray, wet, stratified, HCL not tested, fine to medium sand, nonplastic fines, micaceous. Length Recovered: 1 ft. Length Retained: 1 ft. **SILTY SAND ALLUVIUM**
4 11 14 (25)	S-22B S-22A	VM	S-22B: ML SILT with sand, subangular sand, dense, dark gray, moist, stratified, HCl not tested, fine sand, nonplastic, trace organics, micaceous. S-22A: SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, moist to wet, homogeneous, HCl not tested, fine to medium sand. Length Recovered: 1 ft. Length Retained: 1 ft. **SAND ALLUVIUM 2**

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-8

Job No. 24-1-04201-001 SR Elevation 18.2 ft

Figure A8 Sheet 5 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic#

Depth (ft)	Elevation (ft)	Profile	◆ Field SPT (N) ✚ Moisture Content ▨ RQD 20 40 60 80	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				13 16 21 (37)		S-23	VM	SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 1 ft. Length Retained: 1 ft.		
	-80							**SAND ALLUVIUM 2**		
100				11 17 21 (38)		S-24	MC GS	SP, M.C.=24%, Fines=3.4% Poorly graded SAND, subangular to subrounded sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 1 ft. Length Retained: 1 ft.		
	-85									
105				13 19 28 (48)		S-25		SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, wet, homogenous, HCl not tested, fine to coarse sand, trace to few coarse sand-sized pumice fragments. Length Recovered: 1 ft. Length Retained: 1 ft.		
	-90									
110				19 18 19 (37)		S-26		SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 1 ft. Length Retained: 1 ft.		
	-95									
115				14 18 22 (30)		S-27		SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 0.75 ft. Length Retained: 0.75 ft.		
	-100									
120										

LOG OF TEST BORING

Start Card ______

HOLE No. B-8

Job No. 24-1-04201-001 SR ______ Elevation 18.2 ft

Figure A8 Sheet 6 of 6

Project South Kelso Railroad Grade Separation Project

Driller Western States Lic# ______

Depth (ft)	Elevation (ft)	Profile	◆ Field SPT (N) ✚ Moisture Content ▨ RQD 20 40 60 80	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				20 20 20 (40)	⧗	S-28		SP Poorly graded SAND, subangular to subrounded sand, dense, dark gray, wet, homogenous, HCl not tested, fine to medium sand, micaceous. Length Recovered: 1.25 ft. Length Retained: 1.25 ft. **SAND ALLUVIUM 2**		
								End of test hole boring at 121.5 ft below ground elevation. This is a summary Log of Test Boring.		
	-105									
125										
	-110									
130										
	-115									
135										
	-120									
140										
	-125									
145										

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card

HOLE No. B-9

Job No. 24-1-04201-001
SR
Elevation 20.1 ft

Figure A9 Sheet 1 of 1

Project South Kelso Railroad Grade Separation Project
Driller Western States Lic#

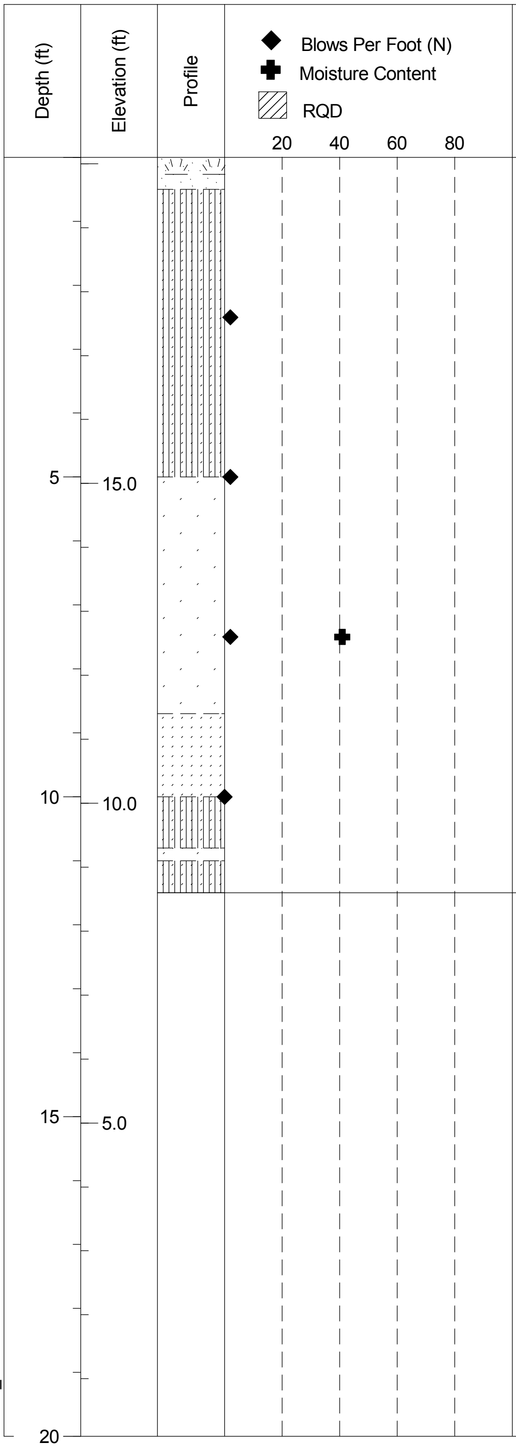
Site Address Grass Field, between Douglas St and Hazel St
Inspector Hoda Soltani

Start April 24, 2018
Completion April 24, 2018
Well ID#
Equipment CME 850 Track Rig #7

Station
Offset
Hole Dia (inches) 8
Method Hollow-stem auger

Northing 298057.814
Easting 1030144.9346
Collected by
Datum WA83-SF, NAVD88

County Cowlitz
Subsection NW1/4 of NW1/4
Section 2
Range 2W
Township 7N



Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				Sod		
				SM Silty SAND, brown, moist, not sampled. **SILTY SAND ALLUVIUM**		
1 1 1 (2)	⧗	S-1	VM	SM Silty SAND, subangular to subrounded sand, very loose, brown, moist, homogenous, HCl not tested, fine sand, nonplastic fines, trace organics and roots. Length Recovered: 1.25 ft. Length Retained: 1.25 ft.		
0 1 1 (2)	⧗	S-2		ML SILT with Sand, very loose, brown, moist, homogenous, HCl not tested, fine sand, nonplastic. Length Recovered: 1.25 ft. Length Retained: 1.25 ft. **SILT ALLUVIUM**	during drilling	
1 1 1 (2)	⧗	S-3C S-3B S-3A	MC GS	S-3C and S-3B: ML, M.C.=41%, Fines=79.1% SILT with Sand, very loose, brown, moist to wet, stratified, HCl not tested, fine sand, nonplastic. S-3A: SP Poorly graded SAND, subangular to subrounded sand, very loose, gray-brown, wet, stratified, HCl not tested, fine sand. Length Recovered: 1.3 ft. Length Retained: 1.3 ft. **SAND ALLUVIUM 1**	▽	
0 0 0 (0)	⧗	S-4C S-4B S-4A		S-4C: SM Silty SAND, subangular sand, very loose, brown, wet, stratified, HCl not tested, fine sand, nonplastic fines. **SILTY SAND ALLUVIUM** S-4B: ML SILT with Sand, very loose, brown, wet, stratified, HCl not tested, fine sand, nonplastic, SILT interbed. S-4A: SM Silty SAND, subangular sand, very loose, brown, wet, stratified, HCl not tested, fine sand, nonplastic fines. Length Recovered: 2 ft. Length Retained: 1.5 ft.		

End of test hole boring at 11.5 ft below ground elevation.
This is a summary Log of Test Boring.

DCP TEST DATA

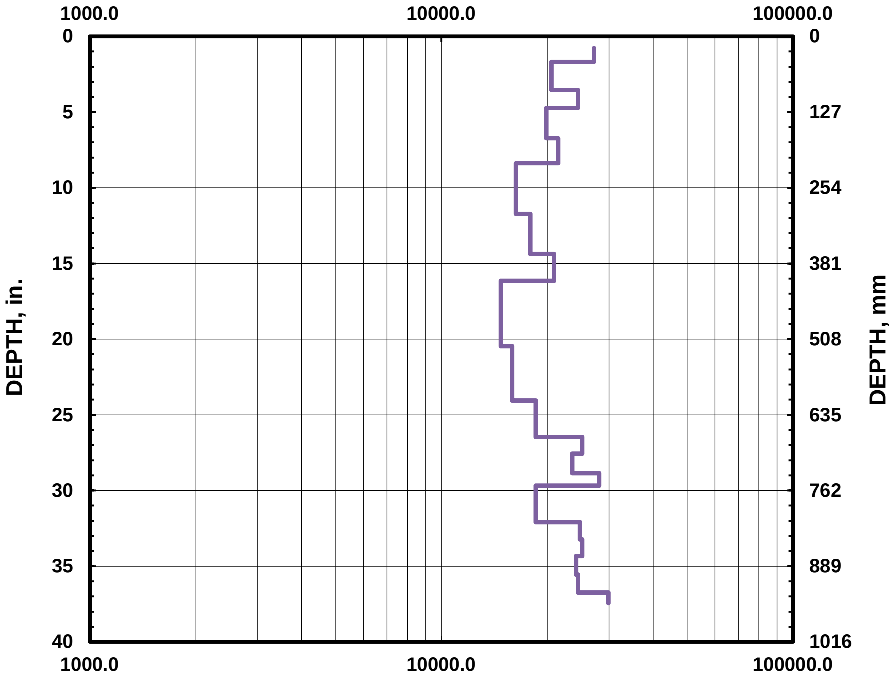
Project: *S Kelso Grade Separation*
Date: *24-Apr-18*
Location: *DCP-1*
Soil Type(s): *SP*

Hammer
○ 10.1 lbs.
● 17.6 lbs.
○ Both hammers used

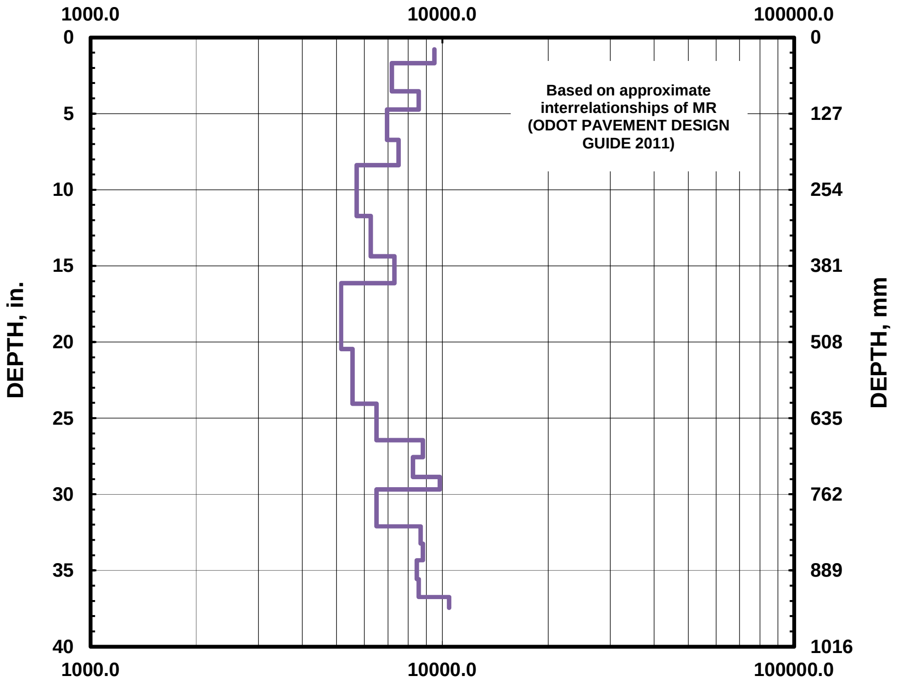
○ CH
○ CL
● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	20	1
5	43	1
5	90	1
5	120	1
5	171	1
5	213	1
5	298	1
5	365	1
5	410	1
5	520	1
5	611	1
5	672	1
5	700	1
5	733	1
5	754	1
5	815	1
5	844	1
5	872	1
5	903	1
5	933	1
5	951	1

SUBGRADE MODULUS (MR) psi



CORRECTED SUBGRADE MODULUS (MR) psi



Based on approximate interrelationships of MR (ODOT PAVEMENT DESIGN GUIDE 2011)

FIG. A12

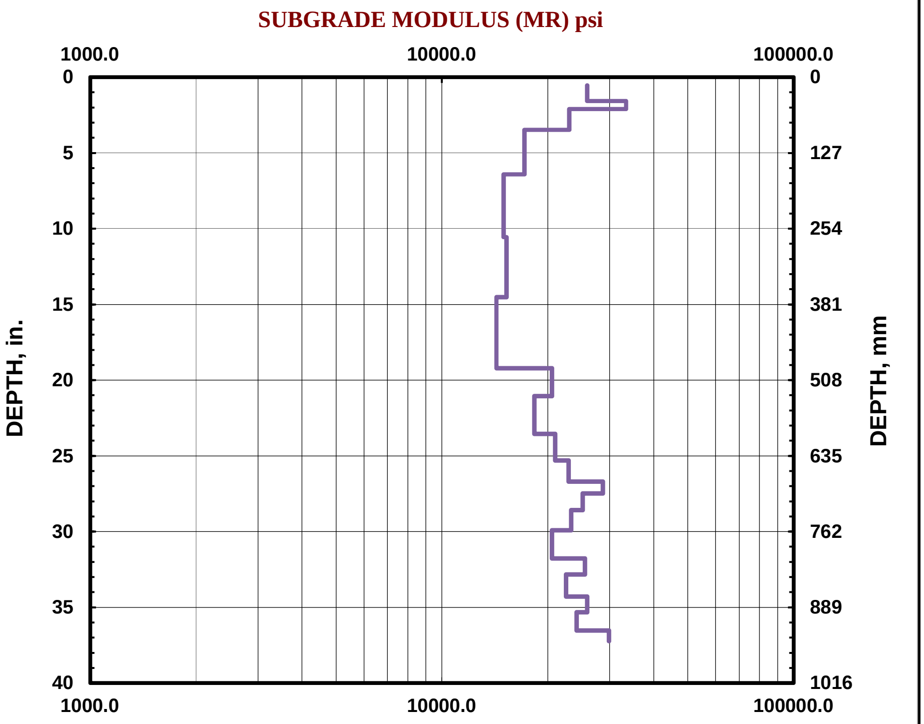
DCP TEST DATA

Project: *S Kelso Grade Separation*
Date: *24-Apr-18*
Location: *DCP-2*
Soil Type(s): *SP*

Hammer
○ 10.1 lbs.
● 17.6 lbs.
○ Both hammers used

○ CH
○ CL
● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	14	1
5	40	1
5	53.5	1
5	88.5	1
5	163	1
5	268.5	1
5	369	1
5	488	1
5	535	1
5	598	1
5	642.5	1
5	678	1
5	698	1
5	726	1
5	760	1
5	807	1
5	834	1
5	871	1
5	897	1
5	928	1
5	946	1



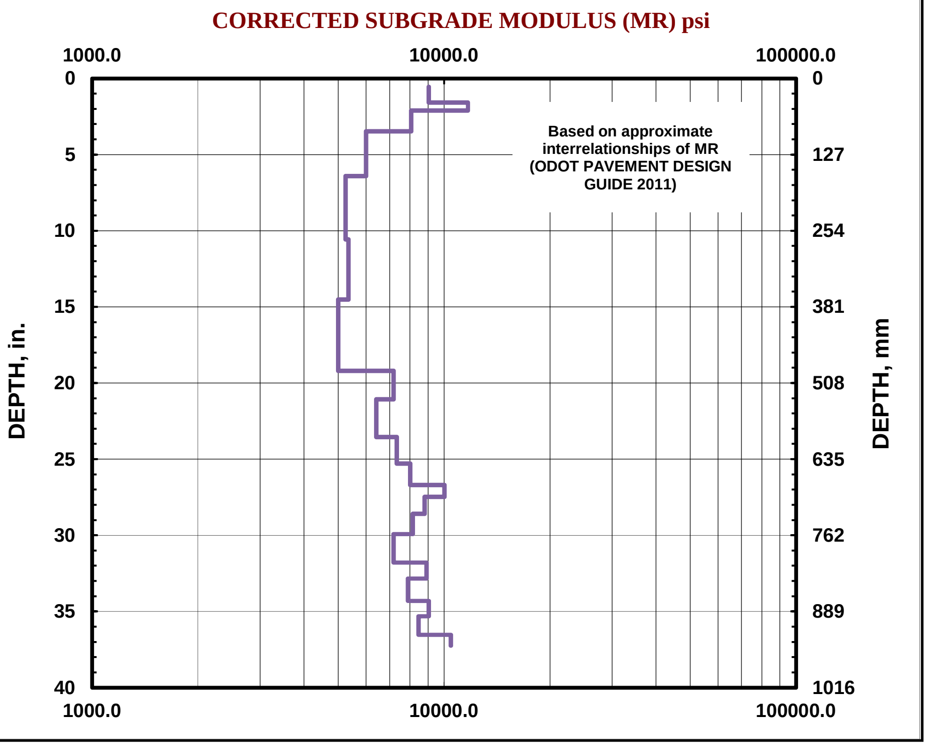


FIG. A13

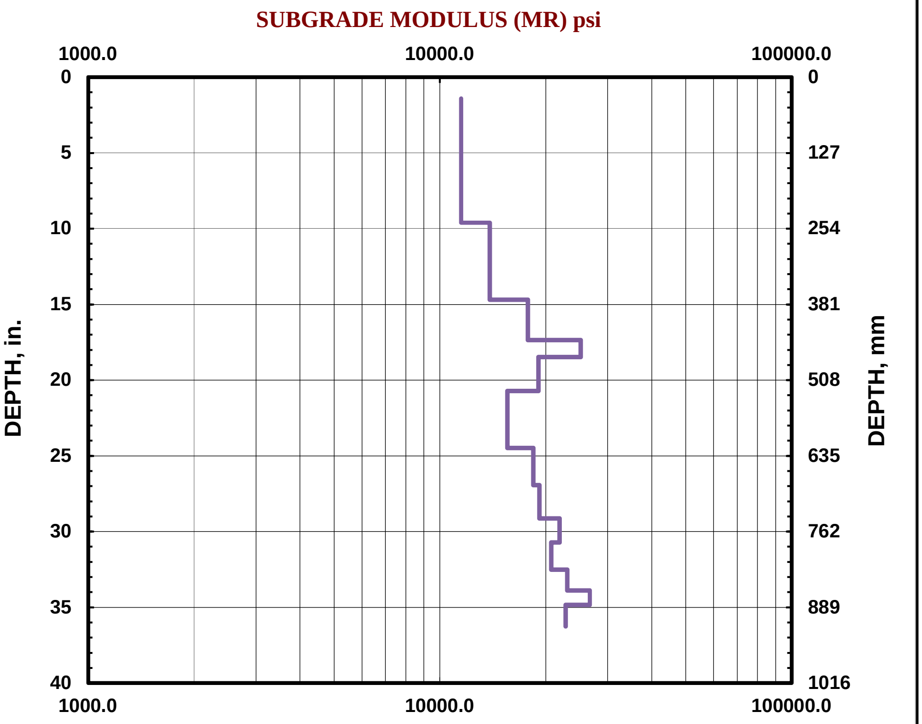
DCP TEST DATA

Project: *S Kelso Grade Separation*
Date: *24-Apr-18*
Location: *DCP-3*
Soil Type(s): *SP*

Hammer
○ 10.1 lbs.
● 17.6 lbs.
○ Both hammers used

○ CH
○ CL
● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	36	1
5	244	1
5	373	1
5	441	1
5	469	1
5	526	1
5	622	1
5	684	1
5	740	1
5	780	1
5	826	1
5	861	1
5	885	1
5	921	1



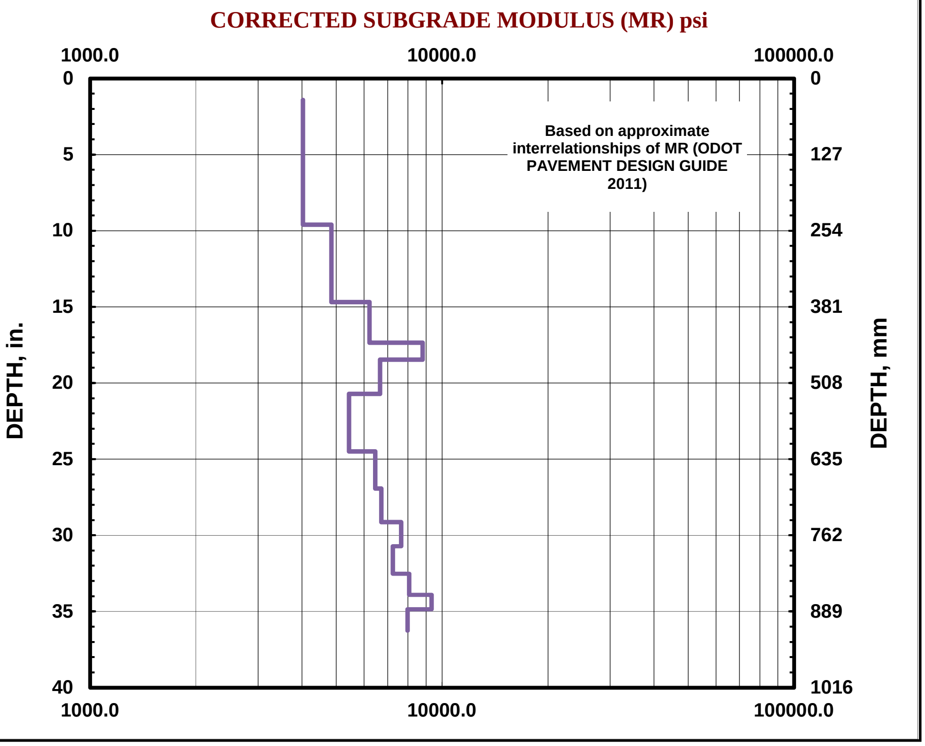


FIG. A14

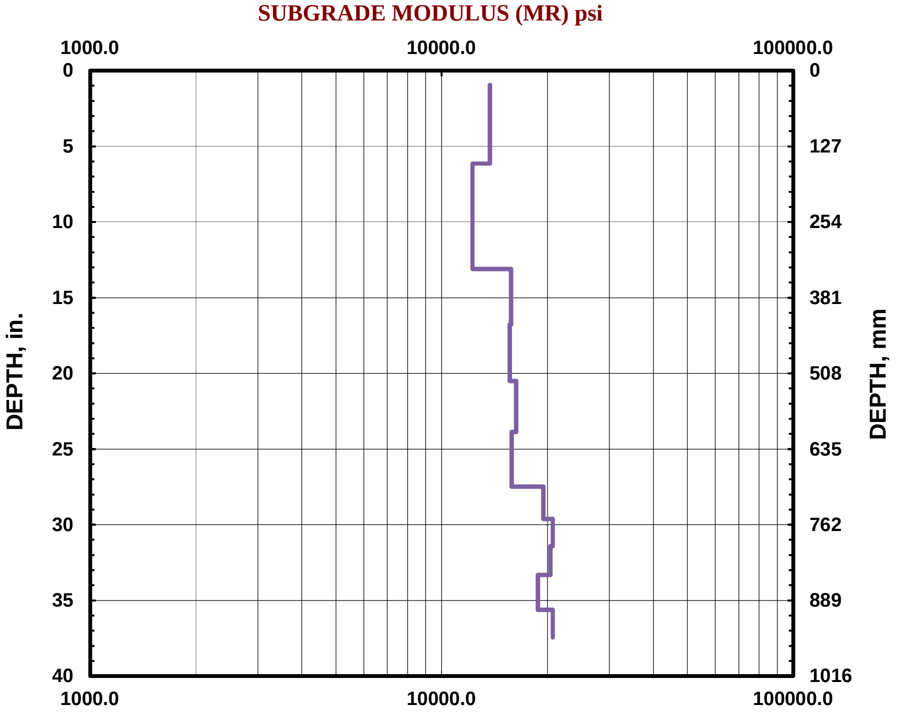
DCP TEST DATA

Project: *S Kelso Grade Separation*
Location: *DCP-4*
Date: *24-Apr-18*
Soil Type(s): *SP*

Hammer
○ 10.1 lbs.
● 17.6 lbs.
○ Both hammers used

○ CH
○ CL
● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	24	1
5	156	1
5	333	1
5	426	1
5	521	1
5	606	1
5	698	1
5	752	1
5	798	1
5	846	1
5	905	1
5	951	1



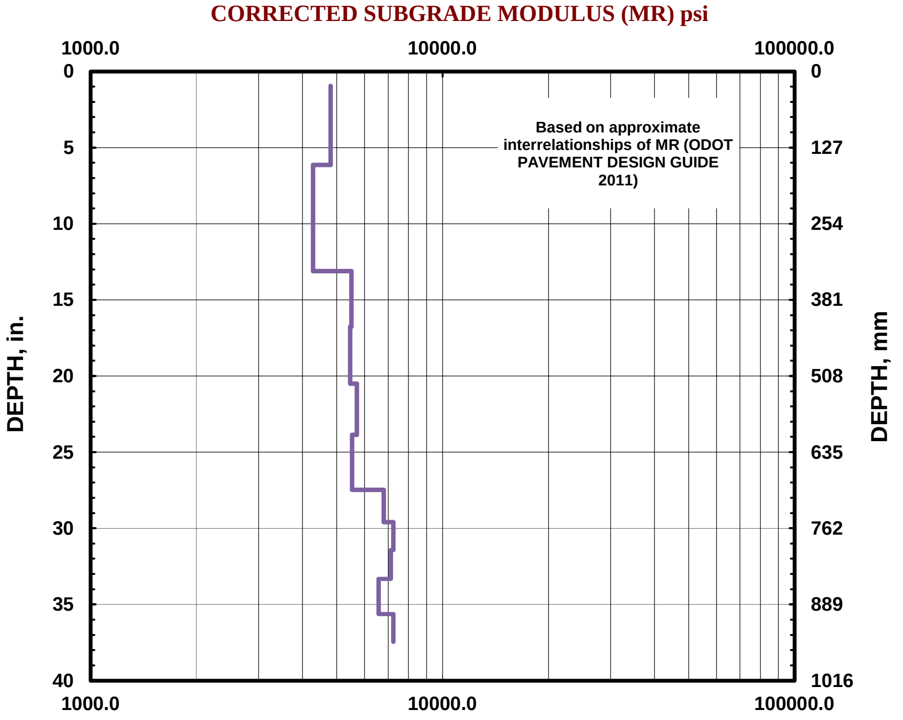


FIG. A15

DCP TEST DATA

Project: *S Kelso Grade Separation*

Date: *24-Apr-18*

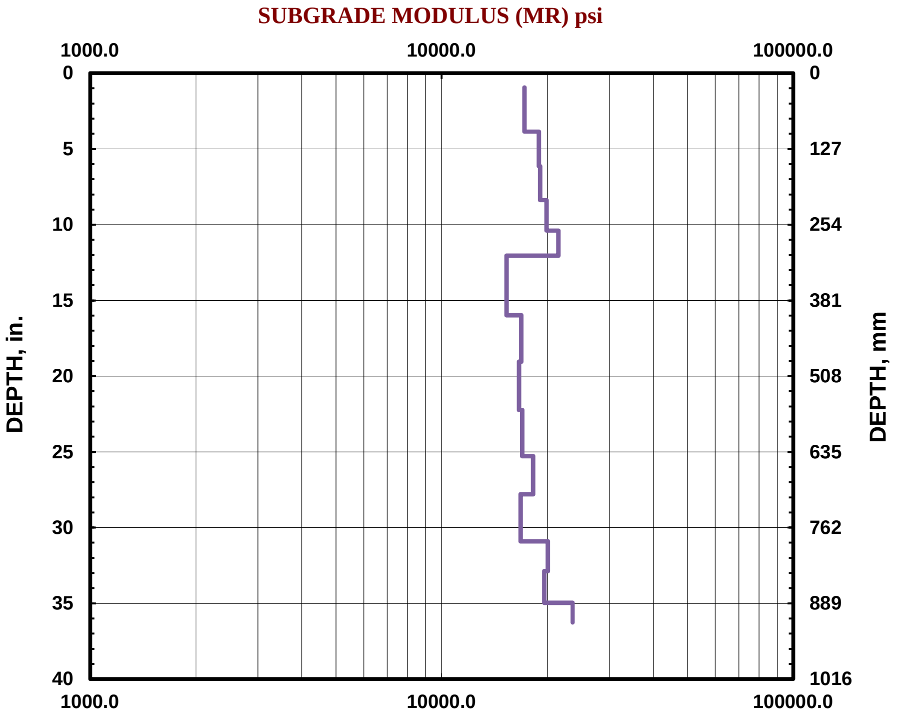
Location: *DCP-5*

Soil Type(s): *SP*

Hammer
○ 10.1 lbs.
● 17.6 lbs.
○ Both hammers used

○ CH
○ CL
● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	24	1
5	98	1
5	156	1
5	213	1
5	264	1
5	306	1
5	406	1
5	484	1
5	565	1
5	642	1
5	706	1
5	785	1
5	835	1
5	888	1
5	921	1



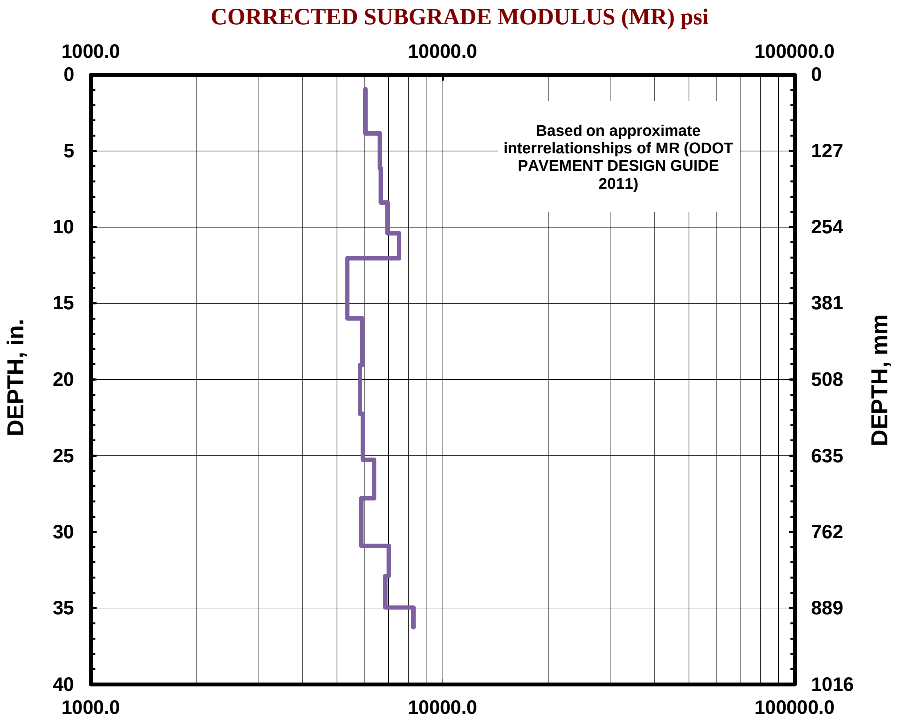


FIG. A16

DCP TEST DATA

Project: *S Kelso Grade Separation*

Location: *DCP-6*

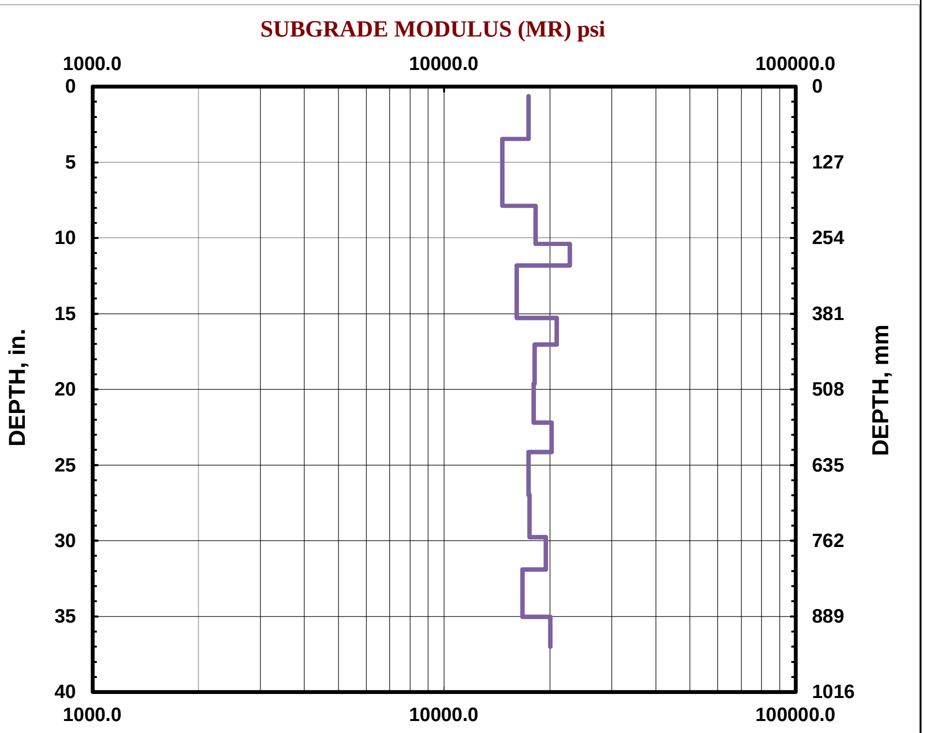
Date: *24-Apr-18*

Soil Type(s): *SP*

Hammer
○ 10.1 lbs.
● 17.6 lbs.
○ Both hammers used

○ CH
○ CL
● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	16	1
5	88	1
5	200	1
5	264	1
5	300	1
5	388	1
5	433	1
5	498	1
5	564	1
5	613	1
5	685	1
5	756	1
5	810	1
5	890	1
5	940	1



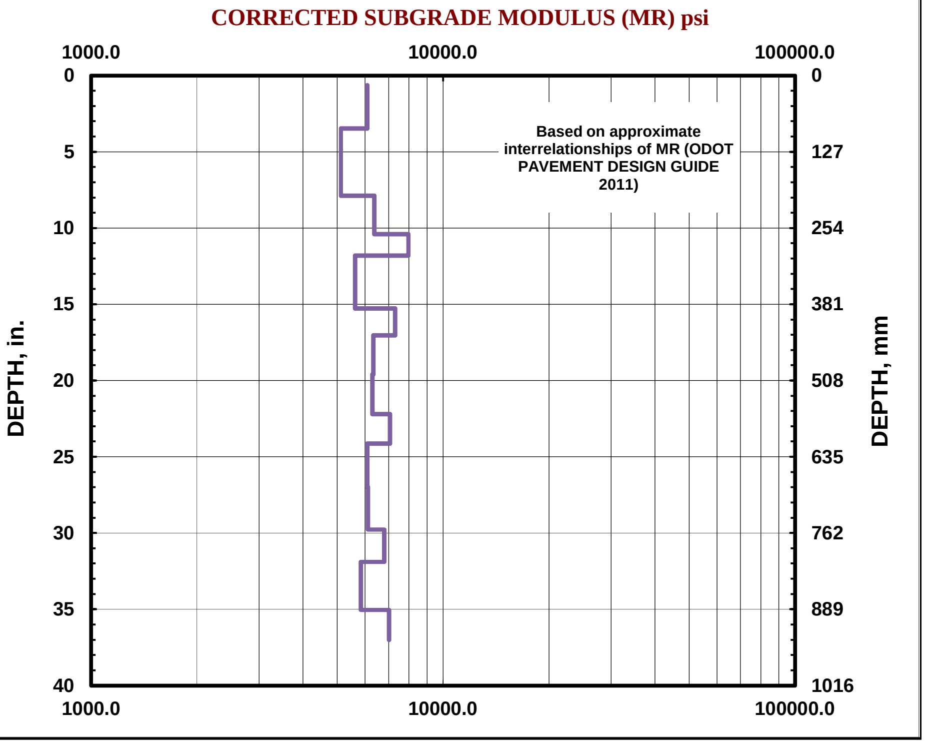


FIG. A17

APPENDIX B

CONE PENETRATION TEST (CPT) EXPLORATIONS

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B.4 CPT HOLE BACKFILL B-2

FIGURES

B1 Interpreted CPT Sounding CPT-1
B2 Interpreted CPT Sounding CPT-2
B3 Interpreted CPT Sounding CPT-3
B4 Interpreted CPT Sounding CPT-4

ATTACHMENTS

CPT-1 Pore Pressure Dissipation and Shear Wave Velocity Measurements
CPT-2 Shear Wave Velocity Measurements
CPT-3 Pore Pressure Dissipation and Shear Wave Velocity Measurements
CPT-4 Shear Wave Velocity Measurements

APPENDIX B

CONE PENETRATION TEST (CPT) EXPLORATIONS

B.1 GENERAL

The field exploration program included four Cone Penetration Tests (CPTs). The CPT locations were not surveyed but were referenced to nearby existing structures and should be considered approximate. Approximate CPT locations are shown on the Site and Exploration Plan, Figure 2. The CPTs were completed between April 18 and April 24, 2018, by Oregon Geotechnical Explorations, Inc. (OGE), of Keizer, Oregon. This appendix describes general CPT methods and presents logs of the materials encountered.

B.2 CONE PENETRATION TESTING

OGE pushed CPT-1 and CPT-2 using a truck-mounted CPT rig, while CPT-3 and CPT-4 were pushed using a smaller, track-mounted CPT rig which uses helical anchors, drilled into the ground, to help the rig to push down with a force greater than its weight.

During a CPT, a specialized cone assembly at the end of a steel probe is hydraulically pushed down through the subsurface. The cone assembly contains load cells and associated strain gauges which monitor the deformation of the load cells. One set of load cells deforms with increasing resistance to cone tip penetration. Another set of load cells deforms with increasing frictional resistance encountered on a sleeve on the outside of the assembly. The cone assembly also contains a piezometer which measures pore pressure. Data from the strain gauges and from the piezometer are transmitted from the cone assembly back through extension rods to a CPT recording device via a cable. Analysis software using industry standard calculations then converts the raw data signals from the instruments into cone resistance, sleeve friction, and pore pressure.

Pore pressure is useful in estimating soil behavior type because penetration has varying effects on pore pressure, depending on the type of material being penetrated. Dissipation of pore pressure can also be measured if the cone advance is temporarily halted. Pore pressure dissipation tests were performed at two depths in CPT-1 through CPT-4 explorations and can be used to estimate the static groundwater level and to estimate the soil hydraulic conductivity at the test locations.

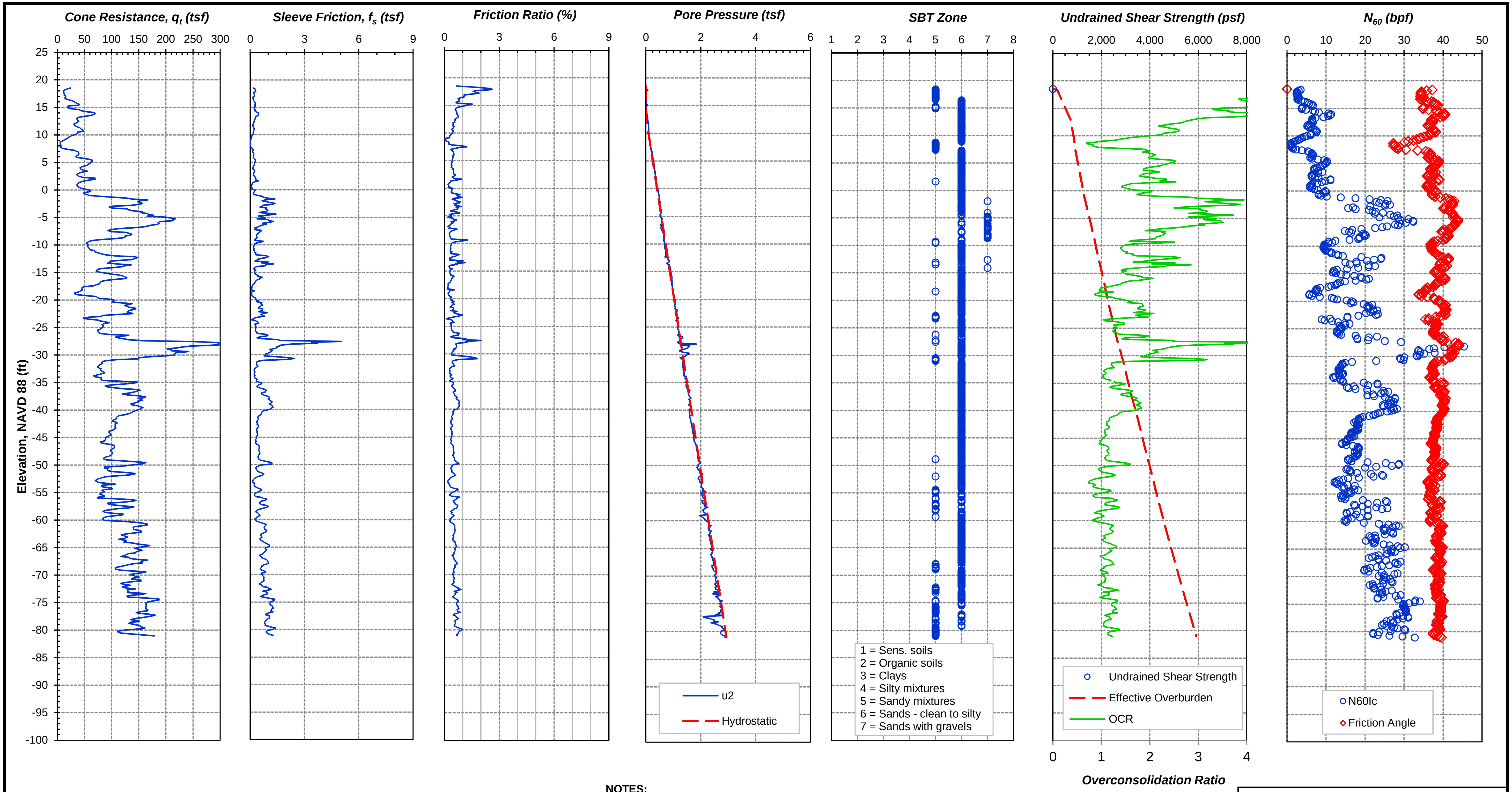
Twelve shear wave velocity tests were performed in CPT-1 and CPT-3 explorations. Six and three shear wave velocity tests were performed in CPT-2 and CPT-4 explorations, respectively.

B.3 CPT LOGS

All raw CPT data was reduced by OGE into values of cone resistance, sleeve friction, and pore pressure. Shannon & Wilson prepared graphic plots of the reduced data, along with several interpreted engineering parameters. The plots are presented in Figures B-1 through B-4, and include cone resistance (q_t) in tons per square foot (tsf), sleeve friction (f_s) in tsf, friction ratio (f_s/q_t) expressed as a percentage, pore pressure in tsf, estimated soil behavior type (SBT), undrained shear strength in pounds per square foot (psf), and estimated SPT N-value (N_{60}) in blows per foot (bpf). Plots of the pore pressure dissipation tests, prepared by OGE, are attached to the end of this appendix, following the figures.

B.4 CPT HOLE BACKFILL

All CPT holes were backfilled in accordance with Washington Department of Ecology regulations. No wells or other instruments were installed in the holes. The holes were backfilled from the bottom up to the existing ground surface using a bentonite chips.



NOTES:

1. SBT zone computed using procedure by Jefferies & Been (2006).
2. Undrained shear strength computed using the following equation:

$$s_u = \sigma'_v \left(s_u / \sigma'_v\right)_{NC} OCR^m$$

where $(s_u/\sigma'_v)_{NC}$ = 0.22 and m = 0.8.

3. Preconsolidation pressure computed using procedure by Mayne and others (2009).
4. N_{60} computed using procedure by Lunne and others (1997).
5. Ground surface elevation apprx. = 20 ft.

S. Kelso Railroad
Grade Separation
Kelso, Washington

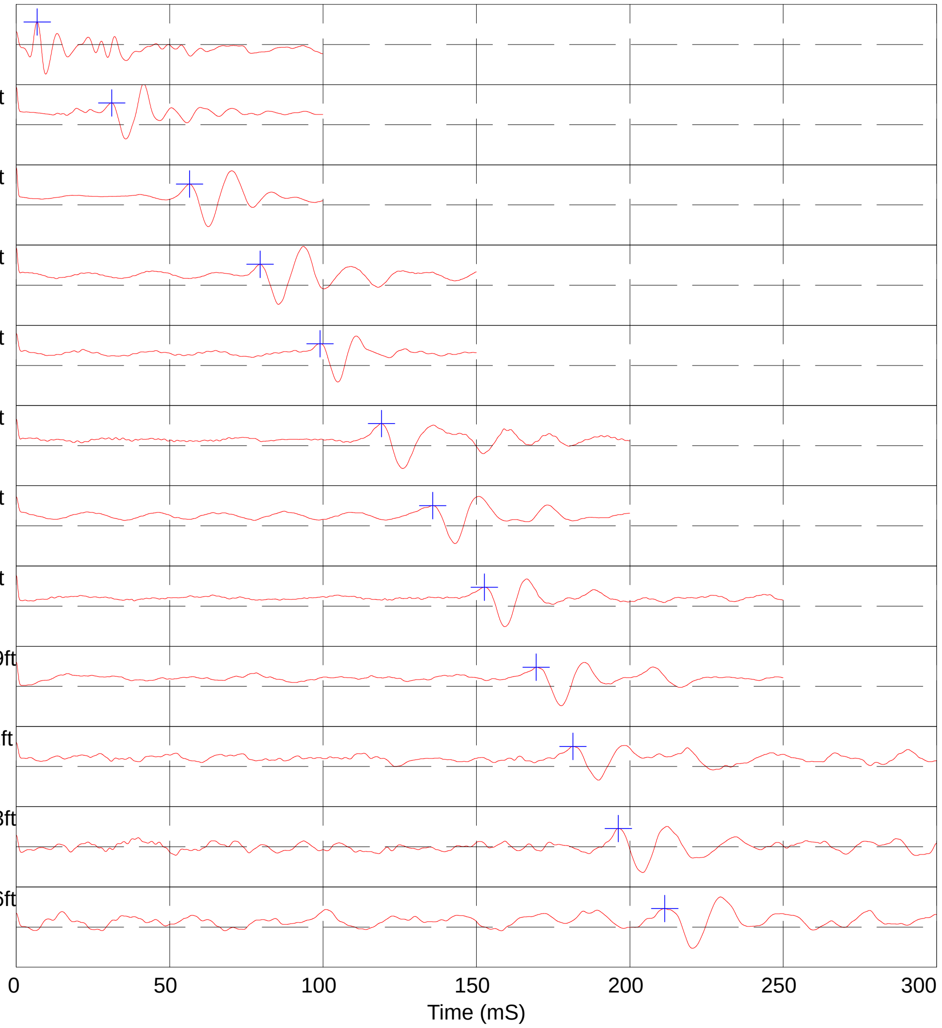
INTERPRETED CPT SOUNDING
CPT-1

September 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B1

COMMENT: Shannon & Wilson / CPT-1 / 302 Hazel St Kelso

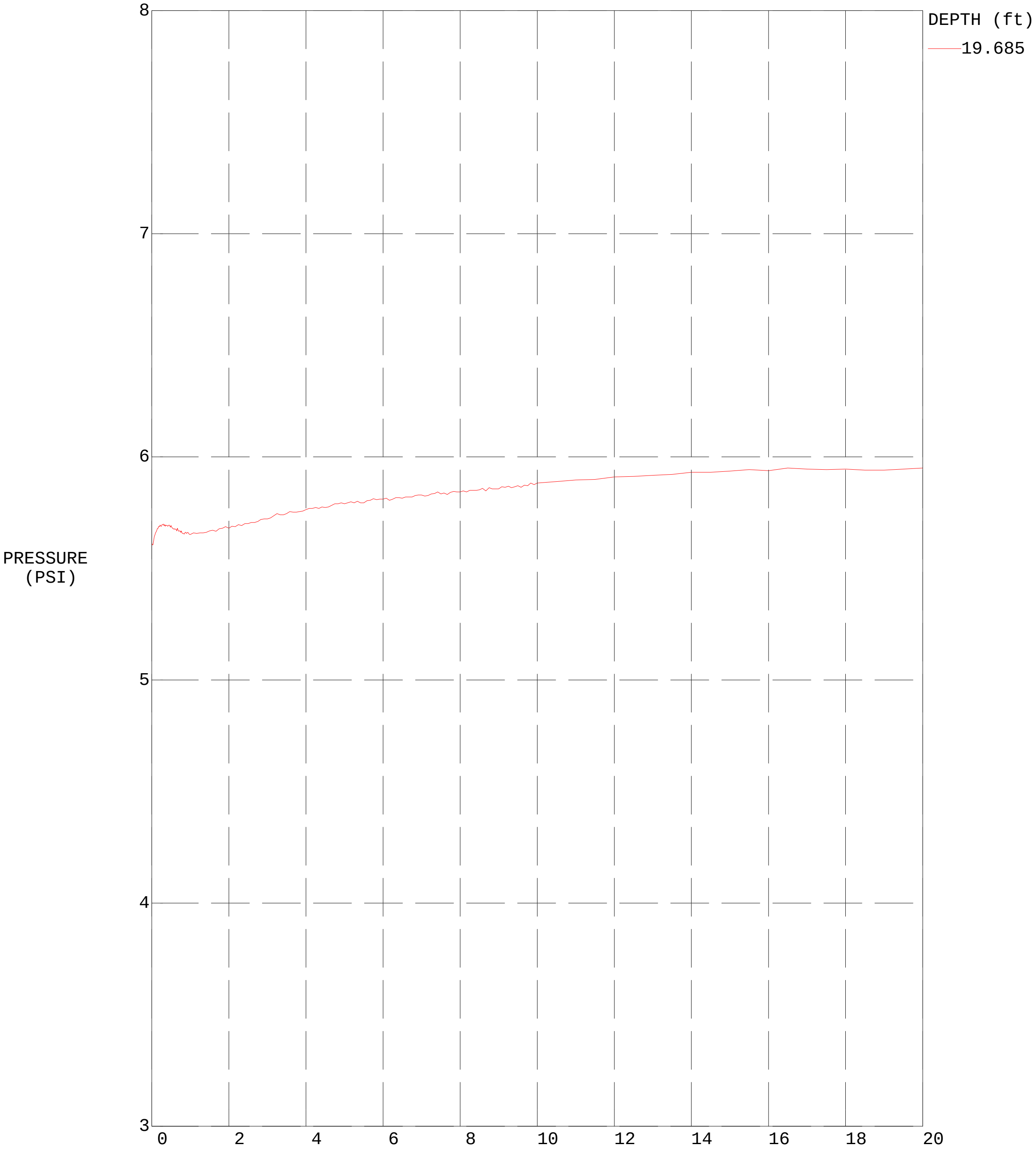


Depth 3.28ft	Ref*	Arrival 6.84mS	Velocity*
Depth 13.12ft	Ref 3.28ft	Arrival 31.13mS	Velocity 388.72ft/S
Depth 26.25ft	Ref 13.12ft	Arrival 56.52mS	Velocity 514.01ft/S
Depth 39.37ft	Ref 26.25ft	Arrival 79.49mS	Velocity 570.33ft/S
Depth 52.49ft	Ref 39.37ft	Arrival 99.02mS	Velocity 671.33ft/S
Depth 65.62ft	Ref 52.49ft	Arrival 119.05mS	Velocity 654.56ft/S
Depth 78.74ft	Ref 65.62ft	Arrival 135.77mS	Velocity 784.70ft/S
Depth 91.86ft	Ref 78.74ft	Arrival 152.57mS	Velocity 781.14ft/S
Depth 104.99ft	Ref 91.86ft	Arrival 169.44mS	Velocity 777.57ft/S
Depth 118.11ft	Ref 104.99ft	Arrival 181.47mS	Velocity 1090.67ft/S
Depth 131.23ft	Ref 118.11ft	Arrival 196.24mS	Velocity 888.72ft/S
Depth 144.36ft	Ref 131.23ft	Arrival 211.35mS	Velocity 868.07ft/S

Hammer to Rod String Distance (ft): 1.97
* = Not Determined

COMMENT: Shannon & Wilson / CPT-1 / 302 Hazel St Kelso

TEST DATE: 4/21/2018 8:59:44 AM

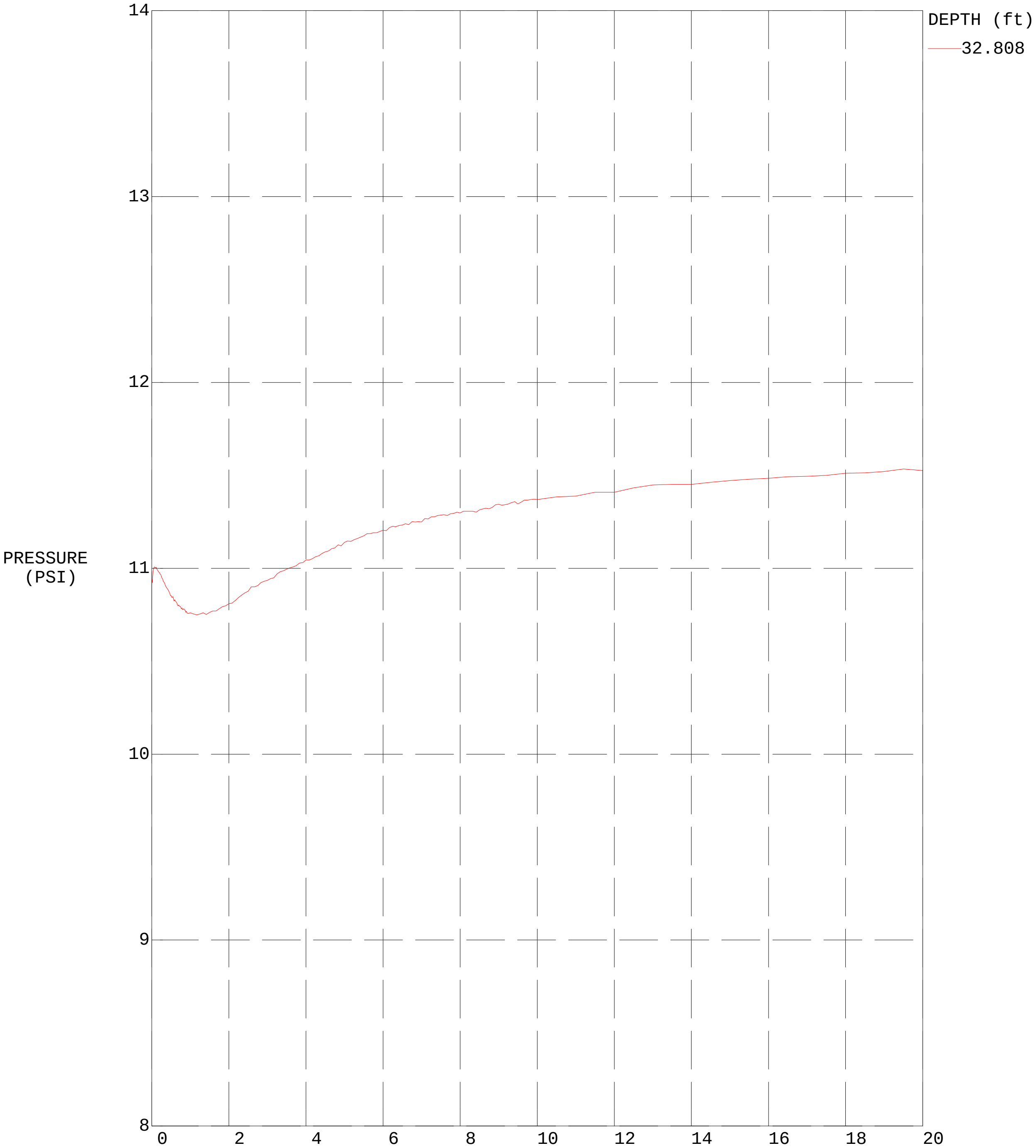


MAXIMUM PRESSURE = 5.95 (PSI)

HYDROSTATIC PRESSURE = 5.954 (PSI), WATER TABLE: 5.95 ft

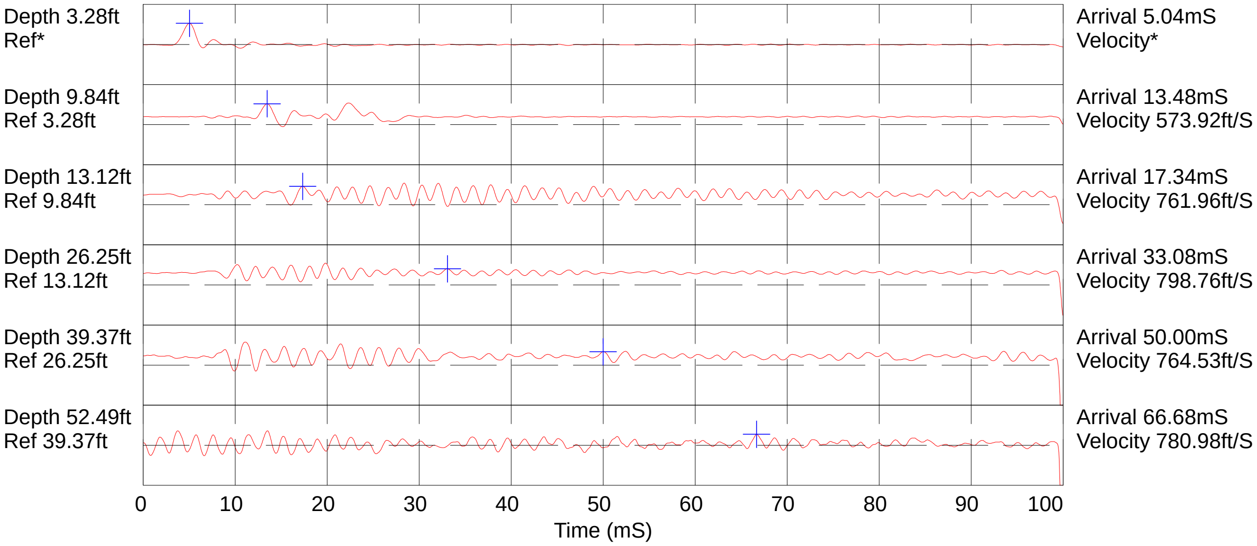
COMMENT: Shannon & Wilson / CPT-1 / 302 Hazel St Kelso

TEST DATE: 4/21/2018 8:59:44 AM



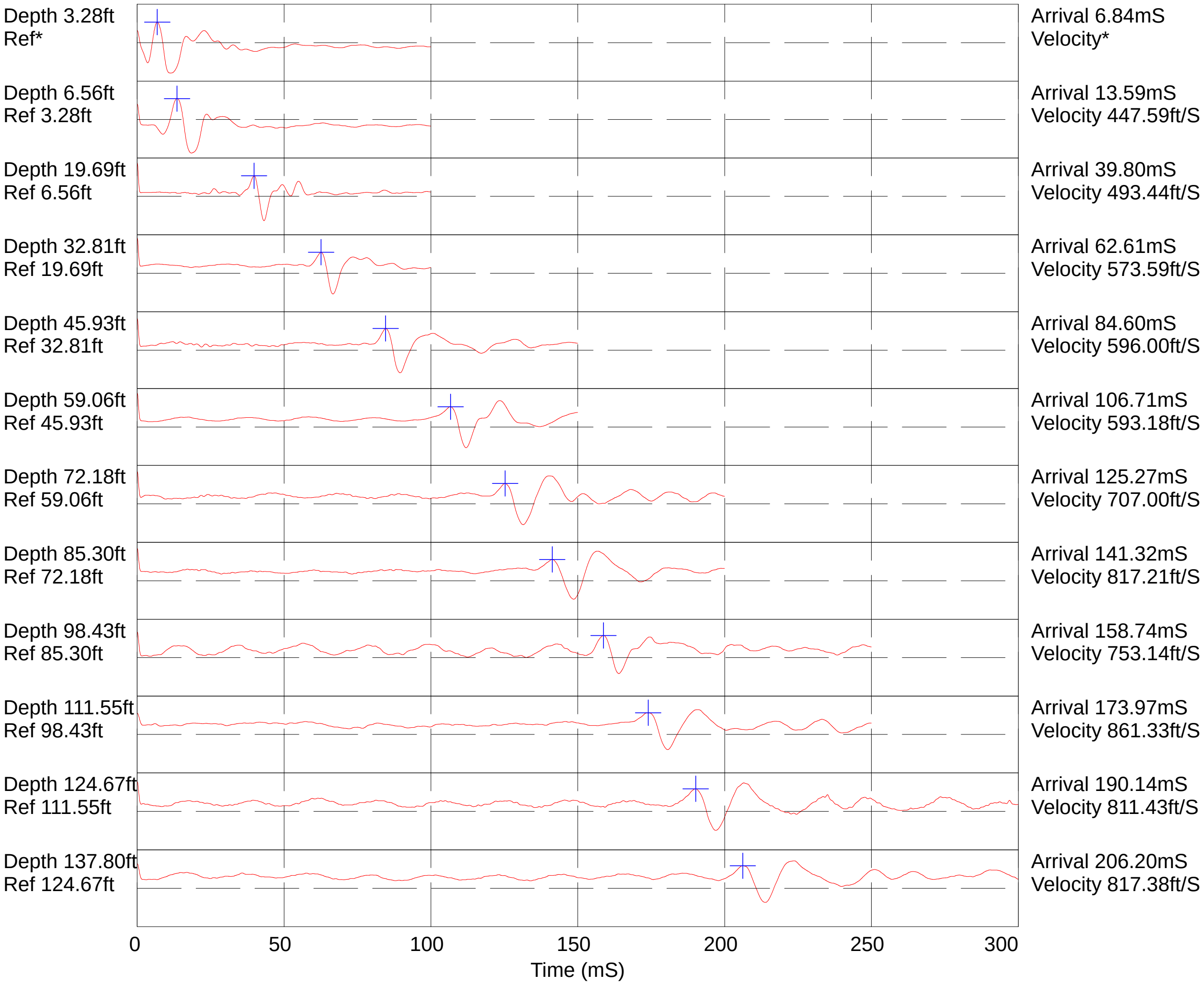
MAXIMUM PRESSURE = 11.535 (PSI)ME: (MINUTES)
HYDROSTATIC PRESSURE = 11.642 (PSI), WATER TABLE: 5.95 ft

COMMENT: Shannon & Wilson / CPT-2 / 302 Hazel St Kelso



Hammer to Rod String Distance (ft): 5.58
* = Not Determined

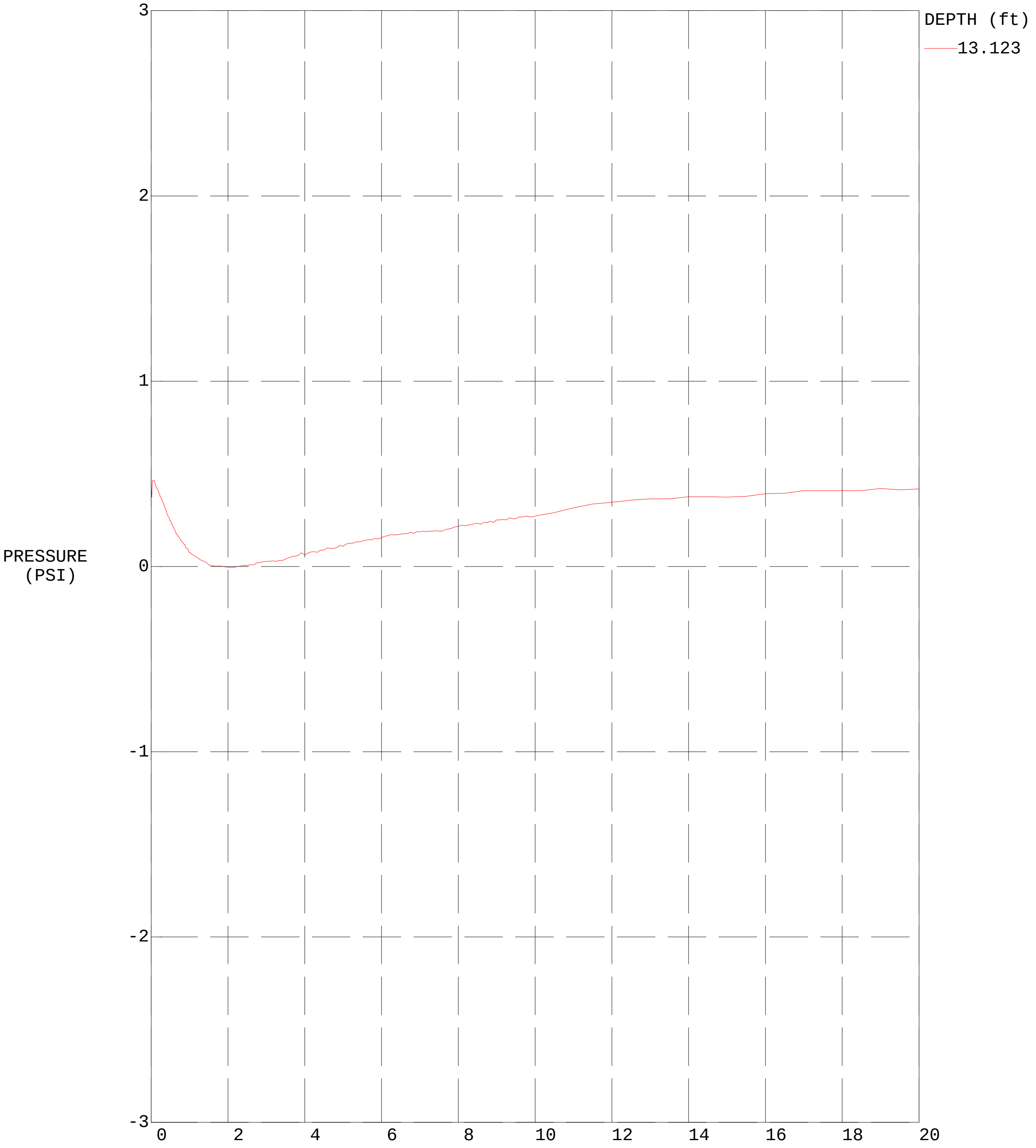
COMMENT: Shannon & Wilson / CPT-3 / 302 Hazel St Kelso



Hammer to Rod String Distance (ft): 1.97
* = Not Determined

COMMENT: Shannon & Wilson / CPT-3 / 302 Hazel St Kelso

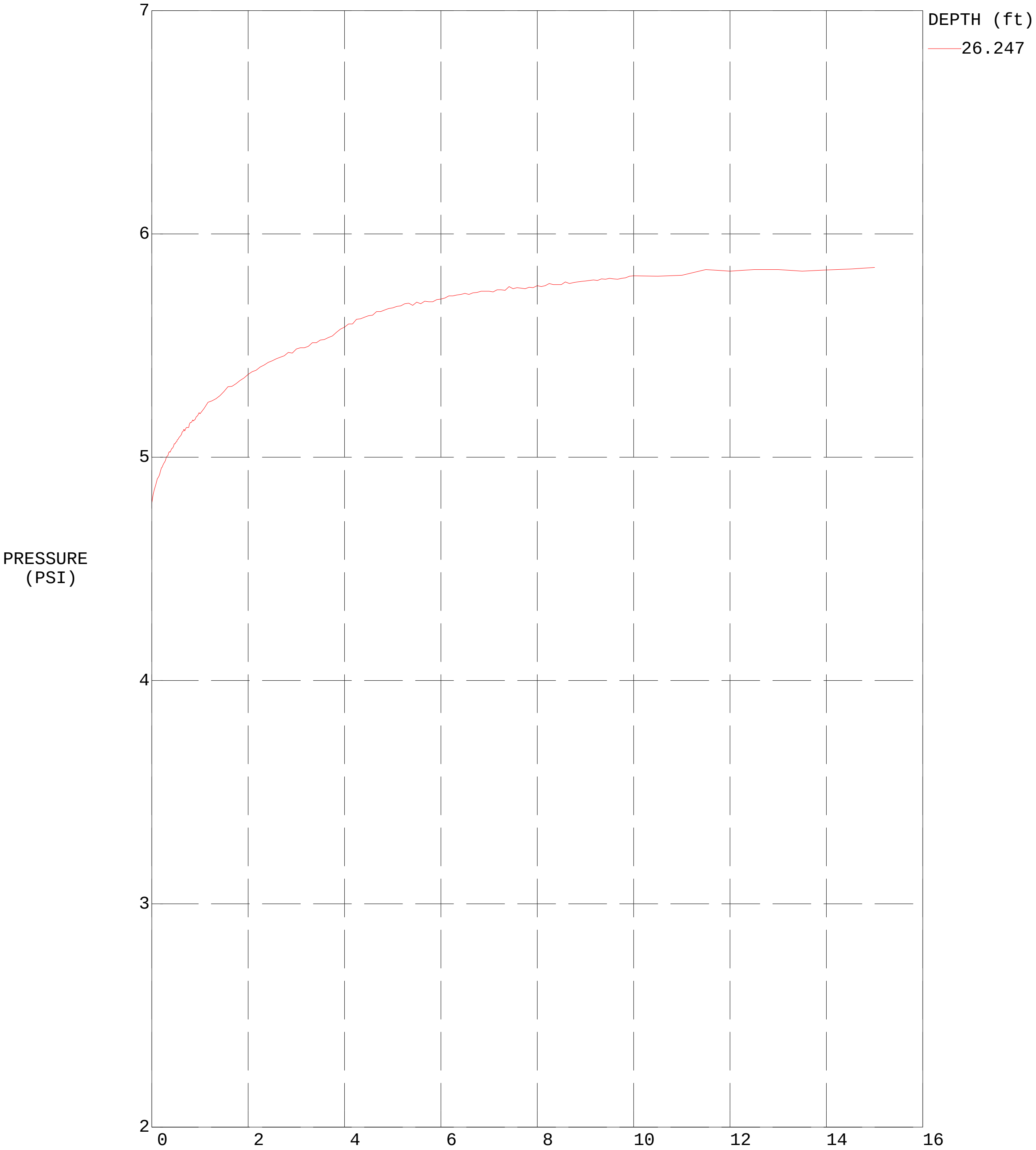
TEST DATE: 4/20/2018 9:27:37 AM



MAXIMUM PRESSURE = 0.462 (PSI)IME: (MINUTES)
HYDROSTATIC PRESSURE = 0.27 (PSI), WATER TABLE: 12.50 ft

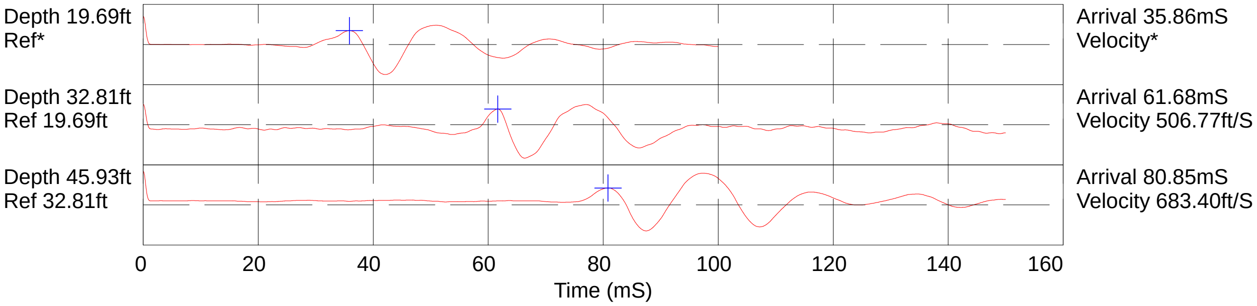
COMMENT: Shannon & Wilson / CPT-3 / 302 Hazel St Kelso

TEST DATE: 4/20/2018 9:27:37 AM

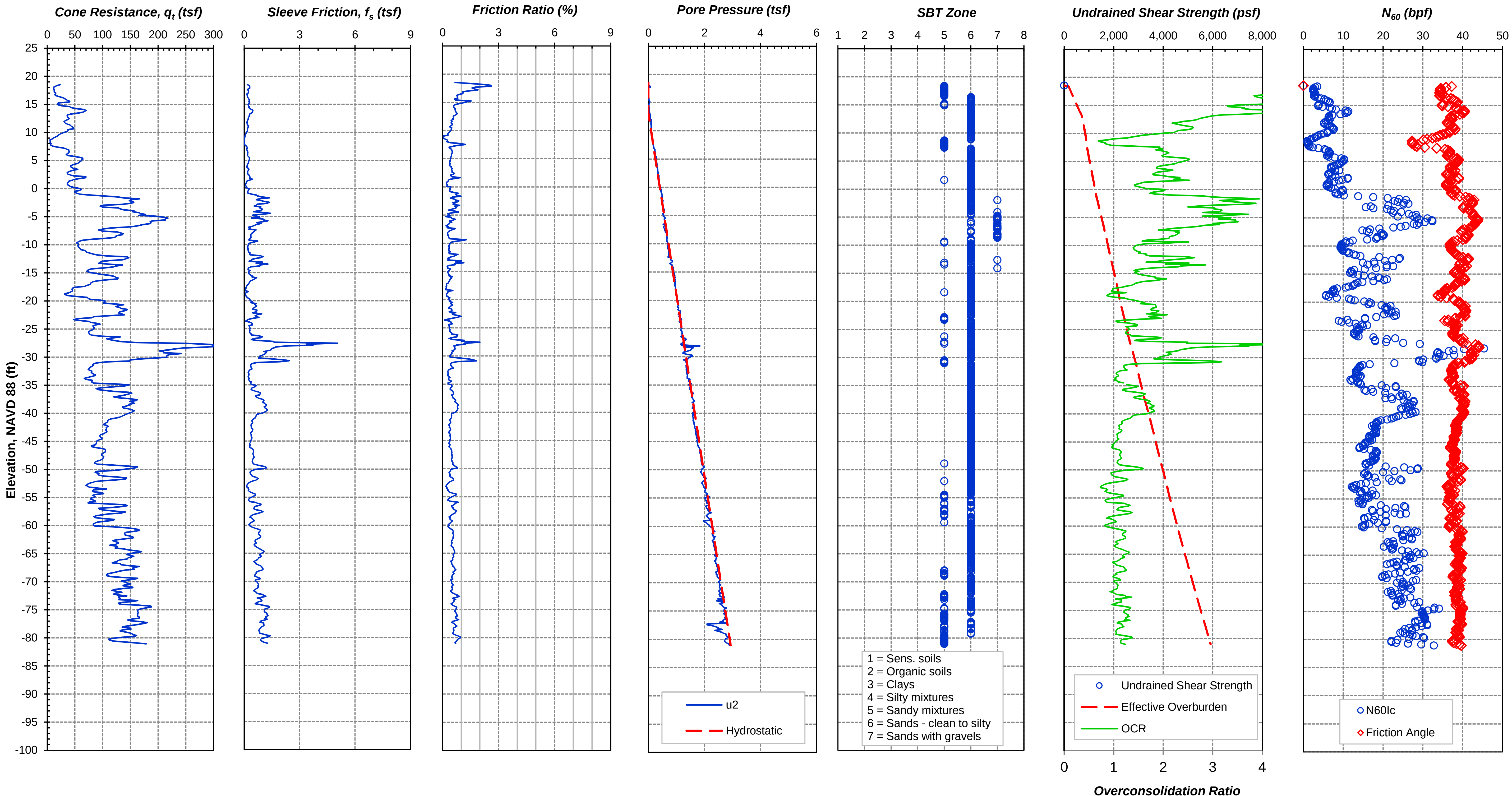


MAXIMUM PRESSURE = 5.85 (PSI)TIME: (MINUTES)
HYDROSTATIC PRESSURE = 5.958 (PSI), WATER TABLE: 12.50 ft

COMMENT: Shannon & Wilson / CPT-4a / 302 Hazel St Kelso



Hammer to Rod String Distance (ft): 1.97
* = Not Determined



NOTES:

1. SBT zone computed using procedure by Jefferies & Been (2006).
2. Undrained shear strength computed using the following equation:

$$s_u = \sigma'_v \left(s_u / \sigma'_v\right)_{NC} OCR^m$$

where $(s_u/\sigma'_v)_{NC}$ = 0.22 and m = 0.8.

3. Preconsolidation pressure computed using procedure by Mayne and others (2009).
4. N_{60} computed using procedure by Lunne and others (1997).
5. Ground surface elevation apprx. = 20 ft.

S. Kelso Railroad
Grade Separation
Kelso, Washington

INTERPRETED CPT SOUNDING
CPT-1

September 2018 24-1-04201-001

SHANNON & WILSON, INC.
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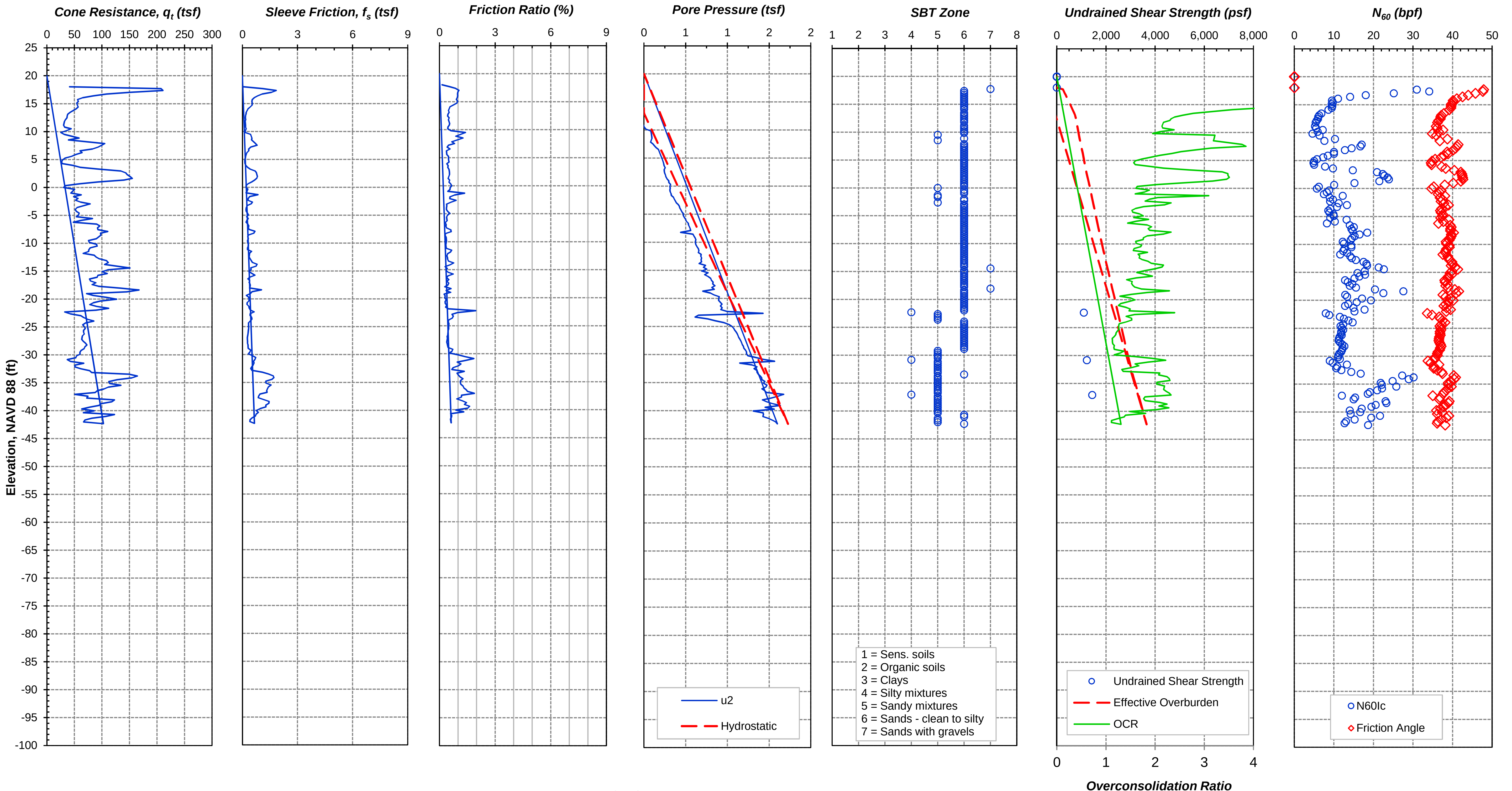
FIG. B1

I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\Expl\CPTs\CPT Logs_SW plots\CPT-2



NOTES:

1. SBT zone computed using procedure by Jefferies & Been (2006).
2. Undrained shear strength computed using the following equation:

$$s_u = \sigma'_v \left(s_u / \sigma'_v\right)_{NC} OCR^m$$

where $(s_u/\sigma'_v)_{NC}$ = 0.22 and m = 0.8.

3. Preconsolidation pressure computed using procedure by Mayne and others (2009).
4. N_{60} computed using procedure by Lunne and others (1997).
5. Ground surface elevation apprx. = 20 ft.

S. Kelso Railroad
Grade Separation
Kelso, Washington

INTERPRETED CPT SOUNDING
CPT-2

September 2018 24-1-04201-001

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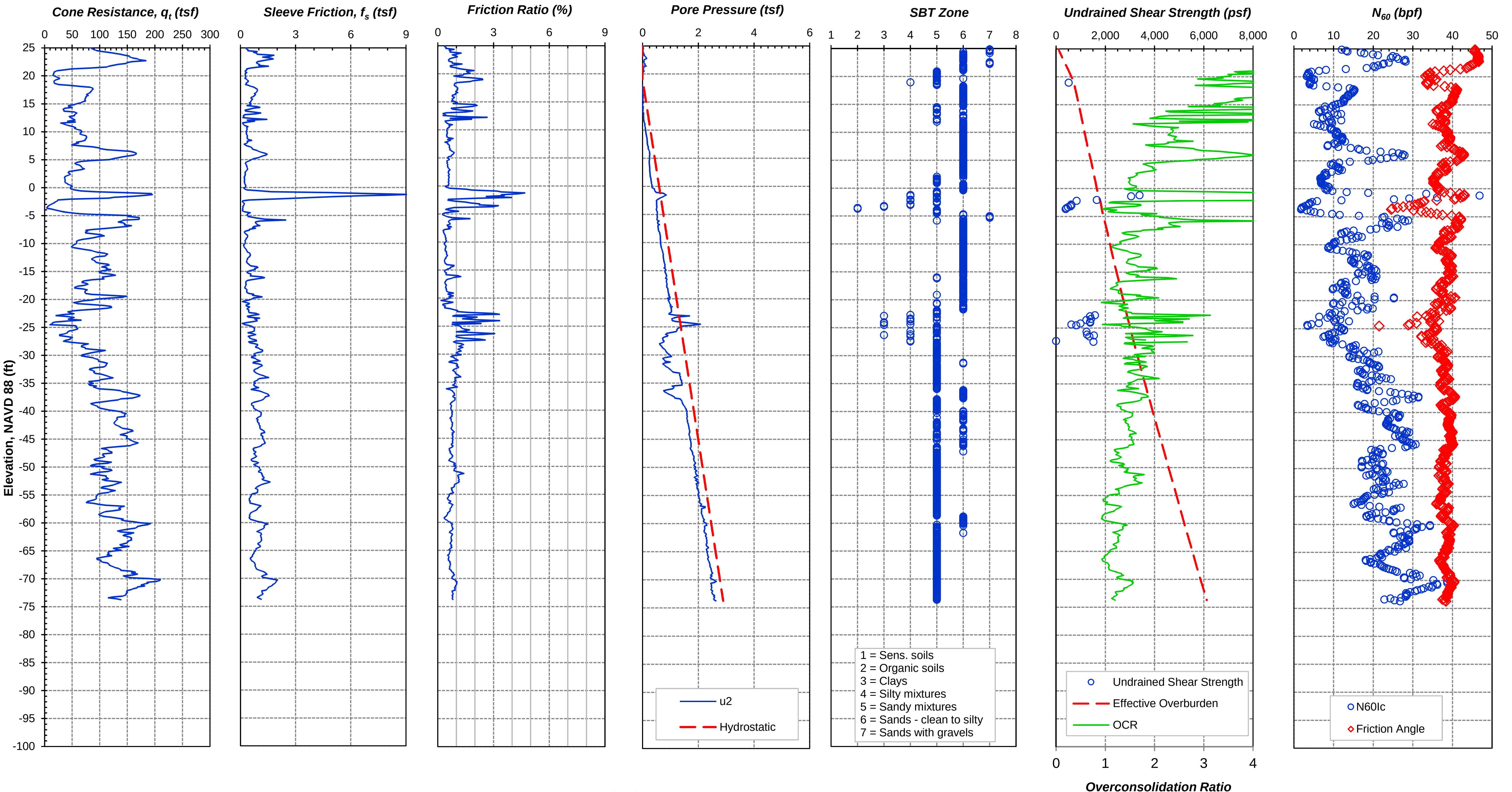
FIG. B2

6/11/2018



NOTES:

1. SBT zone computed using procedure by Jefferies & Been (2006).
2. Undrained shear strength computed using the following equation:

$$s_u = \sigma'_v \left(s_u / \sigma'_v\right)_{NC} OCR^m$$

where $(s_u/\sigma'_v)_{NC}$ = 0.22 and m = 0.8.

3. Preconsolidation pressure computed using procedure by Mayne and others (2009).
4. N_{60} computed using procedure by Lunne and others (1997).
5. Ground surface elevation approx. = 26 ft.

S. Kelso Railroad
Grade Separation
Kelso, Washington

INTERPRETED CPT SOUNDING
CPT-3

September 2018 24-1-04201-001

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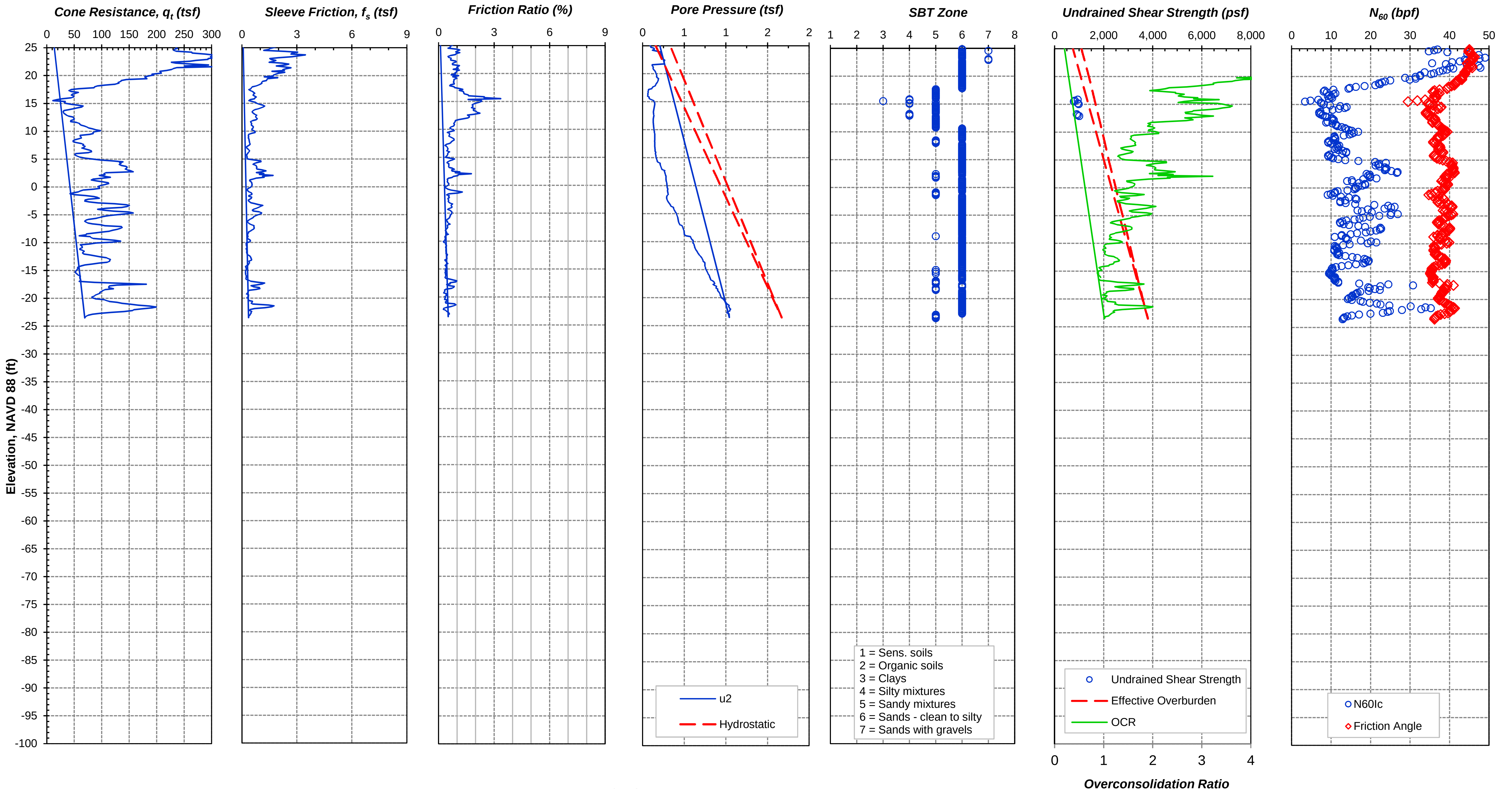
FIG. B3



NOTES:

1. SBT zone computed using procedure by Jefferies & Been (2006).
2. Undrained shear strength computed using the following equation:

$$s_u = \sigma'_v \left(s_u / \sigma'_v\right)_{NC} OCR^m$$

where $(s_u/\sigma'_v)_{NC}$ = 0.22 and m = 0.8.

3. Preconsolidation pressure computed using procedure by Mayne and others (2009).
4. N_{60} computed using procedure by Lunne and others (1997).
5. Ground surface elevation apprx. = 37 ft.

S. Kelso Railroad
Grade Separation
Kelso, Washington

INTERPRETED CPT SOUNDING
CPT-4

September 2018 24-1-04201-001

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FIG. B4

APPENDIX C

LABORATORY TEST RESULTS

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  C.2.2 Particle-size analyses C-1

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APPENDIX C

LABORATORY TEST RESULTS

C.1. GENERAL

The soil samples obtained during the field explorations were classified in the field in general accordance with ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) and revised in accordance with Soil and Rock Classification and Logging from Chapter 4 of the WSDOT Geotechnical Design Manual (May 2015). The samples were then reviewed in the laboratory. The physical characteristics of the samples were noted and the field classifications were modified where necessary. Representative samples were selected for various laboratory tests. We refined our visual-manual soil classifications based on the results of the laboratory tests, using Soil and Rock Classification and Logging from Chapter 4 of the WSDOT Geotechnical Design Manual (May 2015). The refined classifications were then incorporated into the Logs of Test Borings, presented in Appendix A.

The testing program included moisture content analyses, Atterberg limit tests, particle-size analyses, and a one-dimensional consolidation test. Laboratory testing was performed by Shannon & Wilson, Inc. The testing procedures from the laboratory program are summarized in the following paragraphs. Unless noted otherwise, all test procedures were in general accordance to applicable ASTM International (ASTM) standards. “General accordance” means that certain local and common descriptive practices and methodologies have been followed.

C.2. SOIL TESTING

C.2.1 Moisture (Natural water) content

Natural moisture content determinations were performed in accordance with ASTM D2216 on selected soil samples. The natural moisture content is a measure of the amount of moisture in the soil at the time the explorations are performed, and is defined as the ratio of the weight of water to the dry weight of the soil, expressed as a percentage. The results of the moisture content determinations are presented graphically in the Logs of Test Borings in Appendix A, and in the Laboratory Summaries presented in this Appendix.

C.2.2 Particle-size analyses

Particle-size analyses were conducted on selected samples to determine their grain-size distributions. Grain size distributions were determined in accordance with ASTM D6913, or

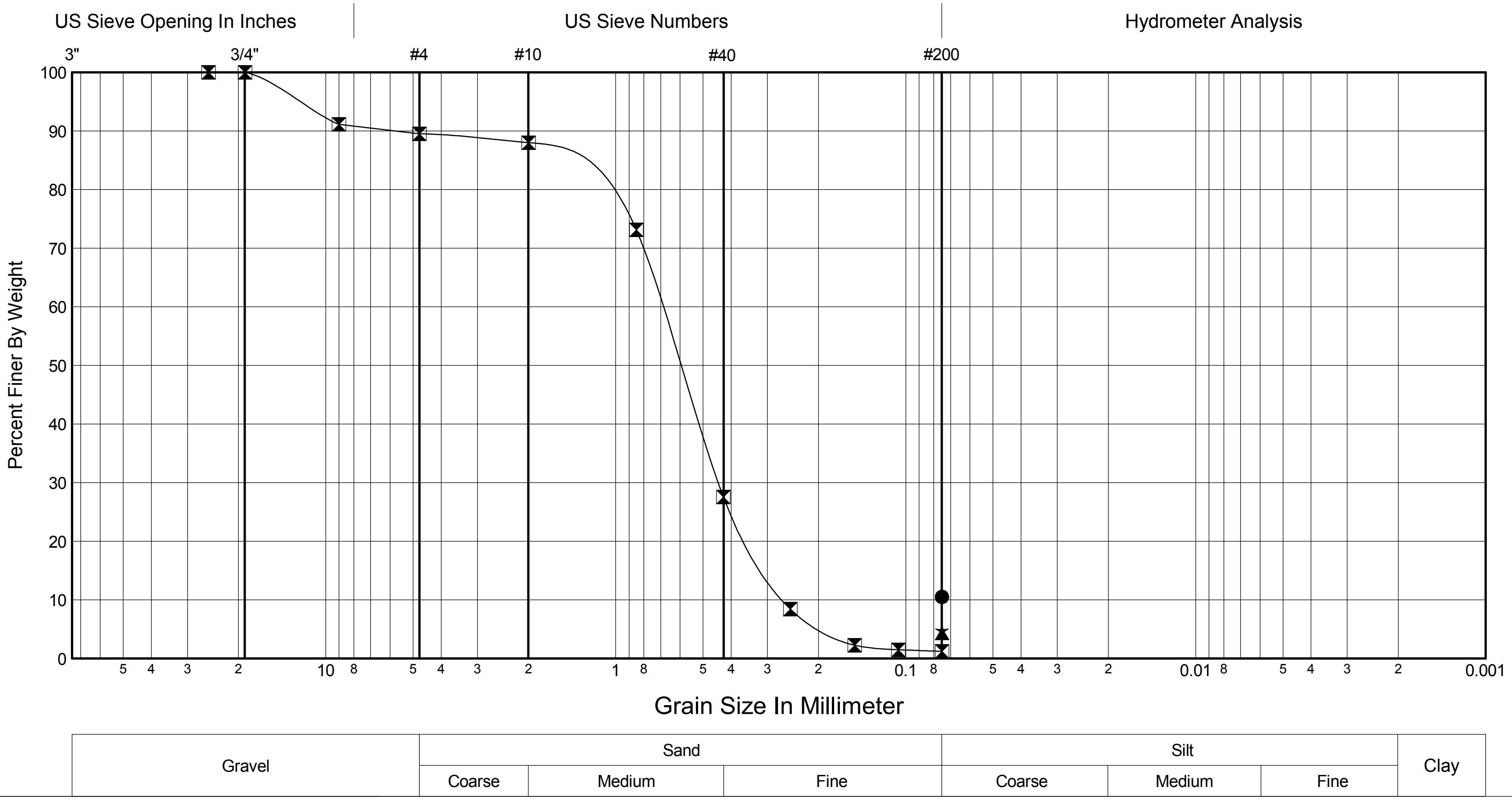
D1140, as applicable. For all samples, a wet sieve analysis was performed to determine the percentage (by weight) of each sample passing the No. 200 (0.075 mm) sieve. For select samples, the material retained on the No. 200 sieve was shaken through a series of sieves to determine the distribution of the plus No. 200 fraction (ASTM D6913). For some samples, only the percentage of the sample passing the No. 200 (0.075mm) sieve was determined (ASTM D1140). The percentage of gravel, sand and fines where tested, is presented in the Logs of Test Borings, and in the Laboratory Summaries presented in this Appendix.

Job No. **24-1-04201-001**　Date **May 29, 2018**
Hole No. **B-1**　Sheet **C1**
Project **South Kelso Railroad Grade Separation Project**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**5.5**	**S-2A**	**SM**	**Silty SAND**	**24**						**-**	**-**	**10.5**							
⧗	**20.0**	**S-6**	**SP**	**Poorly graded SAND with few to little gravel**	**26**						**10.5**	**88.3**	**1.2**	**1.1**	**2.7**	**0.696**	**0.60**	**0.44**	**0.34**	**0.261**
▲	**50.2**	**S-12A**	**SP**	**Poorly graded SAND**	**26**						**-**	**-**	**4.2**							
★	**80.0**	**S-18**	**SP**	**Poorly graded SAND**	**27**						**-**	**-**	**4.8**							

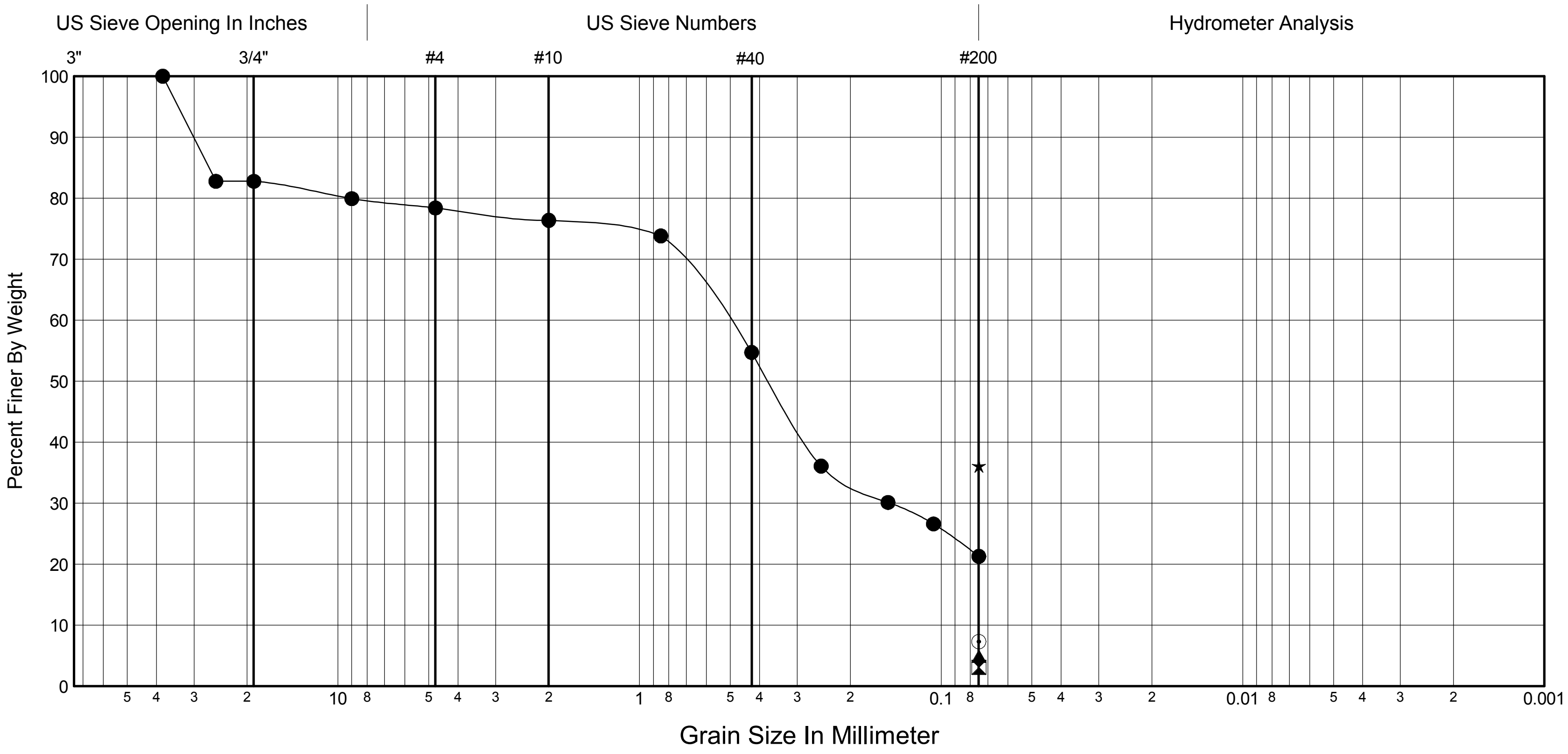


Job No. **24-1-04201-001** Date **May 29, 2018**
Hole No. **B-2** Sheet **C2**
Project **South Kelso Railroad Grade Separation Project**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**10.0**	**S-4**	**SM**	**Silty SAND with gravel**	**19**						**21.6**	**57.1**	**21.3**			**0.515**	**0.37**	**0.15**		
⧗	**20.0**	**S-6**	**SP**	**Poorly graded SAND**	**33**						**-**	**-**	**3.1**							
▲	**35.0**	**S-9**	**SP**	**Poorly graded SAND**	**25**						**-**	**-**	**4.9**							
★	**55.0**	**S-13B**	**SM**	**Silty SAND**	**41**						**-**	**-**	**36.1**							
☉	**60.0**	**S-14B**	**SP**	**Poorly graded SAND**	**31**						**-**	**-**	**7.3**							



Gravel	Sand			Silt			Clay
	Coarse	Medium	Fine	Coarse	Medium	Fine	

Job No. **24-1-04201-001** Date **May 29, 2018**

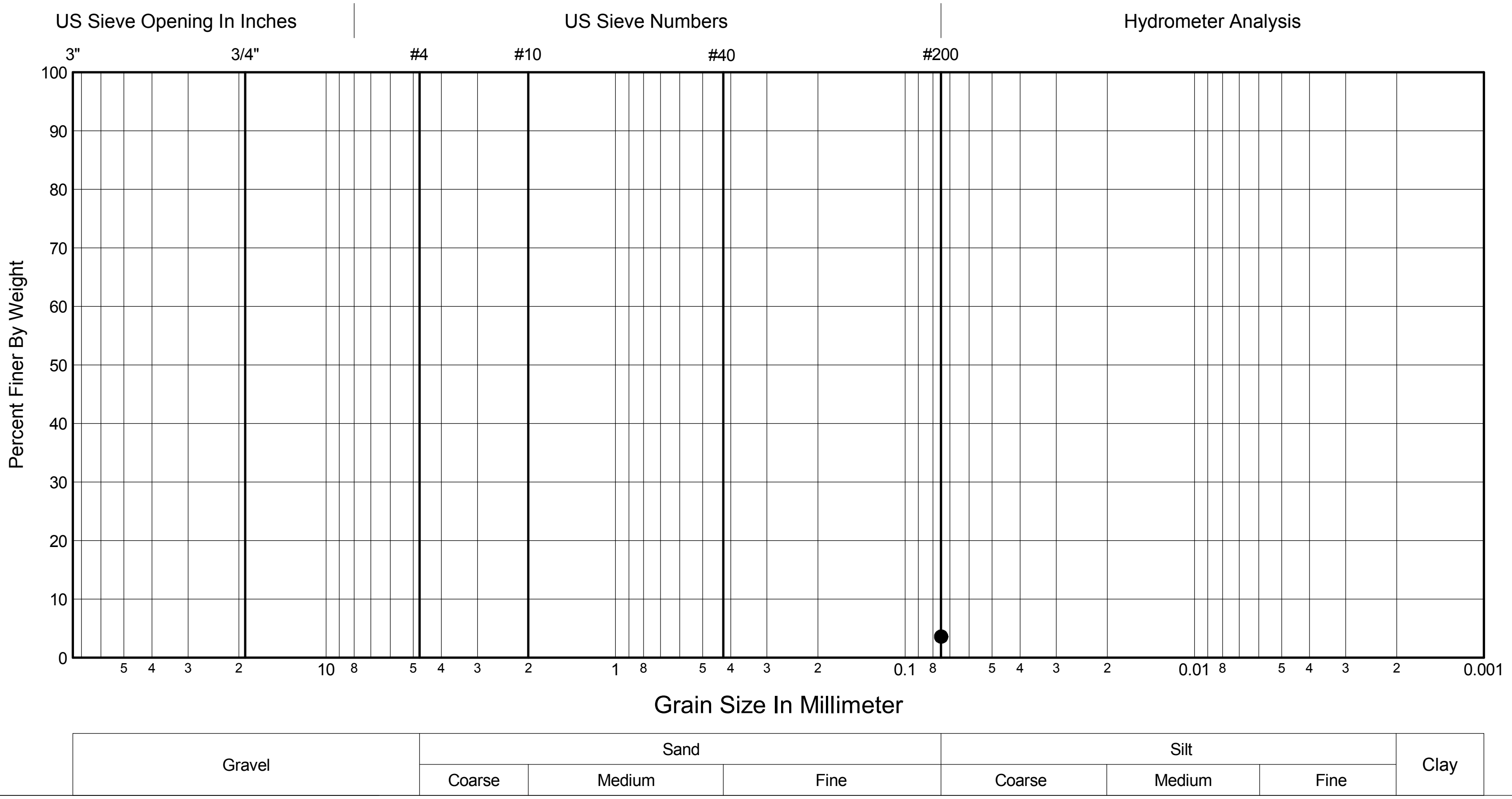
Hole No. **B-2** Sheet **C2**

Project **South Kelso Railroad Grade Separation Project**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**85.0**	**S-19**	**SP**	**Poorly graded SAND**	**26**						**-**	**-**	**3.6**							

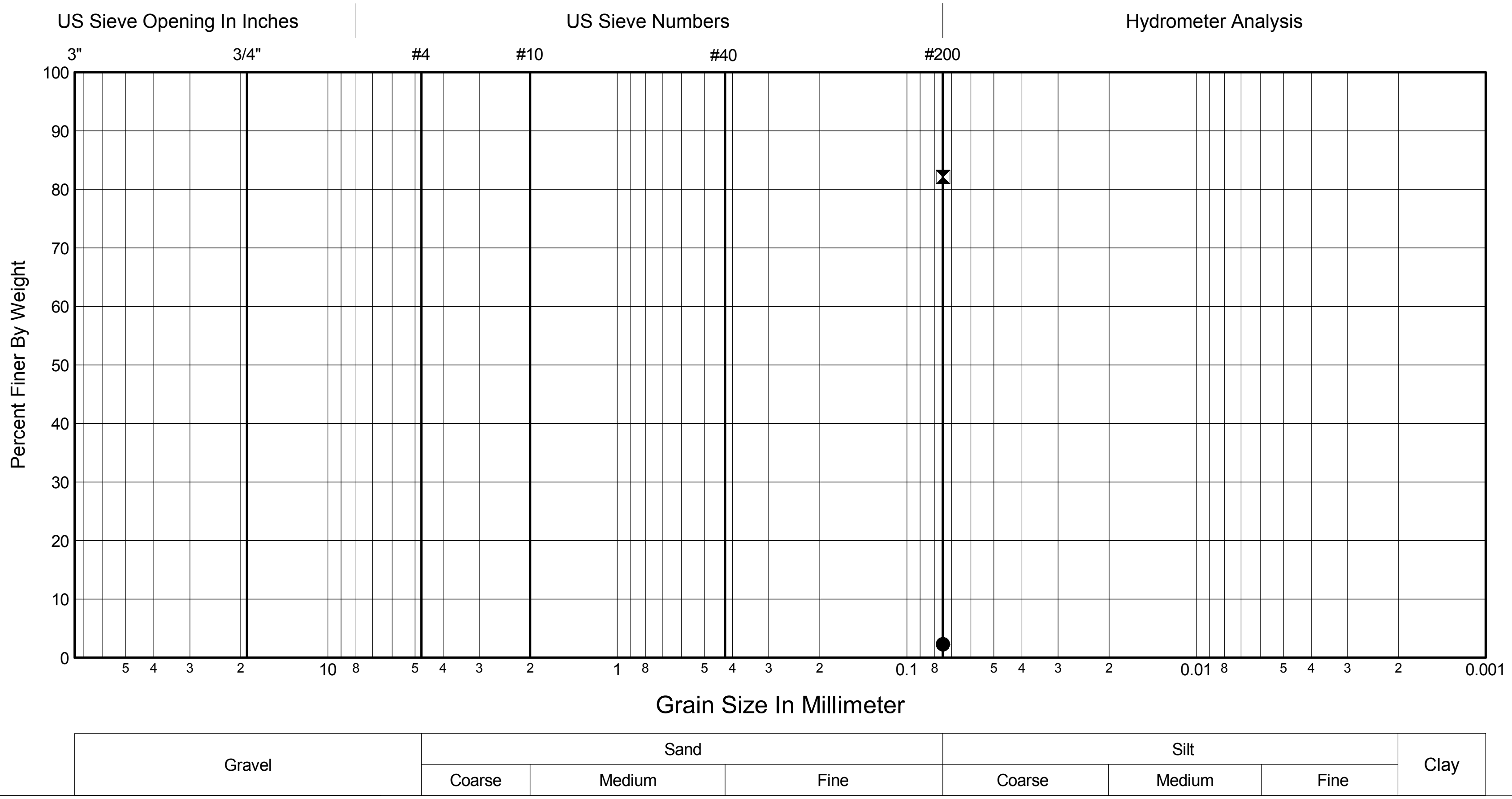


Job No. **24-1-04201-001** Date **May 29, 2018**
Hole No. **B-3** Sheet **C3**
Project **South Kelso Railroad Grade Separation Project**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**15.0**	**S-5**	**SP**	**Poorly graded SAND**	**31**						**-**	**-**	**2.3**							
⧗	**50.0**	**S-15**	**ML**	**SILT with Sand**	**45**						**-**	**-**	**82.1**							



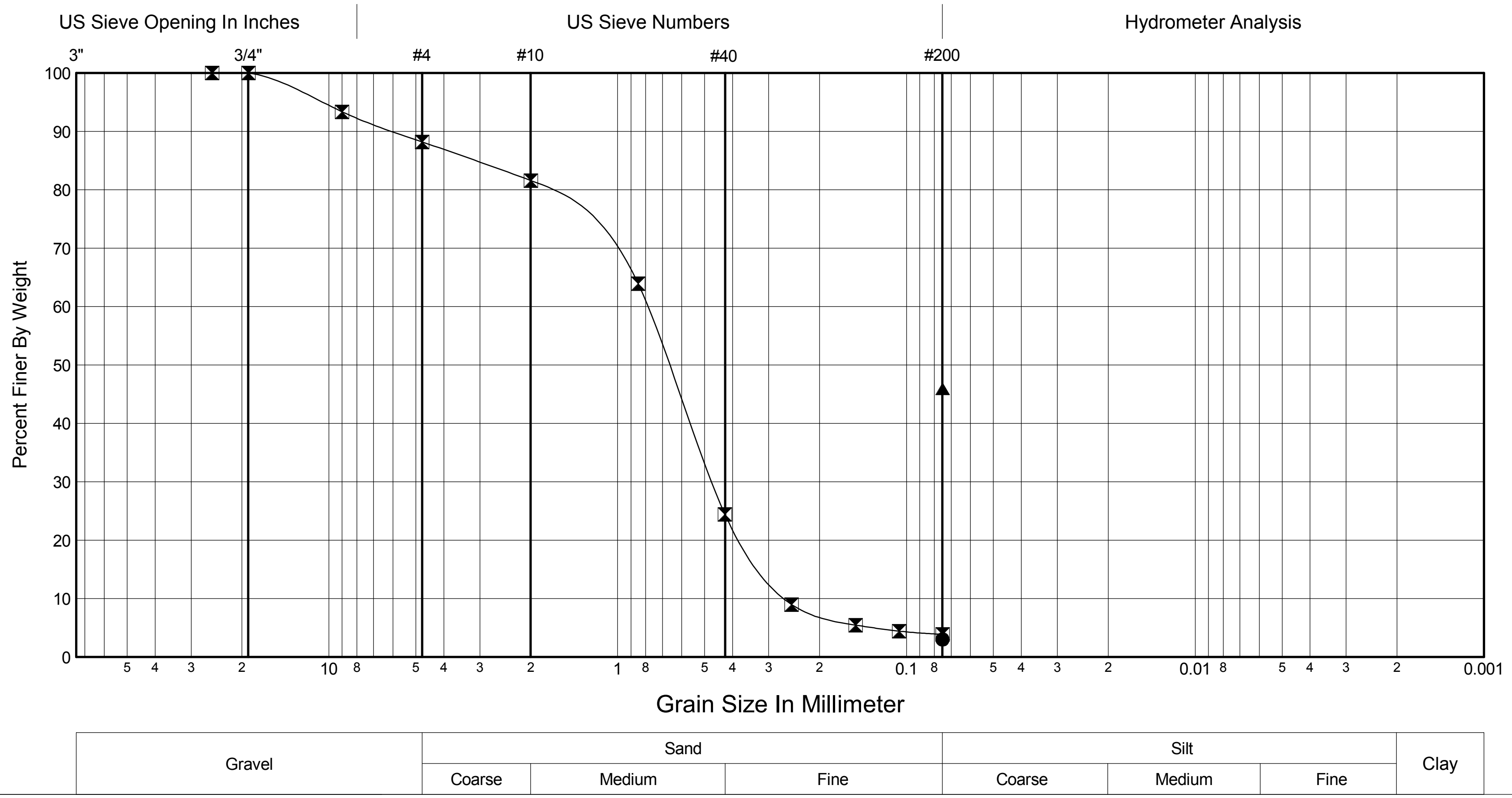
Job No. **24-1-04201-001**
Hole No. **B-4**
Project **South Kelso Railroad Grade Separation Project**

Date **May 29, 2018**
Sheet **C4**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**7.5**	**S-3**	**SP**	**Poorly graded SAND**	**28**						**-**	**-**	**3.0**							
⧗	**20.0**	**S-6B**	**SP**	**Poorly graded SAND with few to little gravel**	**24**						**11.8**	**84.3**	**3.9**	**1.1**	**3.1**	**0.793**	**0.67**	**0.47**	**0.37**	**0.259**
▲	**45.0**	**S-11B**	**SM**	**Silty SAND**	**43**						**-**	**-**	**45.9**							

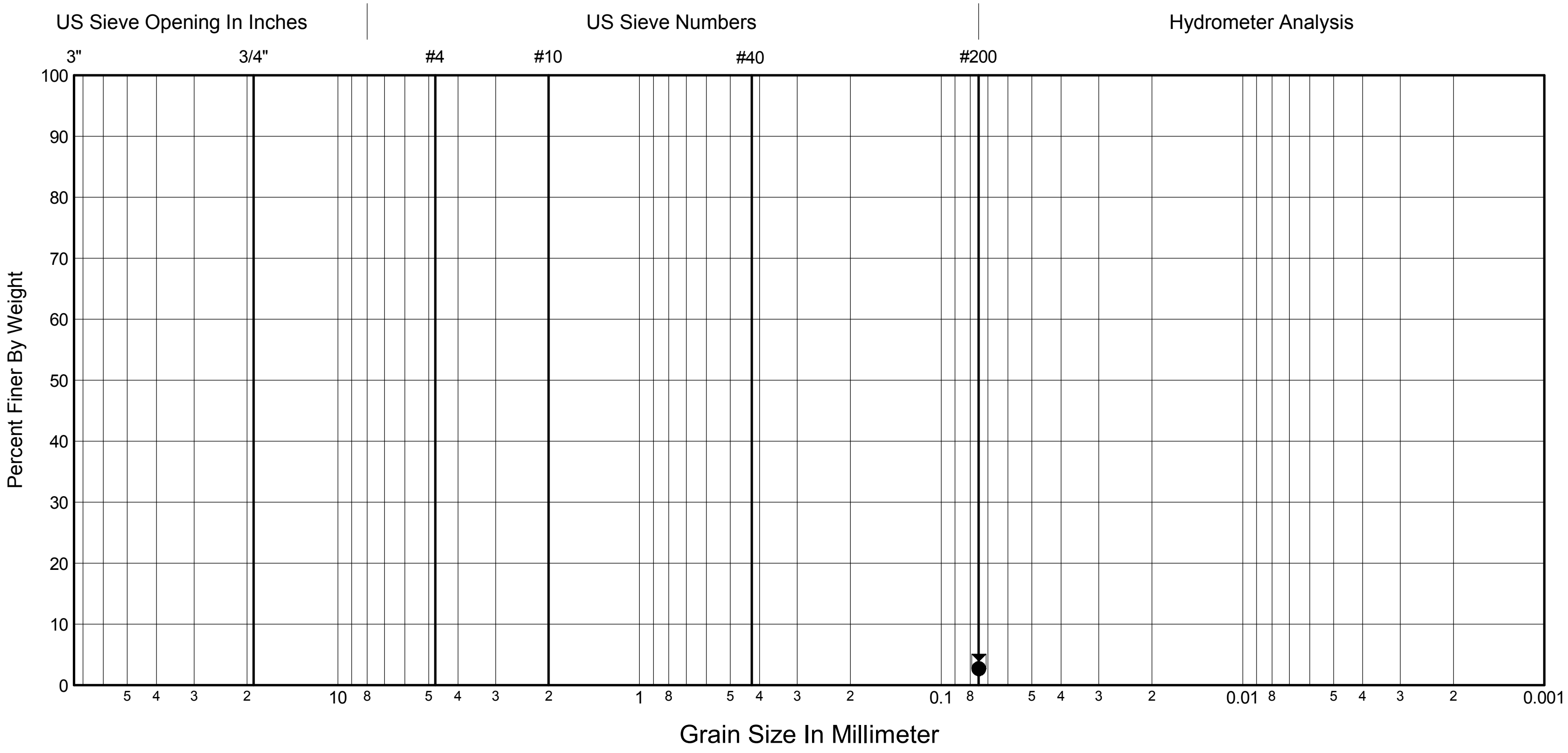


Job No. **24-1-04201-001** Date **May 29, 2018**
Hole No. **B-5** Sheet **C5**
Project **South Kelso Railroad Grade Separation Project**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**5.0**	**S-2**	**SP**	**Poorly graded SAND**	**26**						**-**	**-**	**2.7**							
⧗	**35.0**	**S-9**	**SP**	**Poorly graded SAND with gravel**	**25**						**-**	**-**	**3.9**							



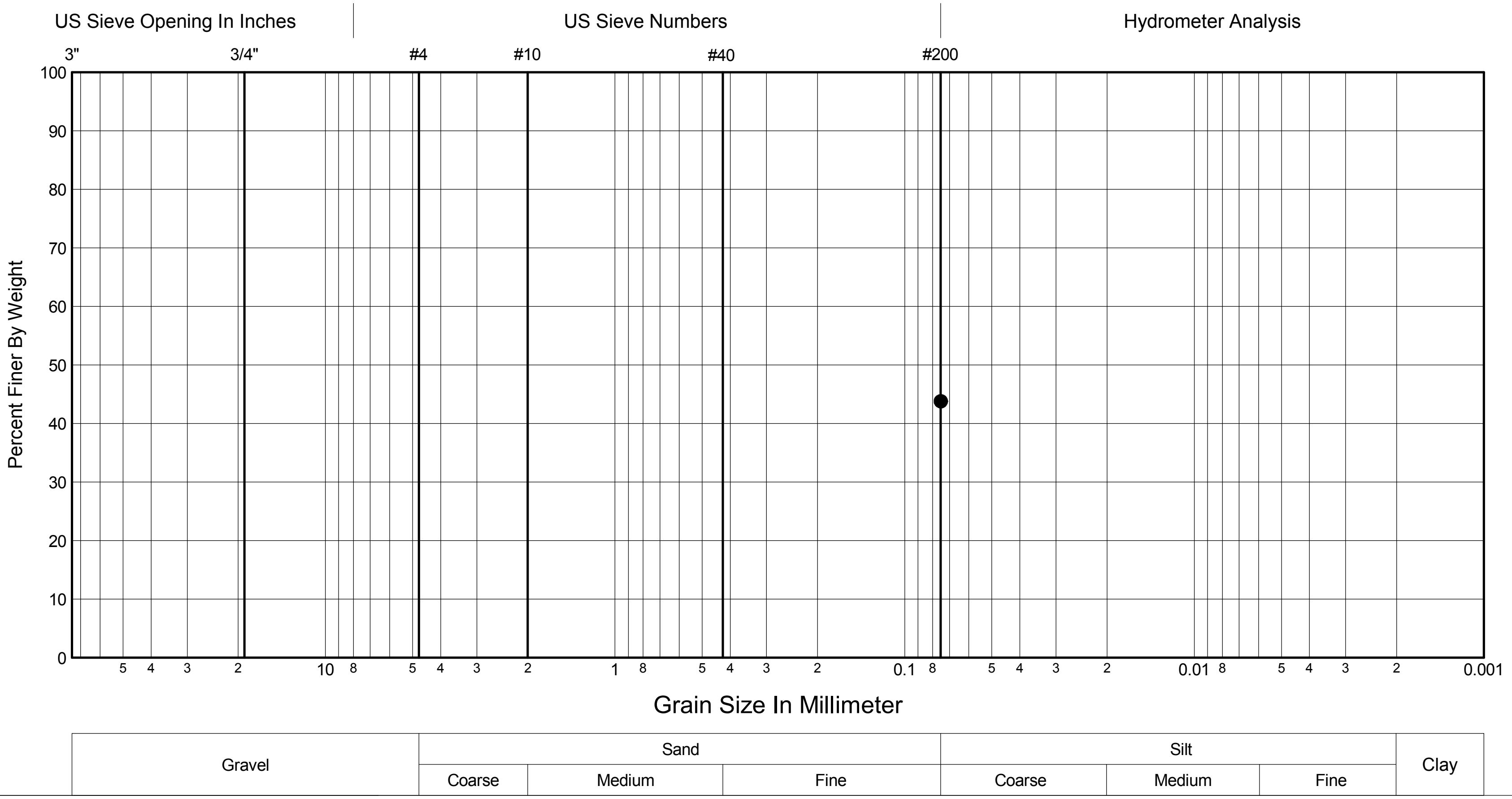
Gravel	Sand			Silt			Clay
	Coarse	Medium	Fine	Coarse	Medium	Fine	

Job No. **24-1-04201-001**
Date **May 29, 2018**
Hole No. **B-6**
Sheet **C6**
Project **South Kelso Railroad Grade Separation Project**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**2.5**	**S-1**	**SM**	**Silty SAND**	**20**						**-**	**-**	**43.8**							

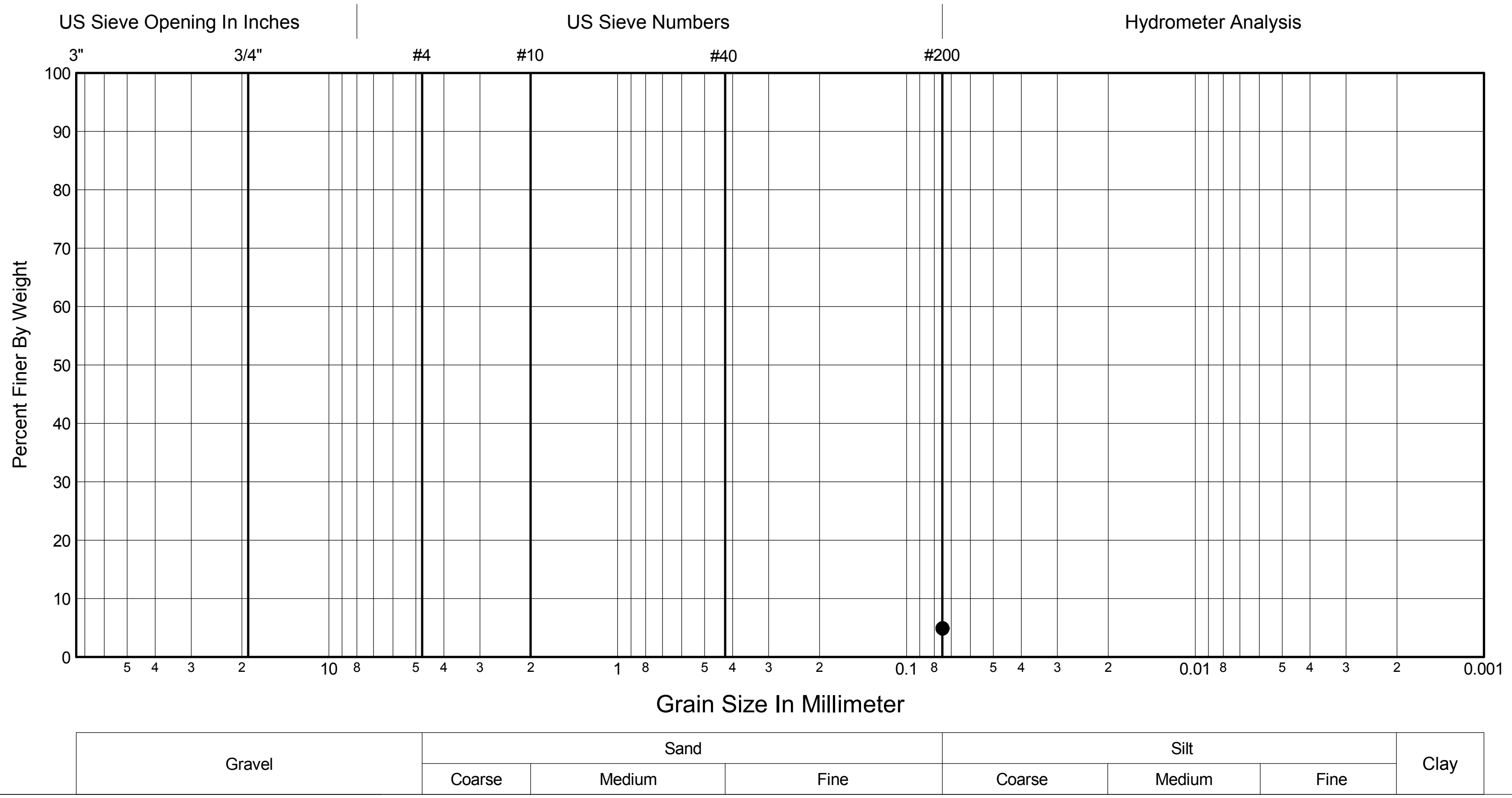


Job No. **24-1-04201-001** Date **May 29, 2018**
Hole No. **B-7** Sheet **C7**
Project **South Kelso Railroad Grade Separation Project**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**5.0**	**S-2**	**SP**	**Poorly graded SAND with gravel**	**12**						**-**	**-**	**4.9**							



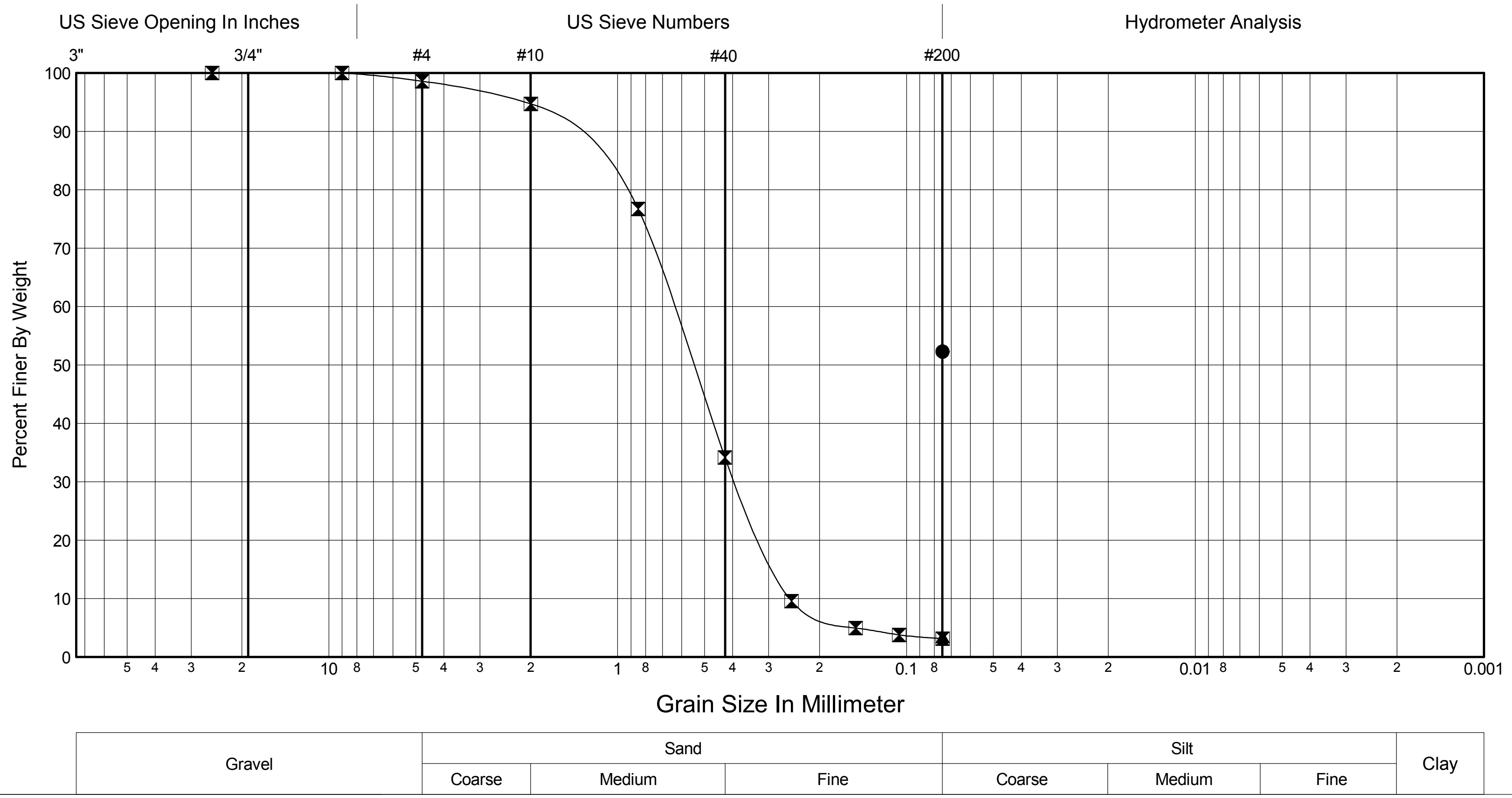
Job No. **24-1-04201-001**
Hole No. **B-8**
Project **South Kelso Railroad Grade Separation Project**

Date **May 29, 2018**
Sheet **C8**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**5.3**	**S-2A**	**ML**	**Sandy SILT**	**41**						**-**	**-**	**52.3**							
⧗	**15.0**	**S-5**	**SP**	**Poorly graded SAND with trace gravel**	**26**						**1.4**	**95.4**	**3.2**	**0.9**	**2.6**	**0.647**	**0.55**	**0.39**	**0.31**	**0.252**
▲	**100.0**	**S-24**	**SP**	**Poorly graded SAND**	**24**						**-**	**-**	**3.4**							

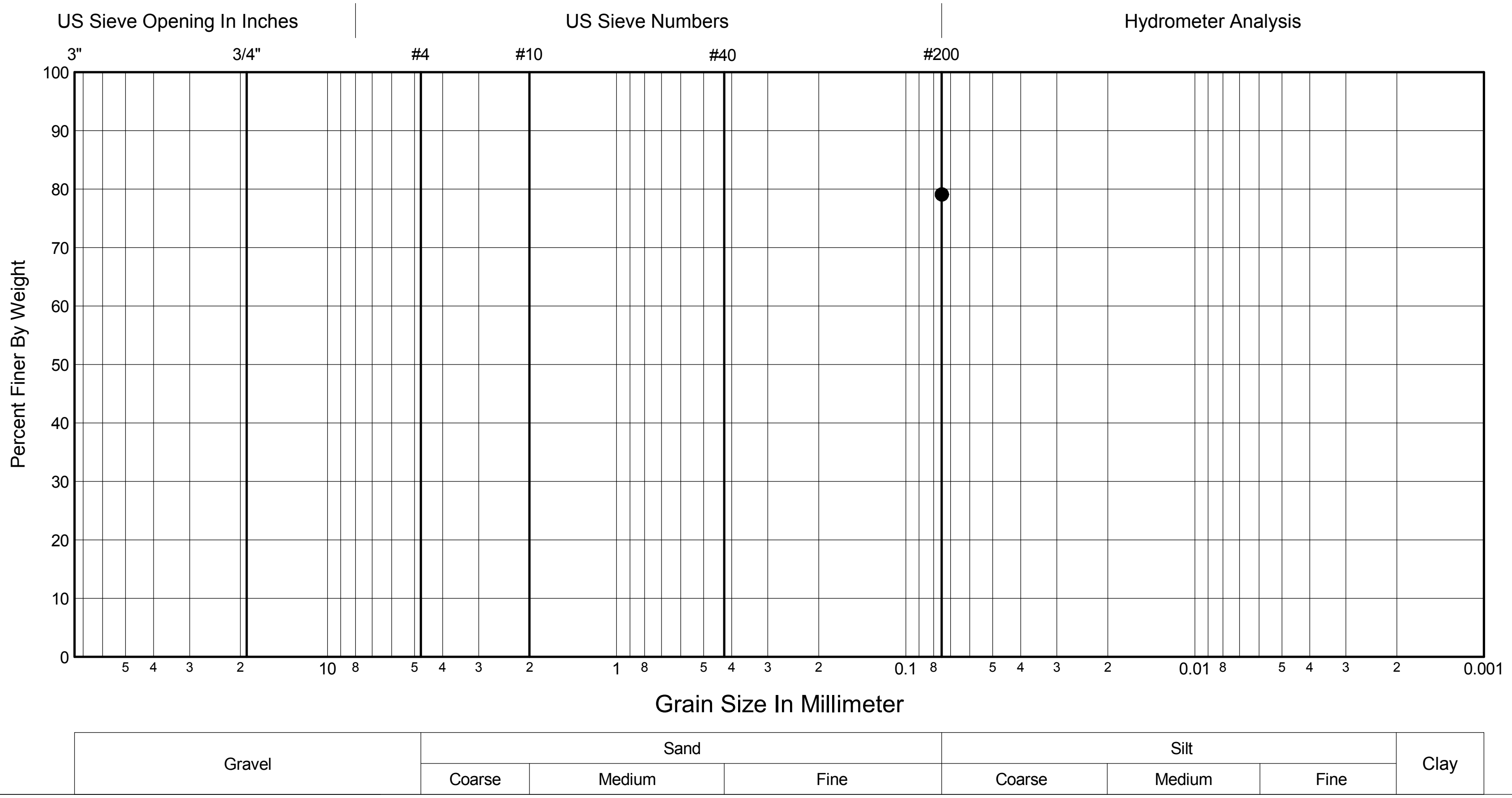


Job No. **24-1-04201-001** Date **May 29, 2018**
Hole No. **B-9** Sheet **C9**
Project **South Kelso Railroad Grade Separation Project**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**7.5**	**S-3C**	**ML**	**SILT with Sand**	**41**						**-**	**-**	**79.1**							



Job No. **24-1-04201-001** Date **May 29, 2018**

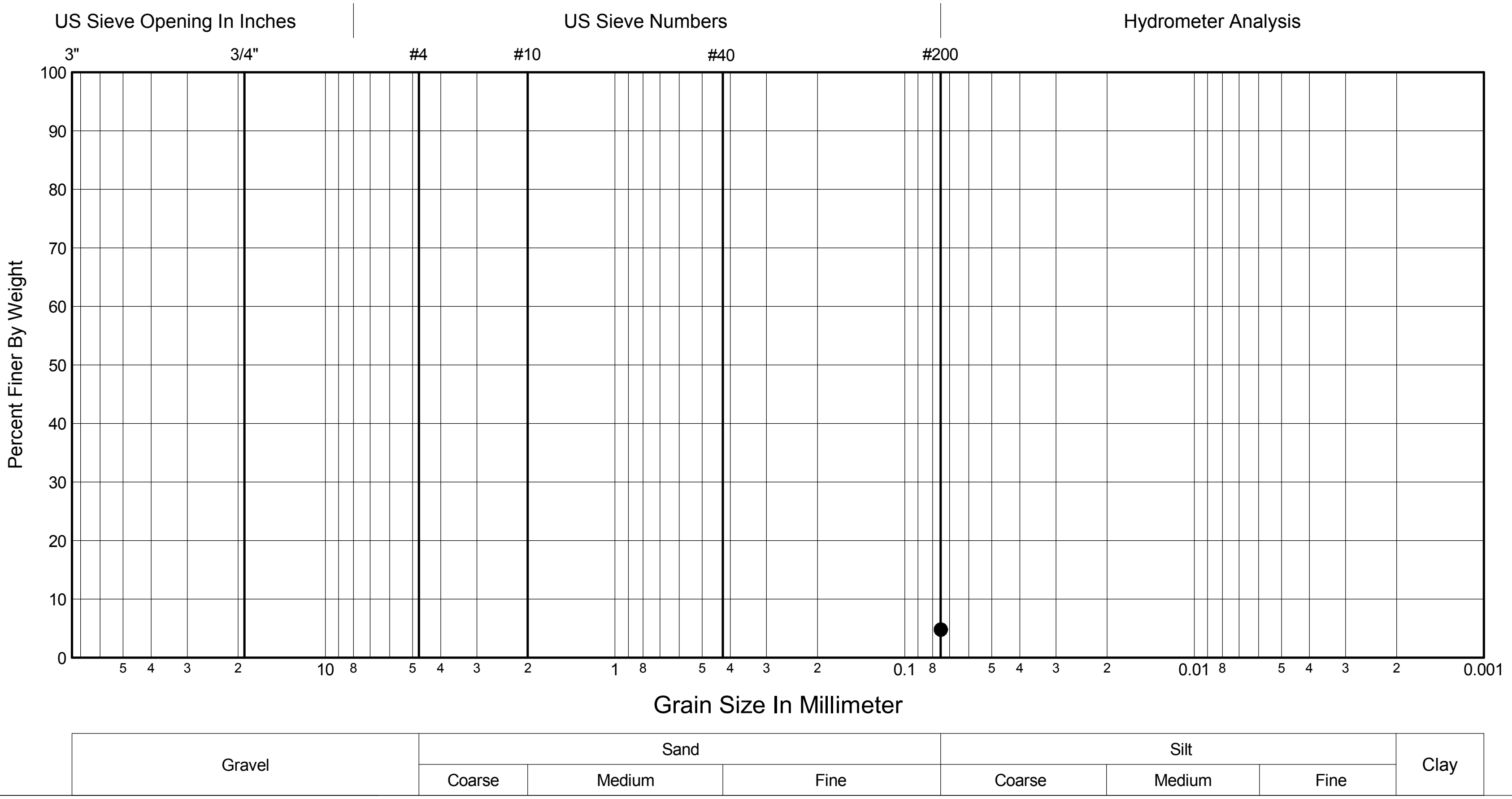
Hole No. **B-10** Sheet **C10**

Project **South Kelso Railroad Grade Separation Project**

Laboratory Summary

Washington State
Department of Transportation

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	**5.0**	**S-2**	**SP**	**Poorly graded SAND with gravel**	**28**						**-**	**-**	**4.8**							



APPENDIX D

GLOBAL STABILITY ANALYSIS RESULTS

APPENDIX D

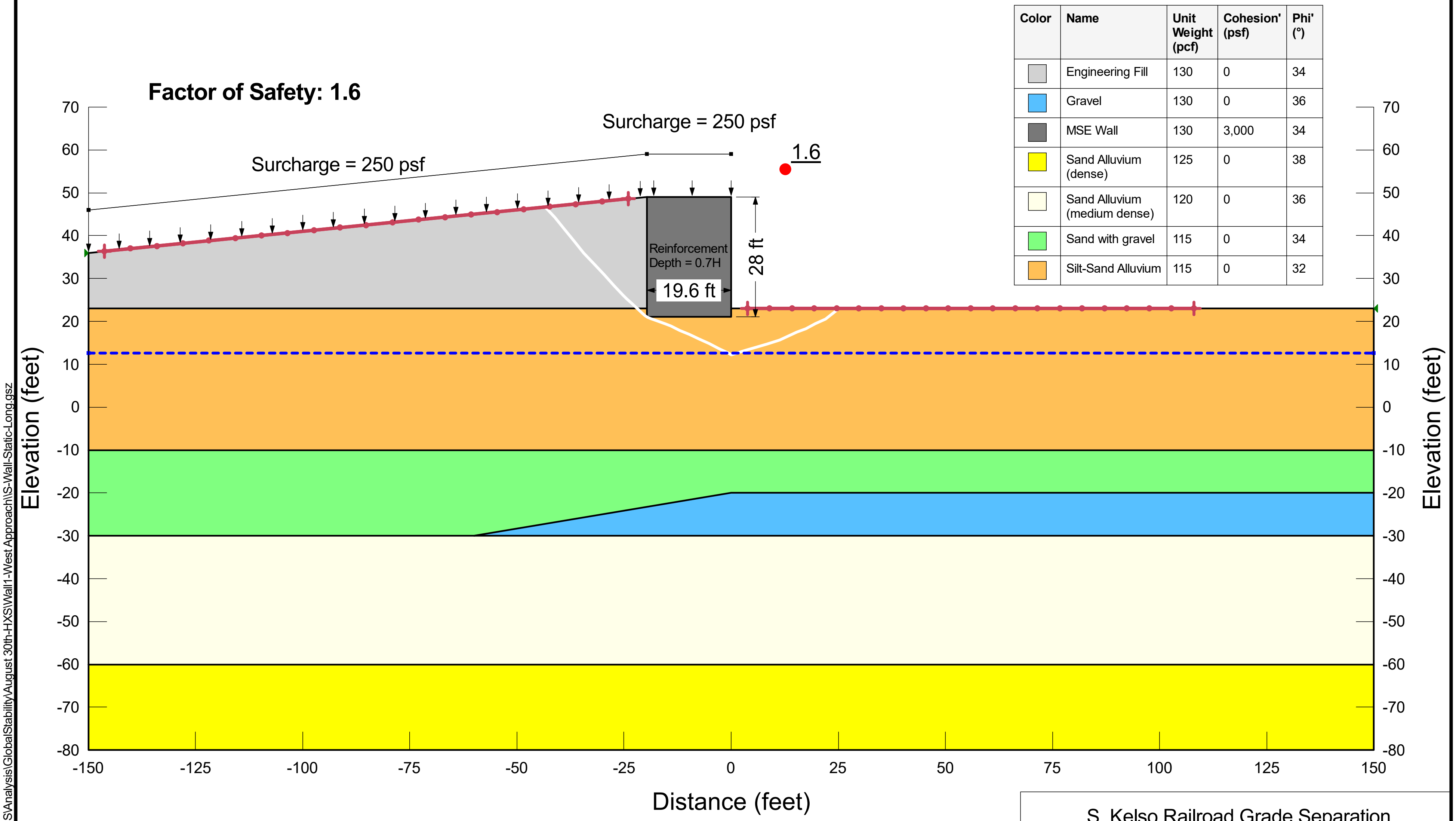
GLOBAL STABILITY ANALYSIS RESULTS

FIGURES

D1 Global Stability Analysis, Wall 1 West Approach, Static,
D2 Global Stability Analysis, Wall 1 West Approach, Seismic
D3 Global Stability Analysis, Wall 1 West Approach, Post-Seismic
D4 Global Stability Analysis, Wall 1 West Approach, Post-Seismic with Geofoam
D5 Global Stability Analysis, Wall 1 West Approach, Post-Seismic with Improved Soil

D6 Global Stability Analysis, Wall 2 East Approach, Static,
D7 Global Stability Analysis, Wall 2 East Approach, Seismic
D8 Global Stability Analysis, Wall 2 East Approach, Post-Seismic
D9 Global Stability Analysis, Wall 2 East Approach, Post-Seismic with Geofoam
D10 Global Stability Analysis, Wall 2 East Approach, Post-Seismic with Improved Soil

D11 Global Stability Analysis, Railway Track Embankment, Post-Seismic with Improved Soil



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Engineering Fill	130	0	34
🟦	Gravel	130	0	36
⬛	MSE Wall	130	3,000	34
🟨	Sand Alluvium (dense)	125	0	38
⬜	Sand Alluvium (medium dense)	120	0	36
🟩	Sand with gravel	115	0	34
🟧	Silt-Sand Alluvium	115	0	32

S. Kelso Railroad Grade Separation

South Kelso, Washington

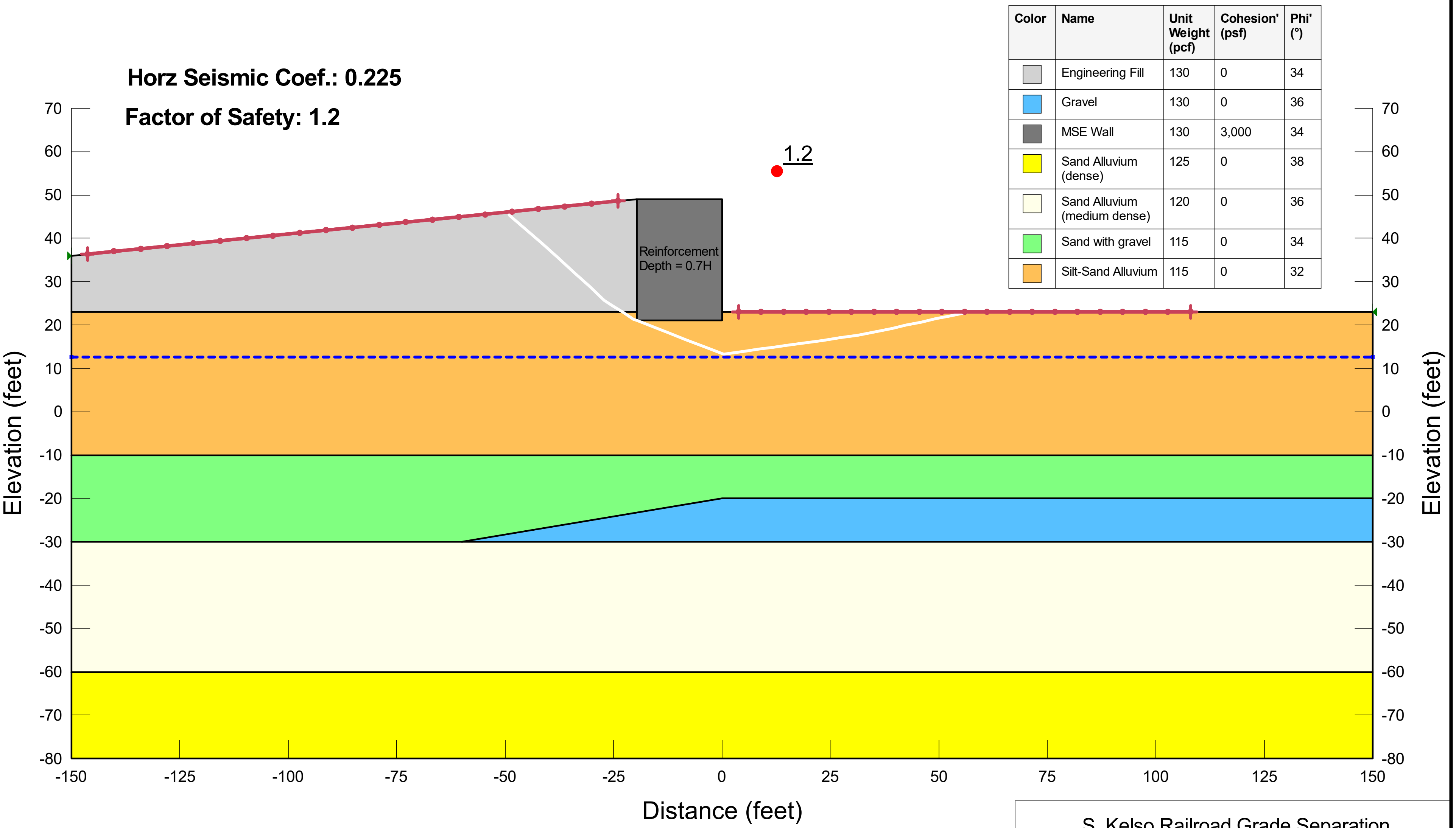
SLOPE STABILITY ANALYSIS
Wall 1 West Approach
Static

September 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D1

I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall1-West Approach\S-Wall-Static-Long.gsz



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
◻	Engineering Fill	130	0	34
🟦	Gravel	130	0	36
⬛	MSE Wall	130	3,000	34
🟨	Sand Alluvium (dense)	125	0	38
⬜	Sand Alluvium (medium dense)	120	0	36
🟩	Sand with gravel	115	0	34
🟧	Silt-Sand Alluvium	115	0	32

S. Kelso Railroad Grade Separation

South Kelso, Washington

SLOPE STABILITY ANALYSIS
Wall 1 West Approach
Seismic

September 2018
24-1-04201-001

SHANNON & WILSON, INC.
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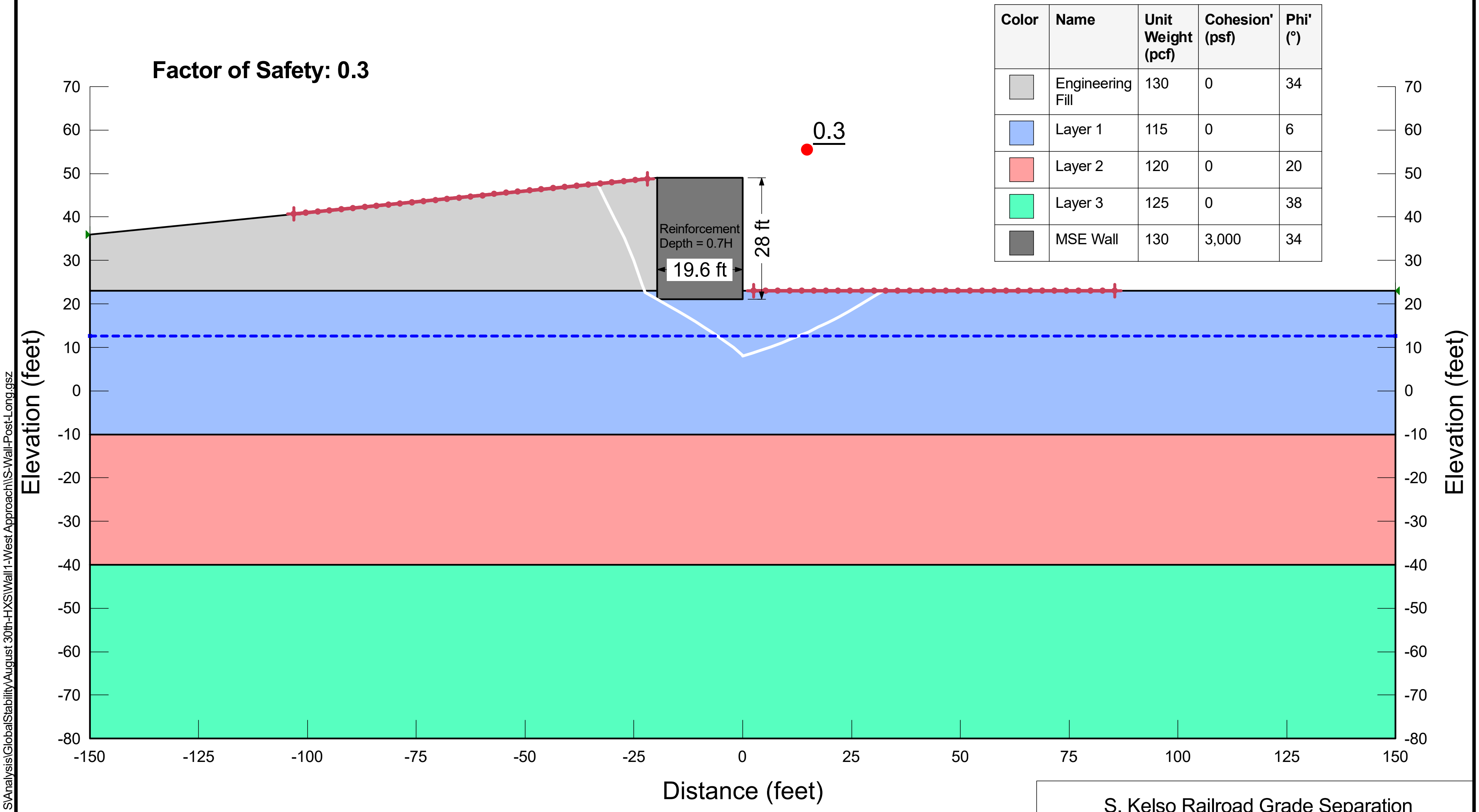
FIG. D2

I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall1-West Approach\S-Wall-Seismic-Long.gsz



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Engineering Fill	130	0	34
🟦	Layer 1	115	0	6
🟥	Layer 2	120	0	20
🟩	Layer 3	125	0	38
⬛	MSE Wall	130	3,000	34

I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall1-West Approach\S-Wall-Post-Long.gsz

S. Kelso Railroad Grade Separation
South Kelso, Washington

SLOPE STABILITY ANALYSIS
Wall 1 West Approach
Post-Seismic

September 2018
24-1-04201-001

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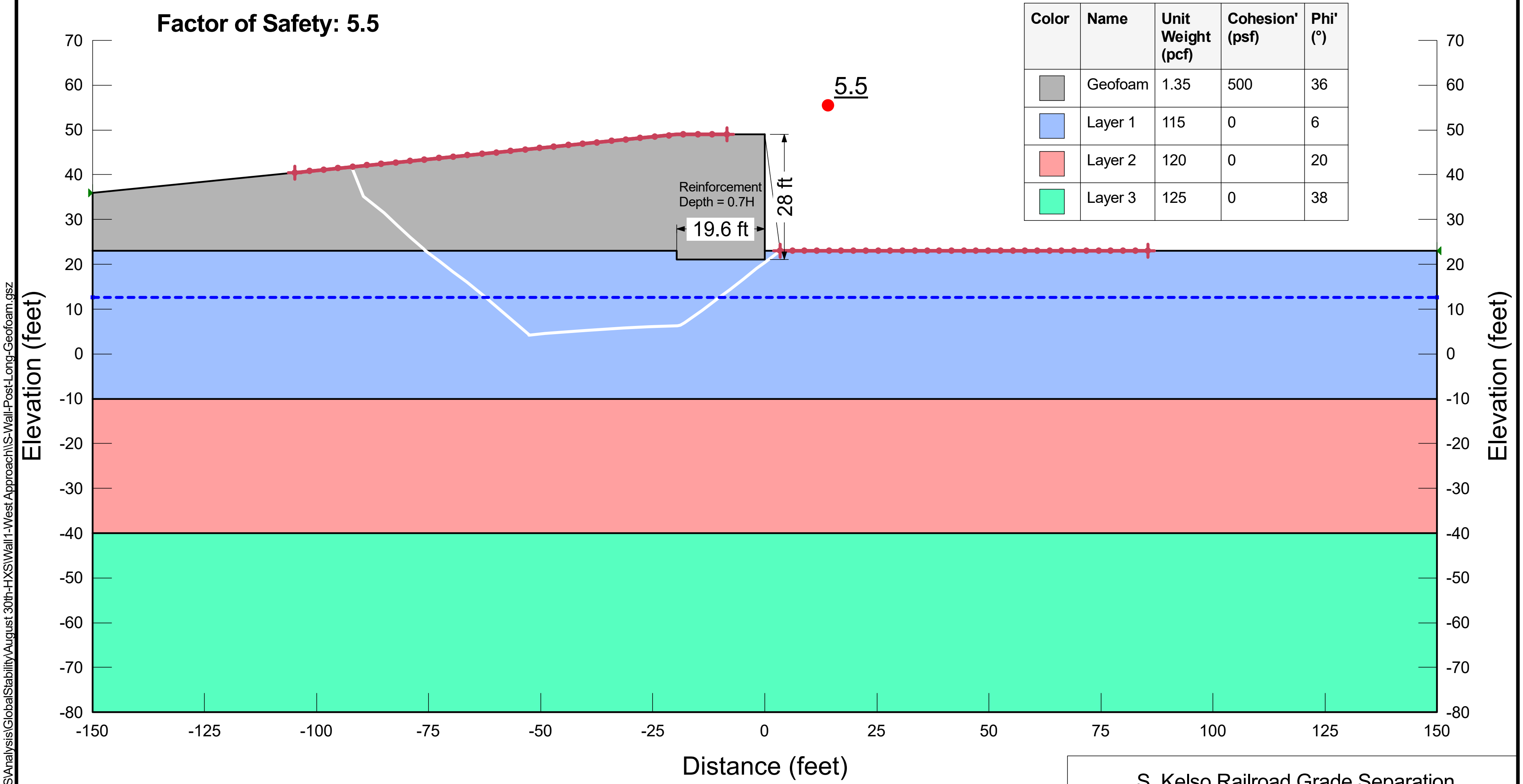
FIG. D3



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Geofoam	1.35	500	36
🟦	Layer 1	115	0	6
🟥	Layer 2	120	0	20
🟩	Layer 3	125	0	38

I:\EF24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall1-West Approach\S-Wall-Post-Long-Geofoam.gsz

S. Kelso Railroad Grade Separation
South Kelso, Washington

SLOPE STABILITY ANALYSIS
Wall 1 West Approach
Post-Seismic with Geofoam

September 2018 24-1-04201-001

SHANNON & WILSON, INC.
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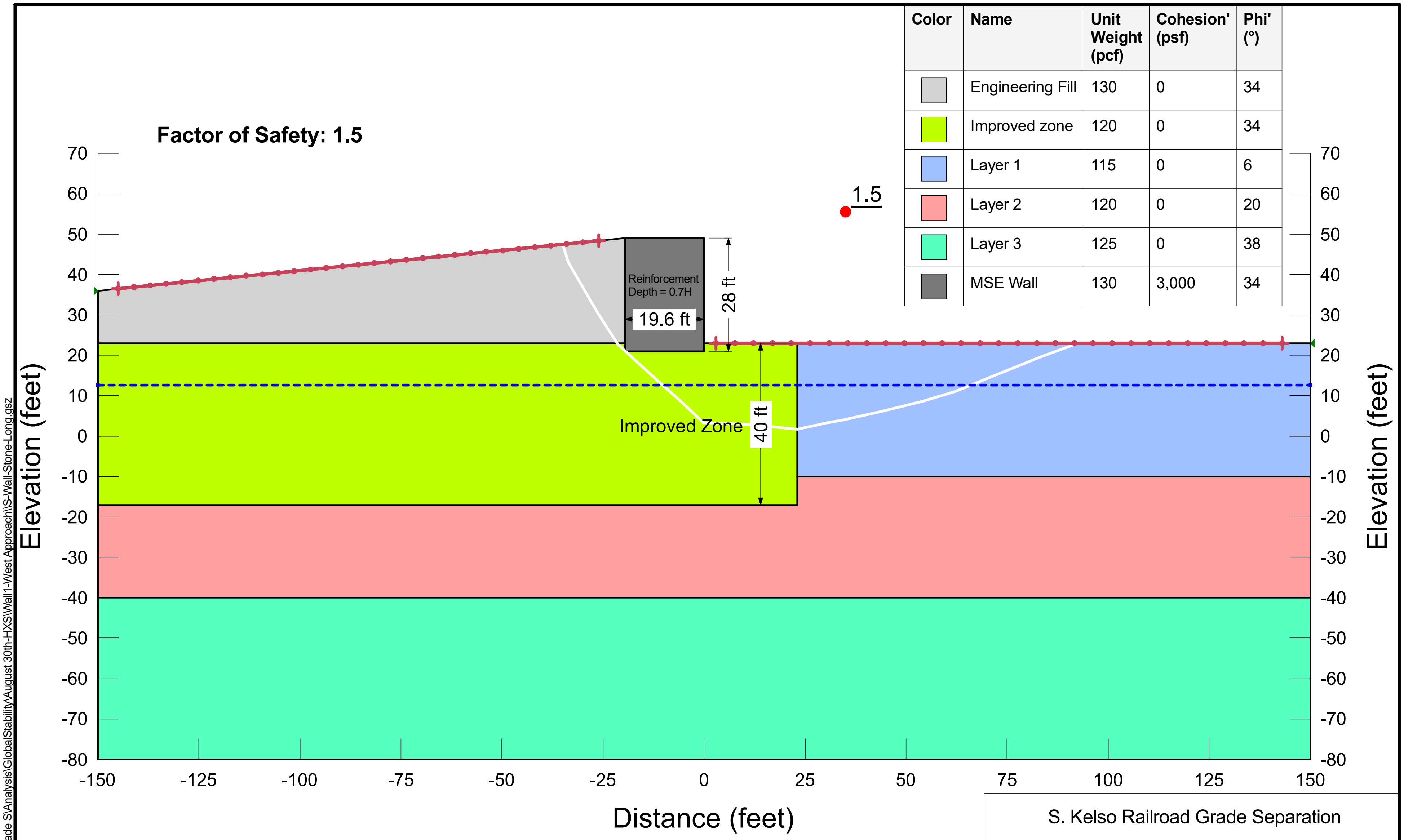
FIG. D4



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Engineering Fill	130	0	34
🟩	Improved zone	120	0	34
🟦	Layer 1	115	0	6
🟥	Layer 2	120	0	20
🟩	Layer 3	125	0	38
⬛	MSE Wall	130	3,000	34

S. Kelso Railroad Grade Separation

South Kelso, Washington

SLOPE STABILITY ANALYSIS
Wall 1 West Approach
Post-Seismic with Improved Soil

September 2018 24-1-04201-001

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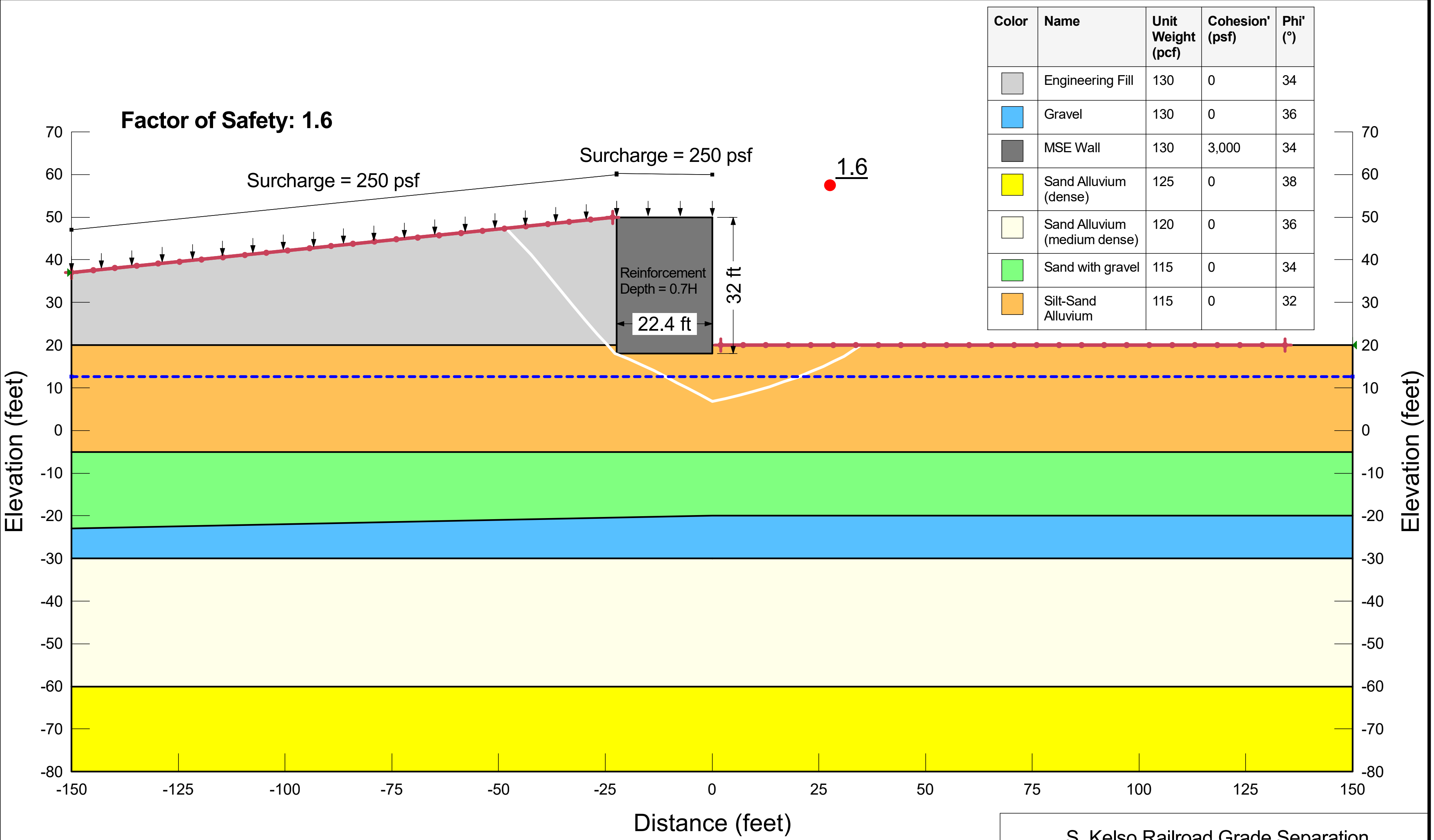
FIG. D5

I:\EF24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall1-West Approach\S-Wall-Stone-Long.gsz



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Engineering Fill	130	0	34
🟦	Gravel	130	0	36
⬛	MSE Wall	130	3,000	34
🟨	Sand Alluvium (dense)	125	0	38
⬜	Sand Alluvium (medium dense)	120	0	36
🟩	Sand with gravel	115	0	34
🟧	Silt-Sand Alluvium	115	0	32

I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall2-East Approach\N-Wall-Static-Long.gsz

S. Kelso Railroad Grade Separation
South Kelso, Washington

SLOPE STABILITY ANALYSIS
Wall 2 East Approach
Static

September 2018
24-1-04201-001

SHANNON & WILSON, INC.
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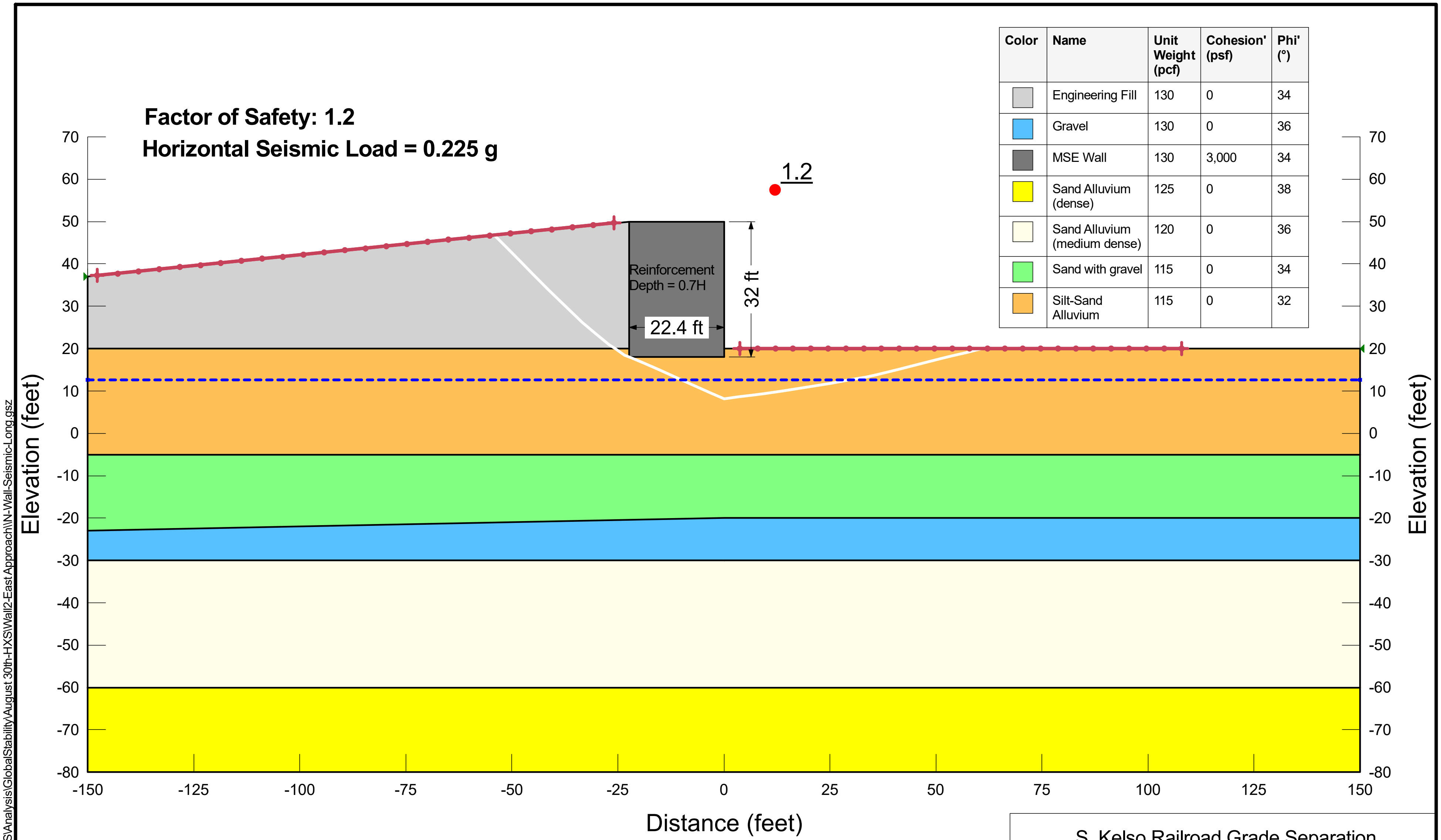
FIG. D6



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Engineering Fill	130	0	34
🟦	Gravel	130	0	36
⬛	MSE Wall	130	3,000	34
🟨	Sand Alluvium (dense)	125	0	38
⬜	Sand Alluvium (medium dense)	120	0	36
🟩	Sand with gravel	115	0	34
🟧	Silt-Sand Alluvium	115	0	32

I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall2-East Approach\N-Wall-Seismic-Long.gsz

S. Kelso Railroad Grade Separation

South Kelso, Washington

SLOPE STABILITY ANALYSIS
Wall 2 East Approach
Seismic

September 2018 24-1-04201-001

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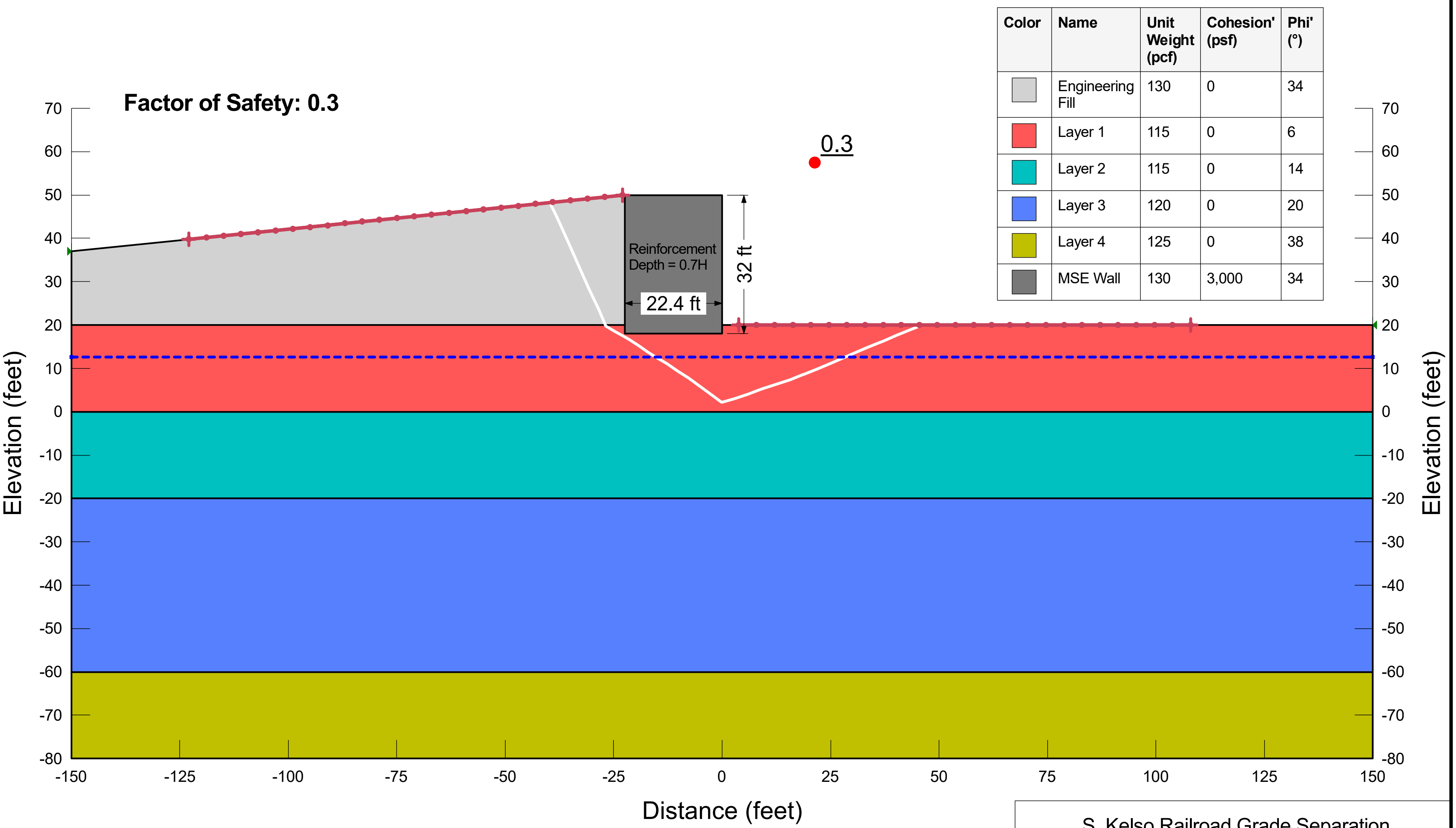
FIG. D7

I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall2-East Approach\N-Wall-Post-Long.gsz



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Engineering Fill	130	0	34
🟥	Layer 1	115	0	6
🟦	Layer 2	115	0	14
🟦	Layer 3	120	0	20
🟨	Layer 4	125	0	38
⬛	MSE Wall	130	3,000	34

S. Kelso Railroad Grade Separation

South Kelso, Washington

SLOPE STABILITY ANALYSIS
Wall 2 East Approach
Post-Seismic

September 2018 24-1-04201-001

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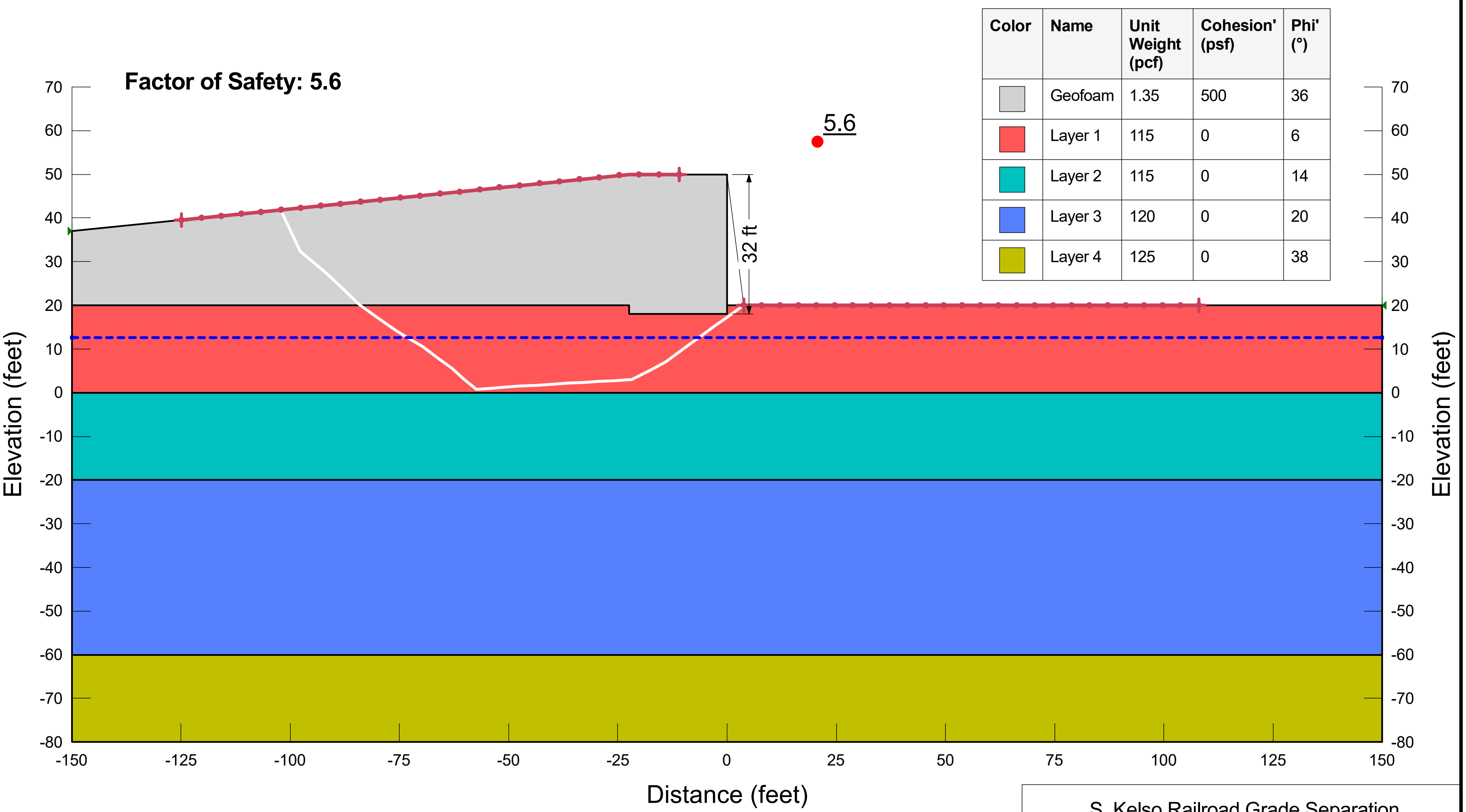
FIG. D8

I:\EF24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall2-East Approach\N-Wall-Post-Long-Geofoam.gsz



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Geofoam	1.35	500	36
🟥	Layer 1	115	0	6
🟦	Layer 2	115	0	14
🟦	Layer 3	120	0	20
🟨	Layer 4	125	0	38

S. Kelso Railroad Grade Separation

South Kelso, Washington

SLOPE STABILITY ANALYSIS
Wall 2 East Approach
Post-Seismic with Geofoam

September 2018　24-1-04201-001

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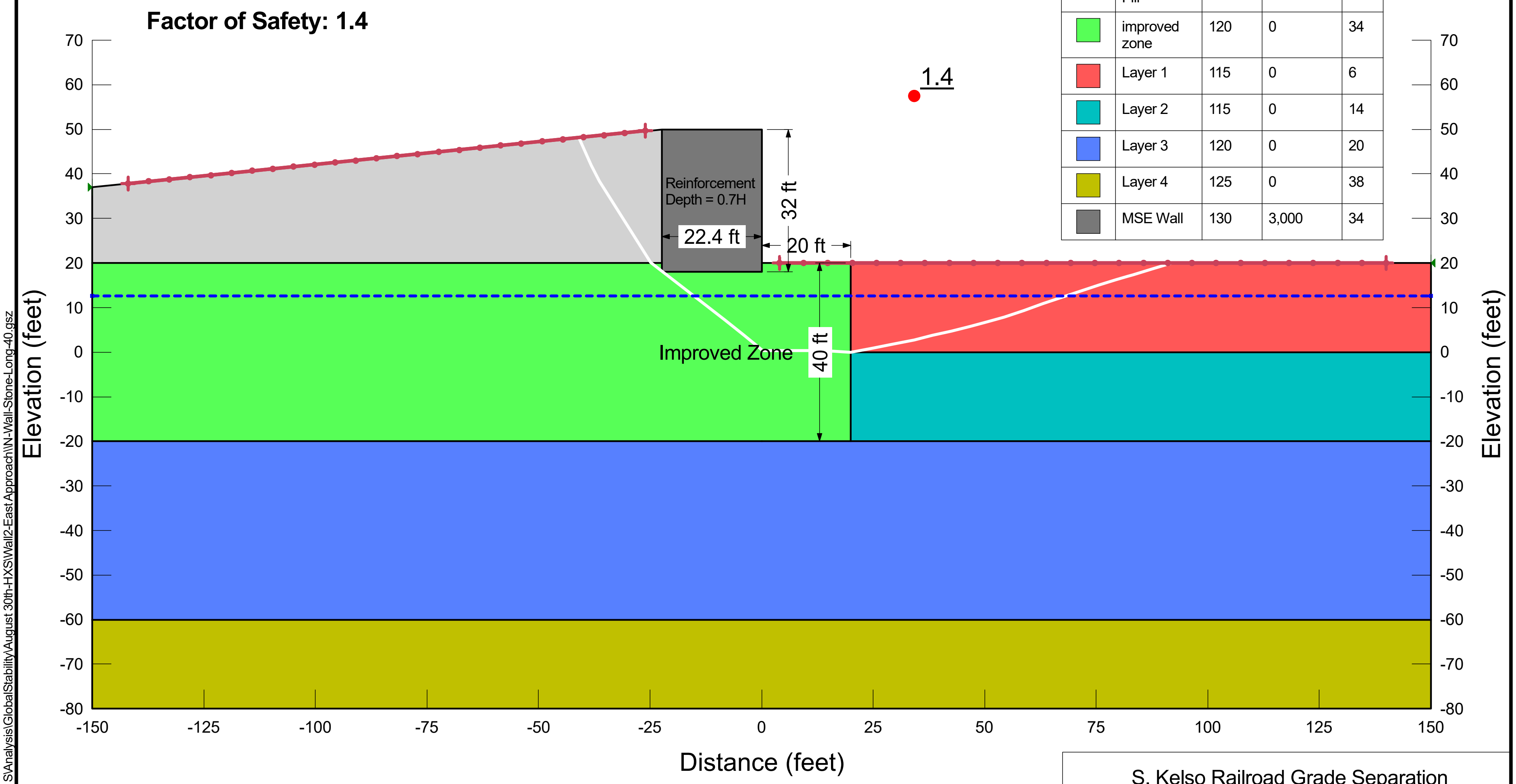
FIG. D9



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Engineering Fill	130	0	34
🟩	improved zone	120	0	34
🟥	Layer 1	115	0	6
🟦	Layer 2	115	0	14
🟦	Layer 3	120	0	20
🟨	Layer 4	125	0	38
⬛	MSE Wall	130	3,000	34

S. Kelso Railroad Grade Separation
South Kelso, Washington

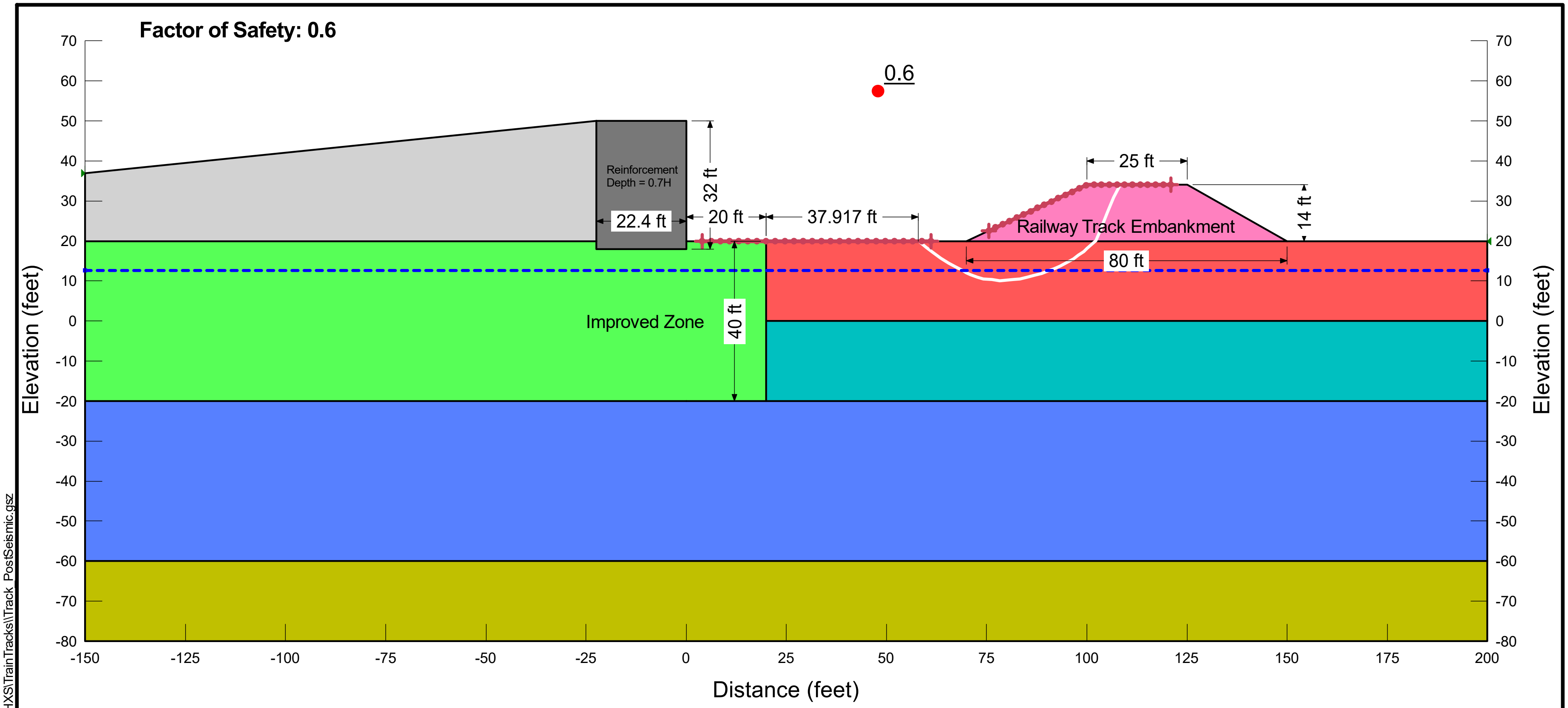
SLOPE STABILITY ANALYSIS
Wall 2 East Approach
Post-Seismic with Improved Soil

September 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D10

I:\EF24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\Wall2-East Approach\N-Wall-Stone-Long-40.gsz



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
⬜	Engineering Fill	130	0	34
🟩	improved zone	120	0	34
🟥	Layer 1	115	0	6
🟦	Layer 2	115	0	14
🟦	Layer 3	120	0	20
🟨	Layer 4	125	0	38
⬛	MSE Wall	130	3,000	34
🟪	Tracks	125	0	32

S. Kelso Railroad Grade Separation

South Kelso, Washington

SLOPE STABILITY ANALYSIS
Railway Track Embankment
Post-Seismic with Improved Soil

September 2018 24-1-04201-001

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FIG. D11

I:\EF\24-1 PDX\04200s\04201 S Kelso Grade S\Analysis\GlobalStability\August 30th-HXS\TrainTracks\Track PostSeismic.gsz

APPENDIX E

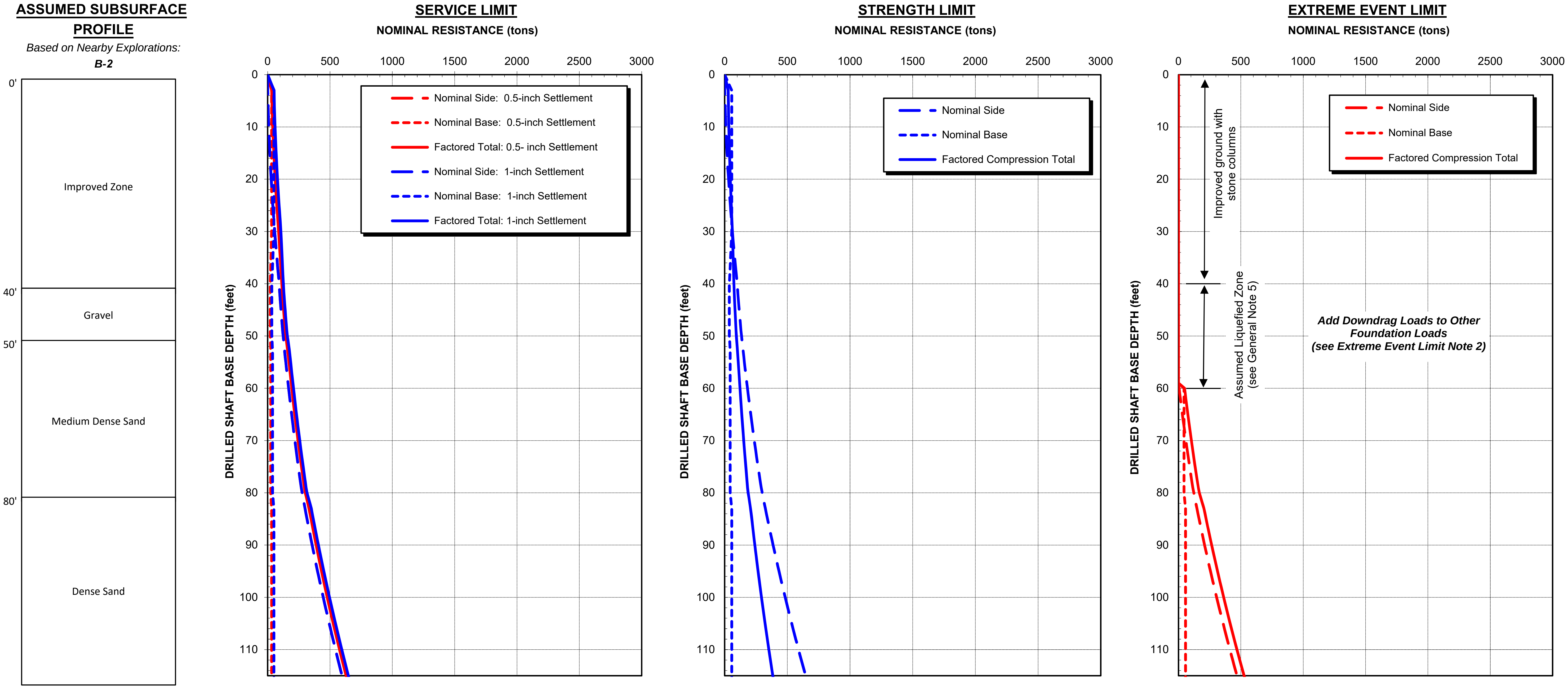
AXIAL SHAFT RESISTANCES

APPENDIX E

AXIAL SHAFT RESISTANCES

FIGURES

E1 Estimated Axial Shaft Resistance, 2-foot Diameter Drilled Shaft, Pier 1
E2 Estimated Axial Shaft Resistance, 3-foot Diameter Drilled Shaft, Pier 1
E3 Estimated Axial Shaft Resistance, 4-foot Diameter Drilled Shaft, Pier 1
E4 Estimated Axial Shaft Resistance, 5-foot Diameter Drilled Shaft, Pier 1
E5 Estimated Axial Shaft Resistance, 6-foot Diameter Drilled Shaft, Pier 1
E6 Estimated Axial Shaft Resistance, 2-foot Diameter Drilled Shaft, Pier 4
E7 Estimated Axial Shaft Resistance, 3-foot Diameter Drilled Shaft, Pier 4
E8 Estimated Axial Shaft Resistance, 4-foot Diameter Drilled Shaft, Pier 4
E9 Estimated Axial Shaft Resistance, 5-foot Diameter Drilled Shaft, Pier 4
E10 Estimated Axial Shaft Resistance, 6-foot Diameter Drilled Shaft, Pier 4
E11 Estimated Axial Shaft Resistance, 6-foot Diameter Drilled Shaft, Piers 2 and 3
E12 Estimated Axial Shaft Resistance, 7-foot Diameter Drilled Shaft, Piers 2 and 3



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 140 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
2-foot Diameter Drilled Shaft
Pier 1

June 2018 24-1-04201-001

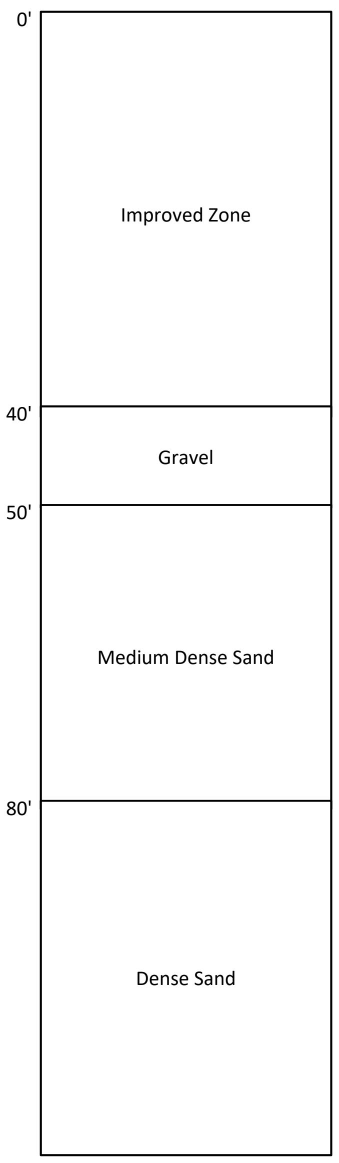
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E1

ASSUMED SUBSURFACE PROFILE

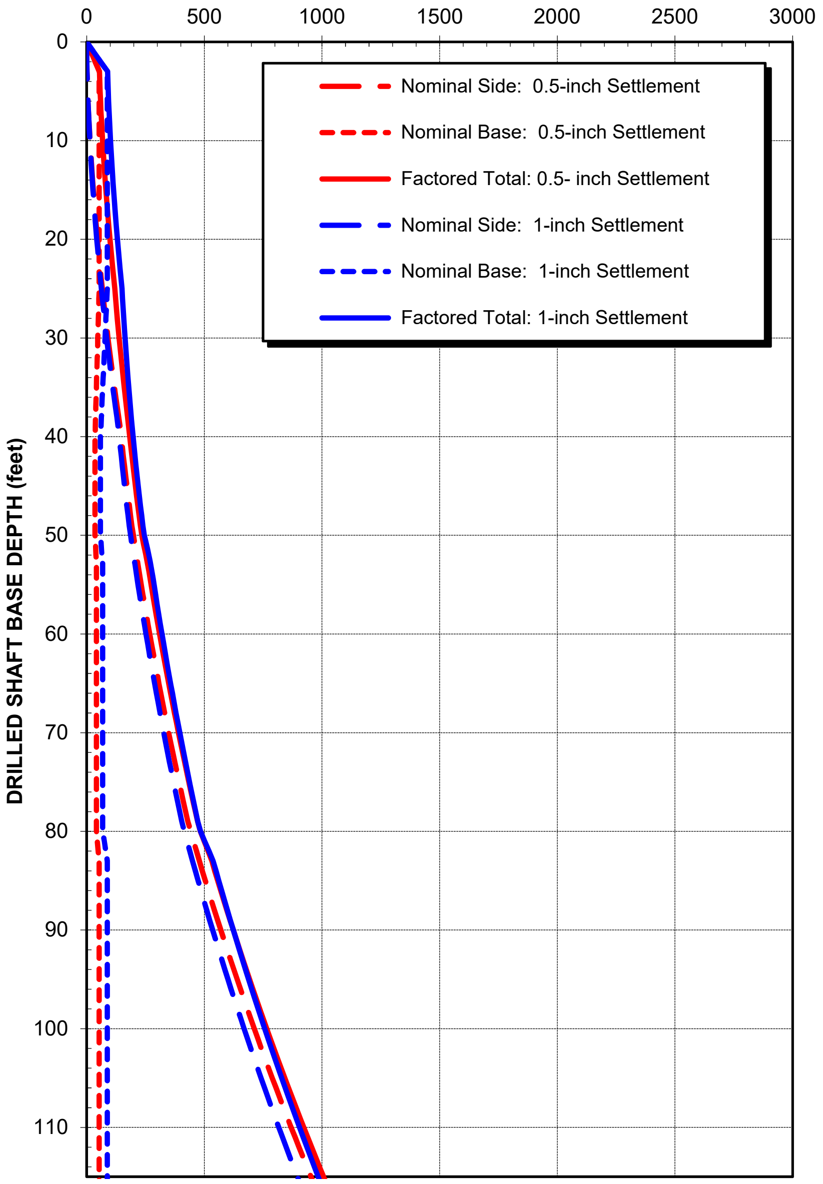
Based on Nearby Explorations:

B-2



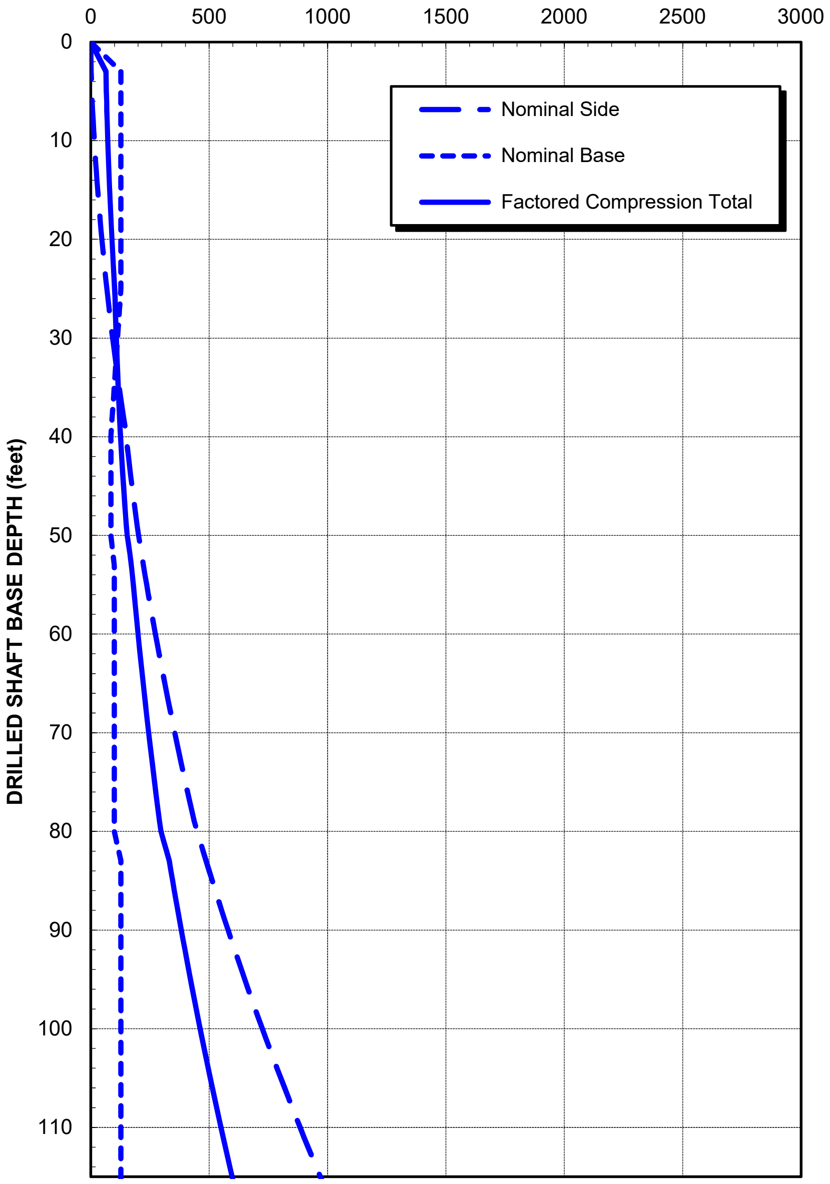
SERVICE LIMIT

NOMINAL RESISTANCE (tons)



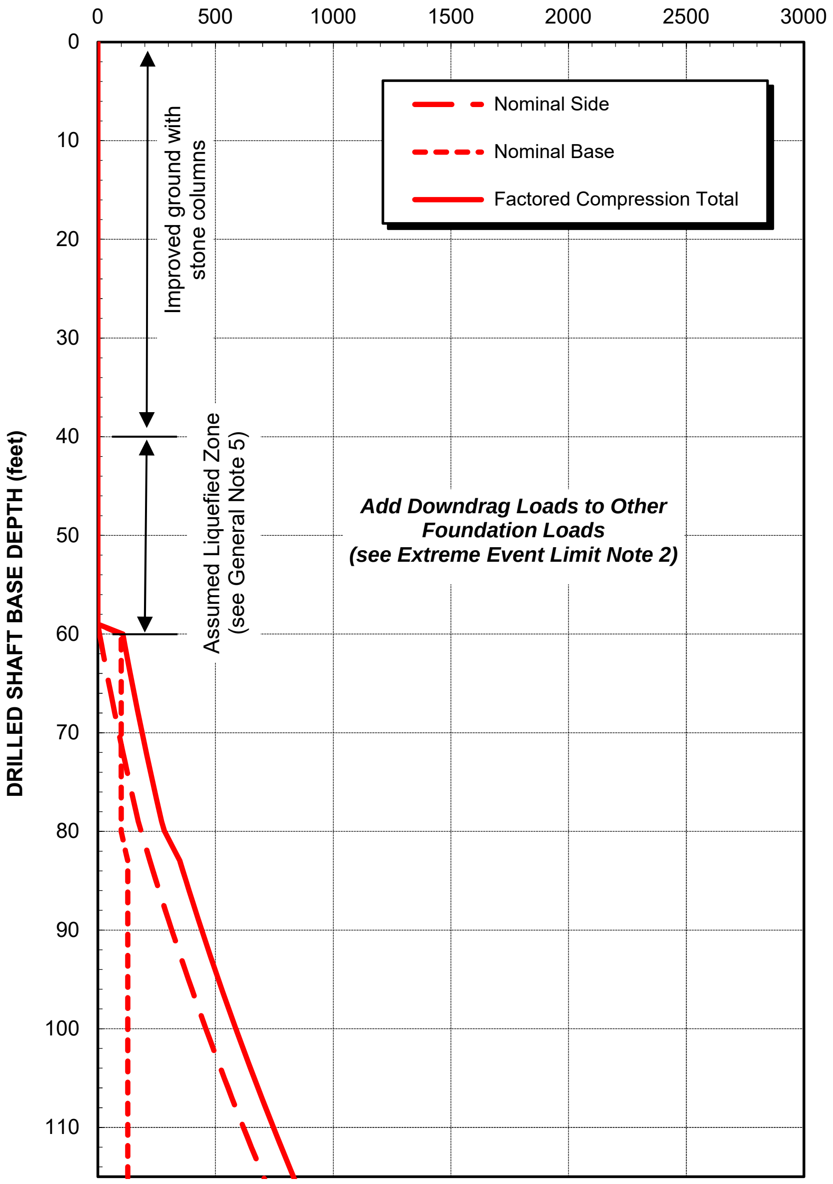
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 210 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
3-foot Diameter Drilled Shaft
Pier 1

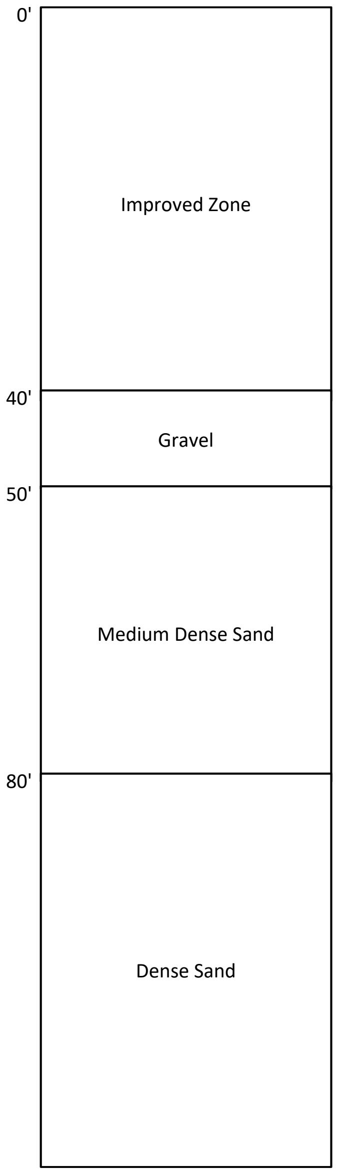
June 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E2

ASSUMED SUBSURFACE PROFILE

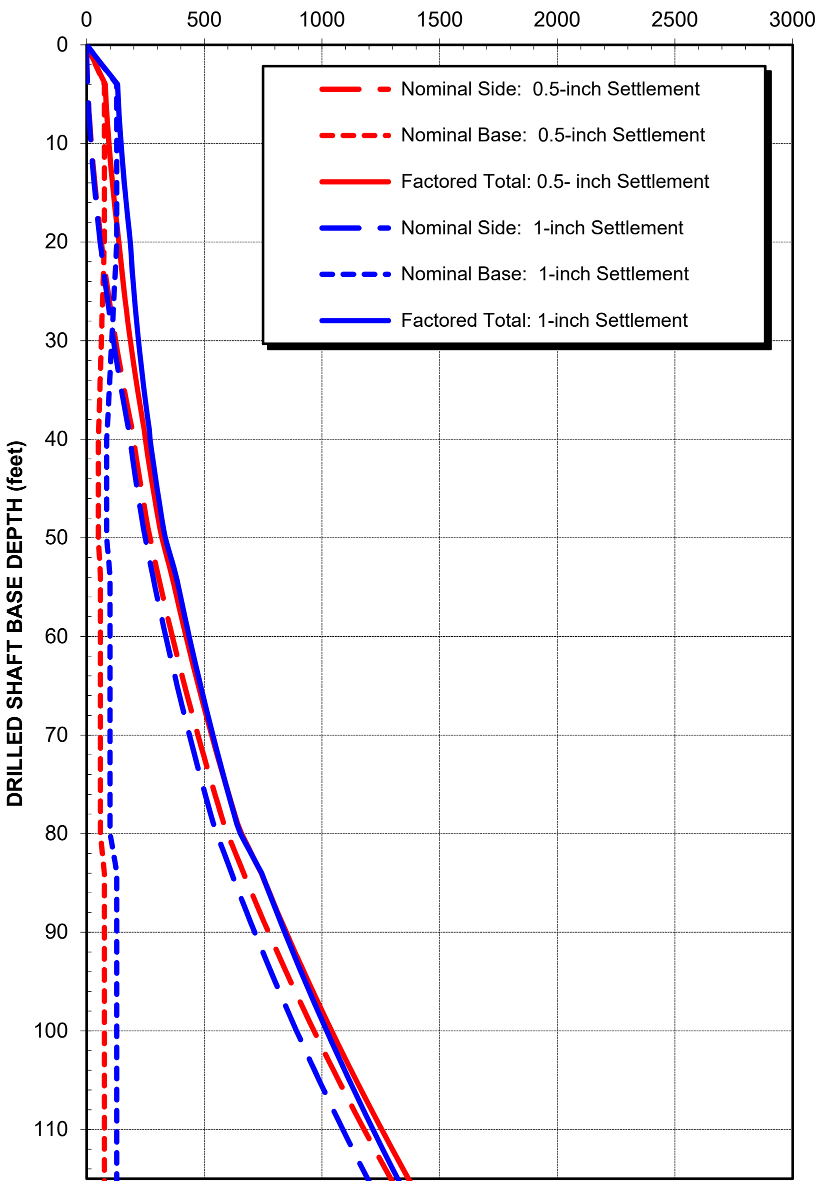
Based on Nearby Explorations:
B-2



Boring Extends to 140.0 feet

SERVICE LIMIT

NOMINAL RESISTANCE (tons)

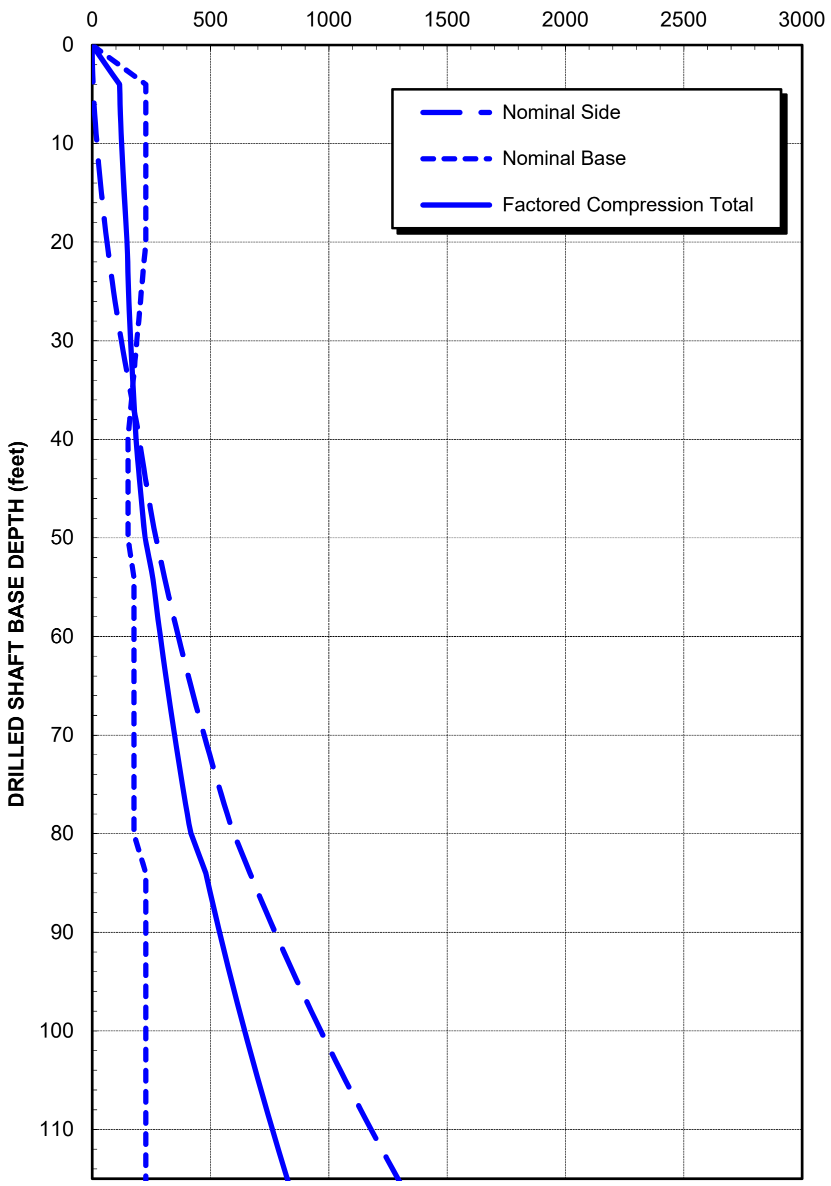


SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT

NOMINAL RESISTANCE (tons)

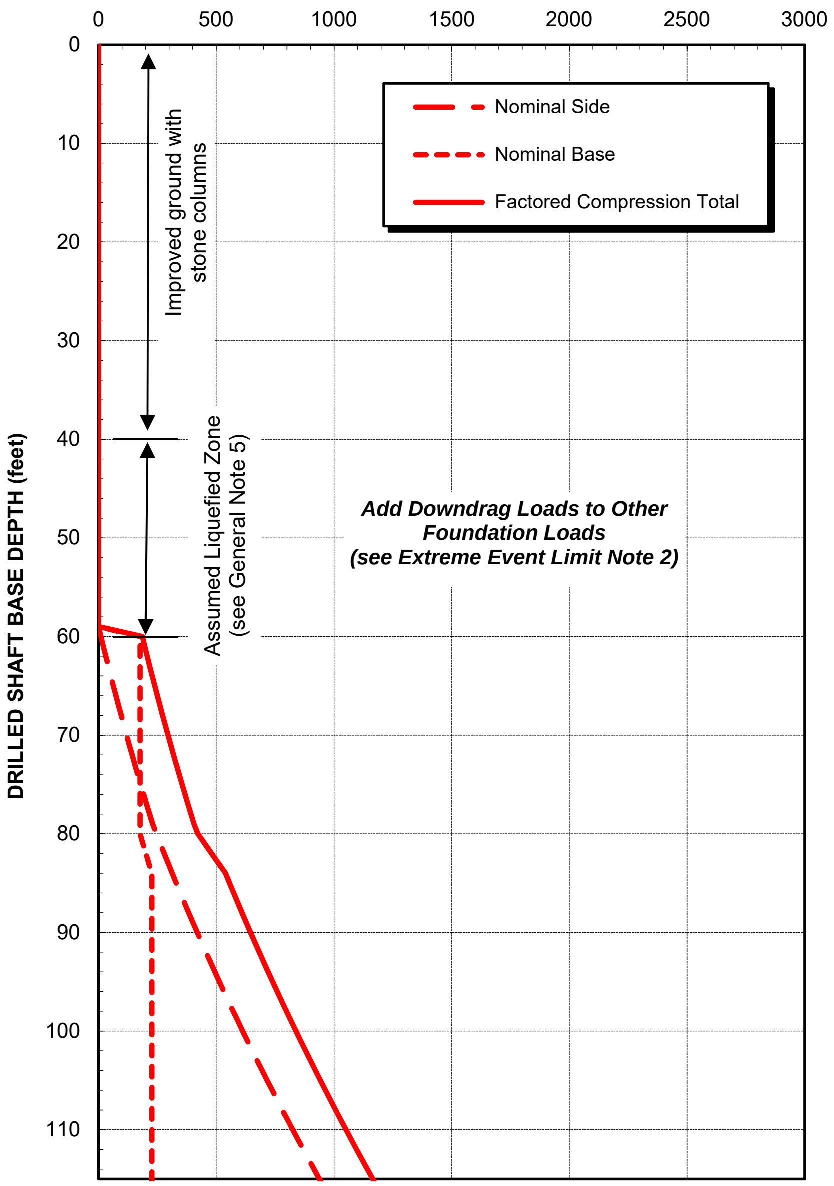


STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 270 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

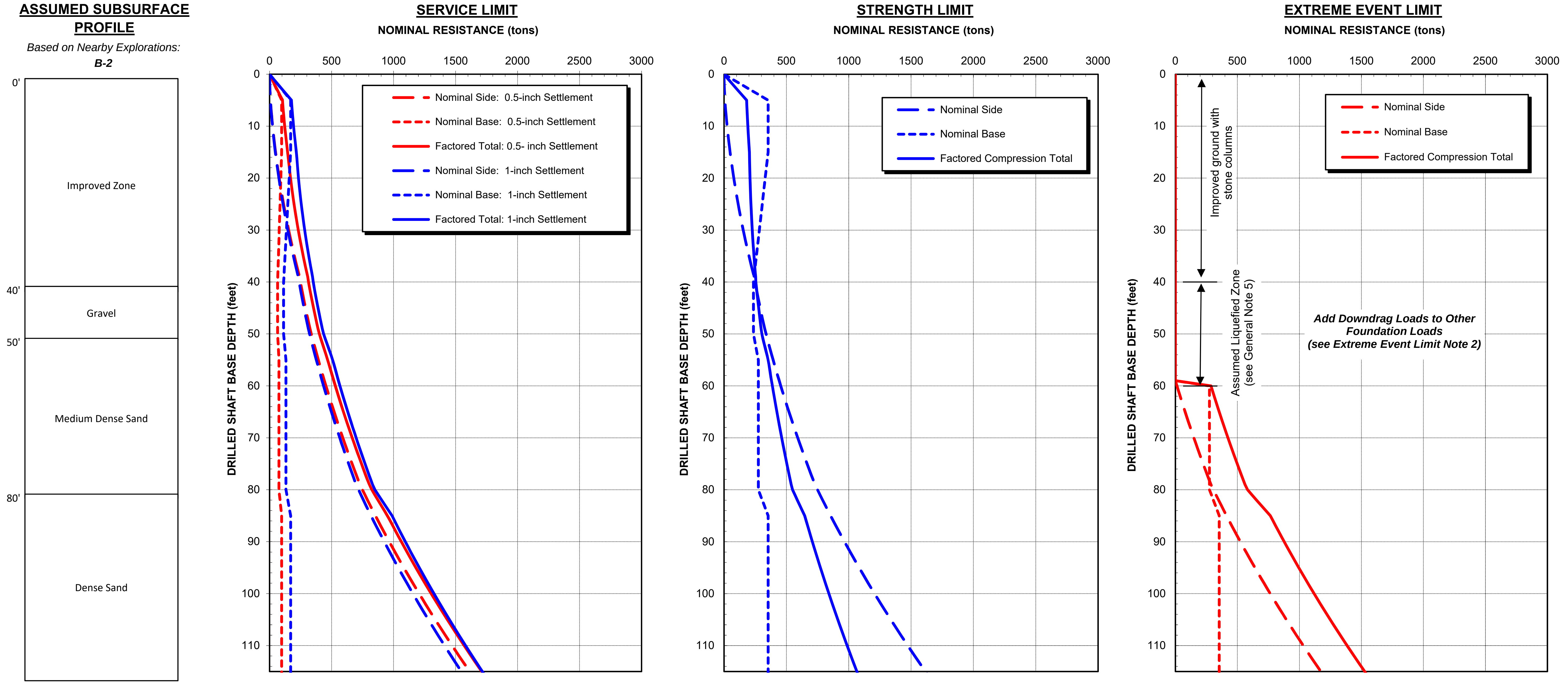
S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
4-foot Diameter Drilled Shaft
Pier 1

June 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E3



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 340 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
5-foot Diameter Drilled Shaft
Pier 1

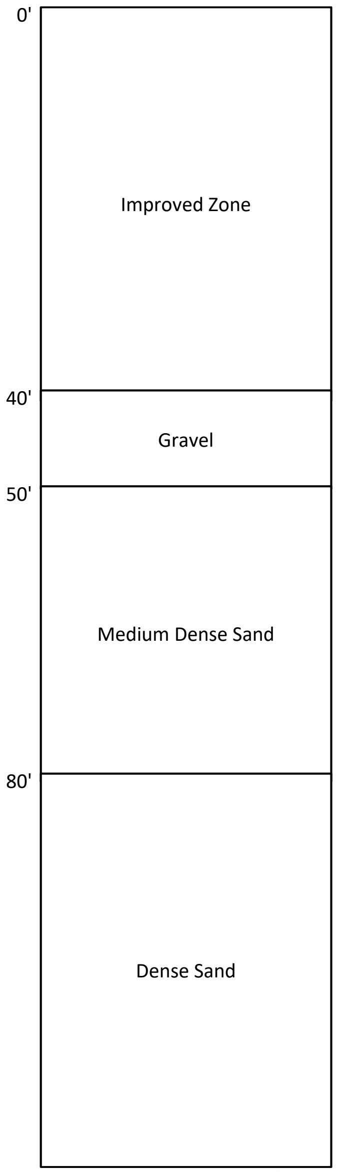
June 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E4

ASSUMED SUBSURFACE PROFILE

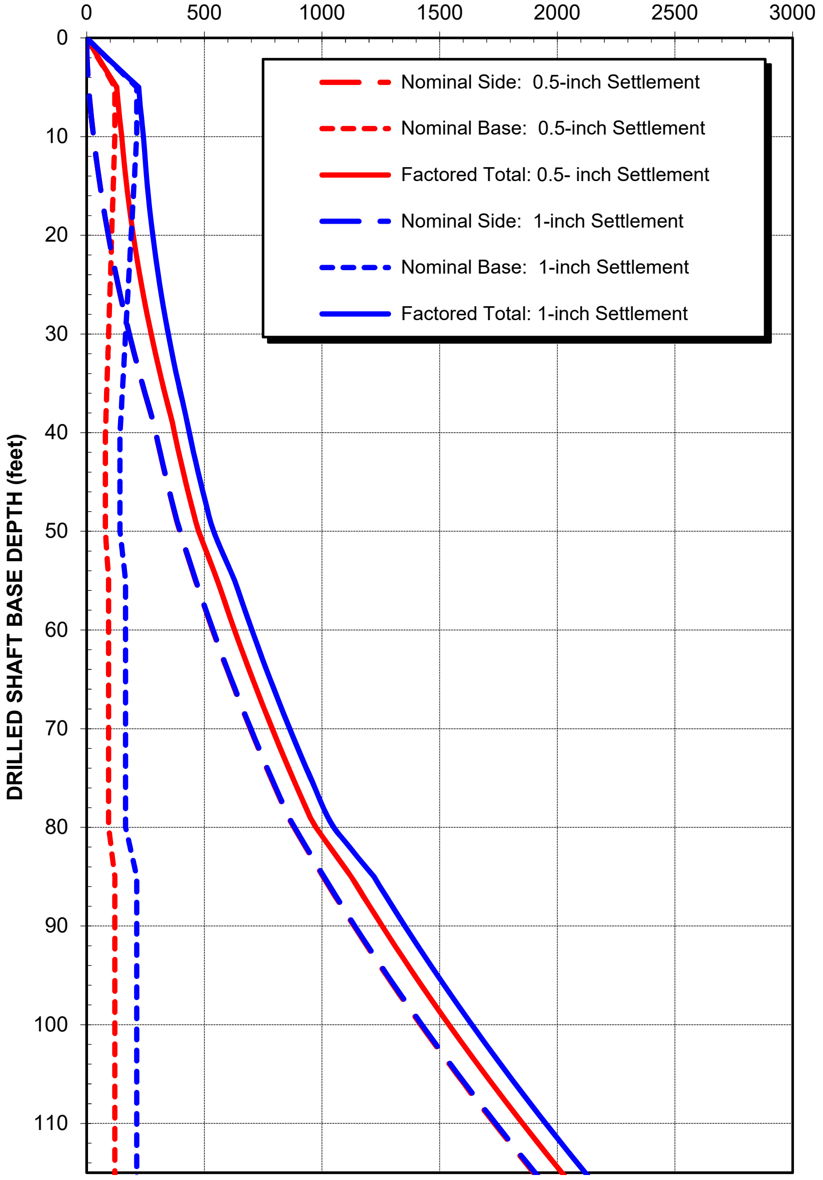
Based on Nearby Explorations:
B-2



Boring Extends to 140.0 feet

SERVICE LIMIT

NOMINAL RESISTANCE (tons)

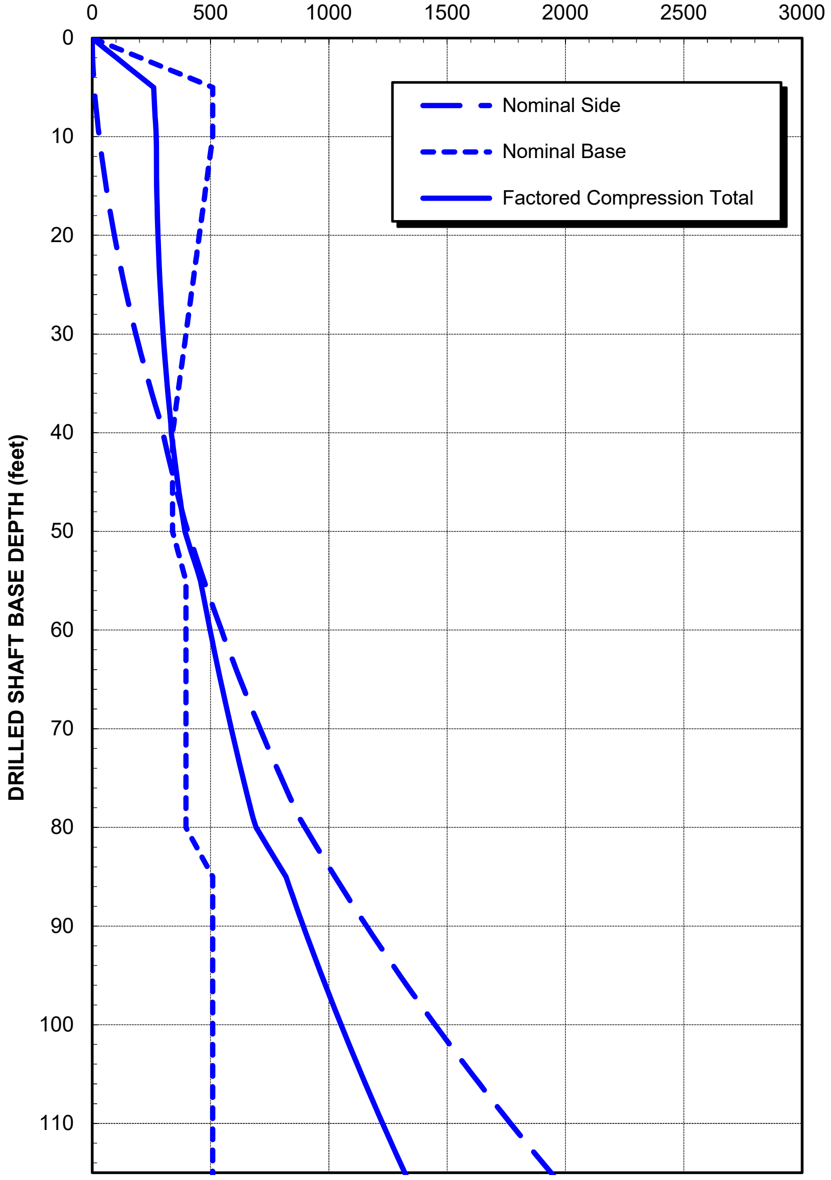


SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT

NOMINAL RESISTANCE (tons)

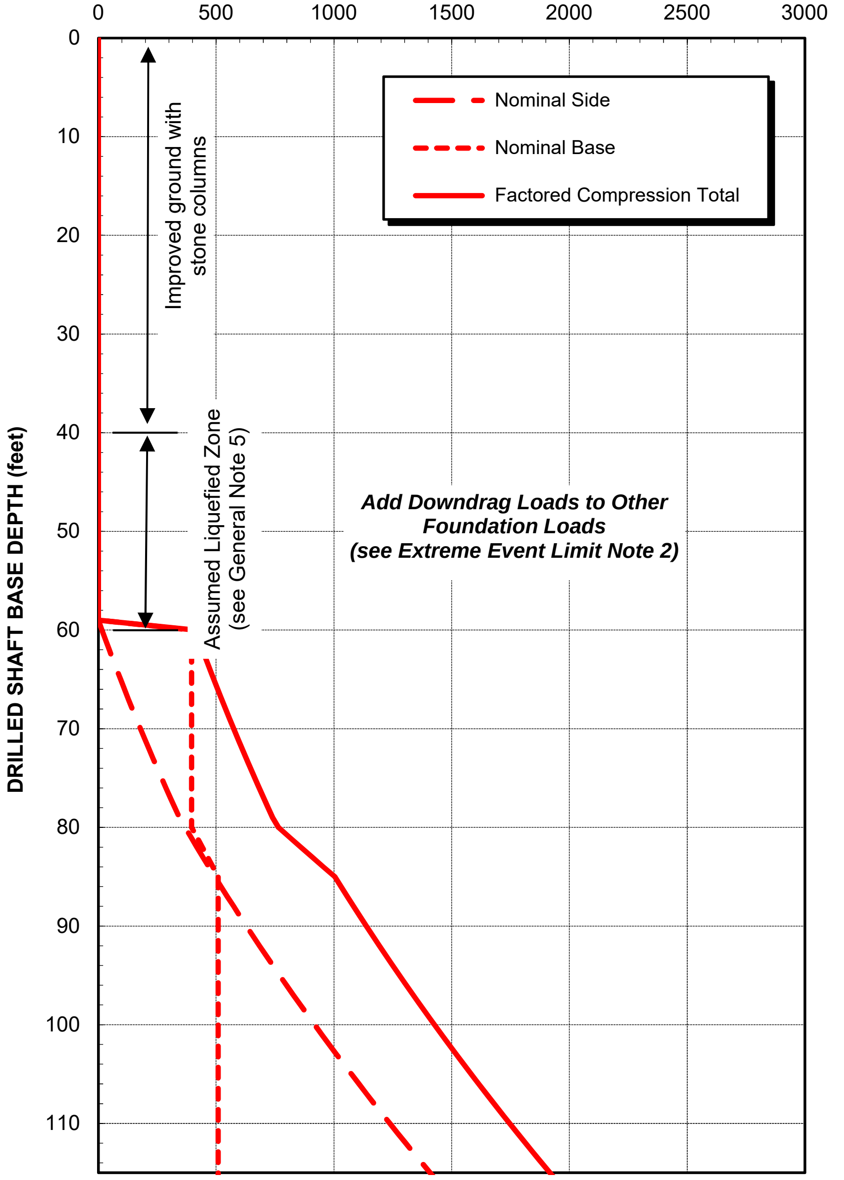


STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 410 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
6-foot Diameter Drilled Shaft
Pier 1

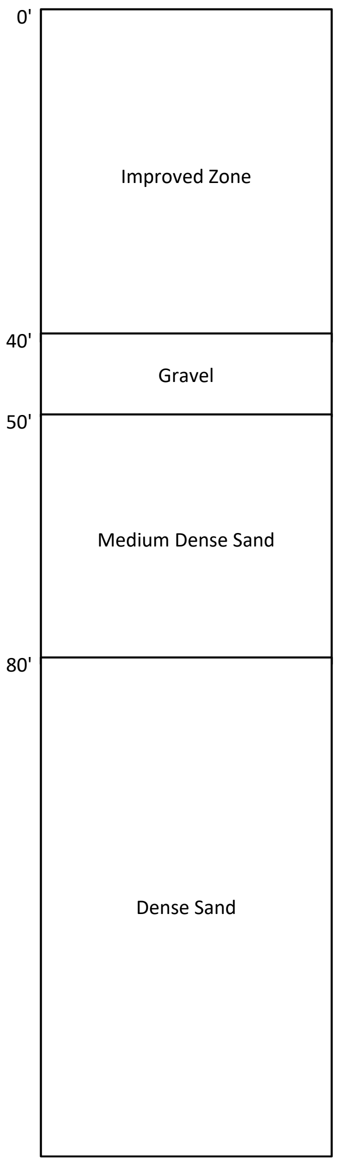
June 2018
24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

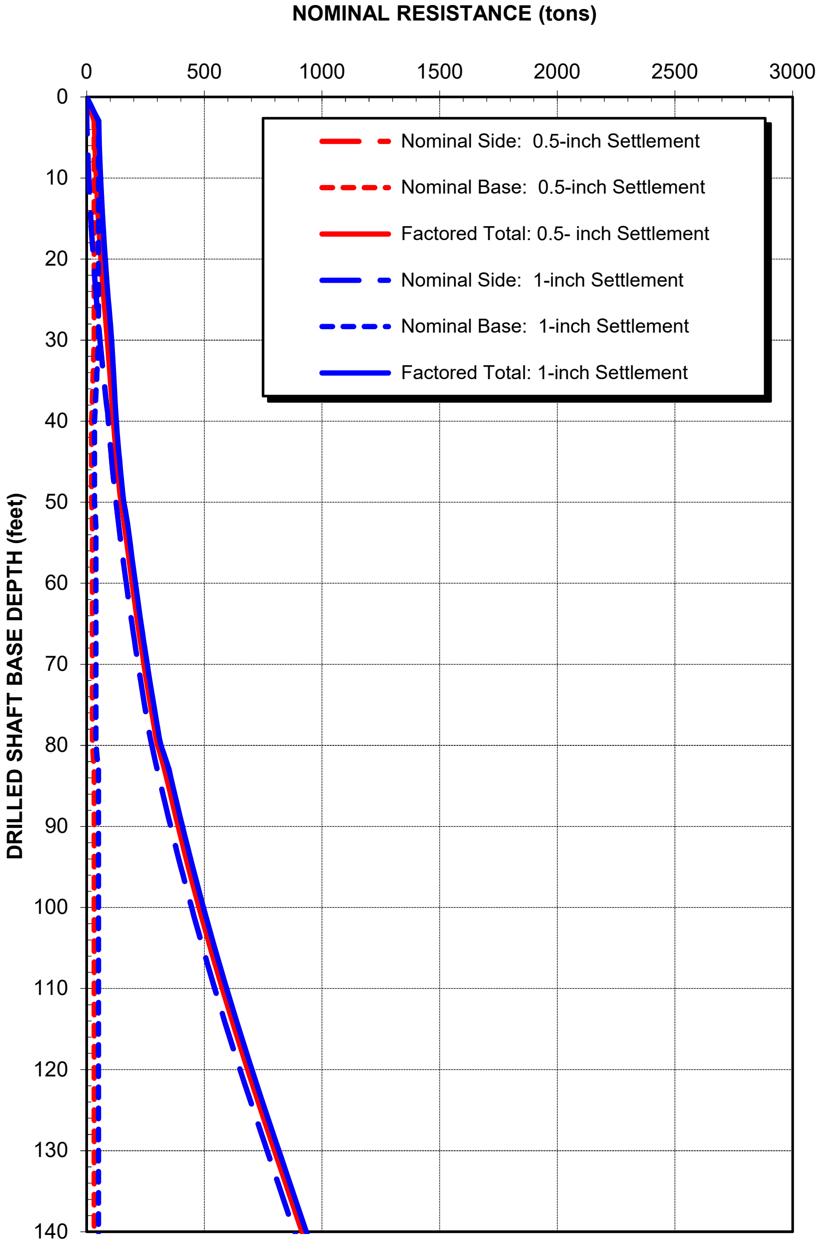
FIG. E5

ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-1



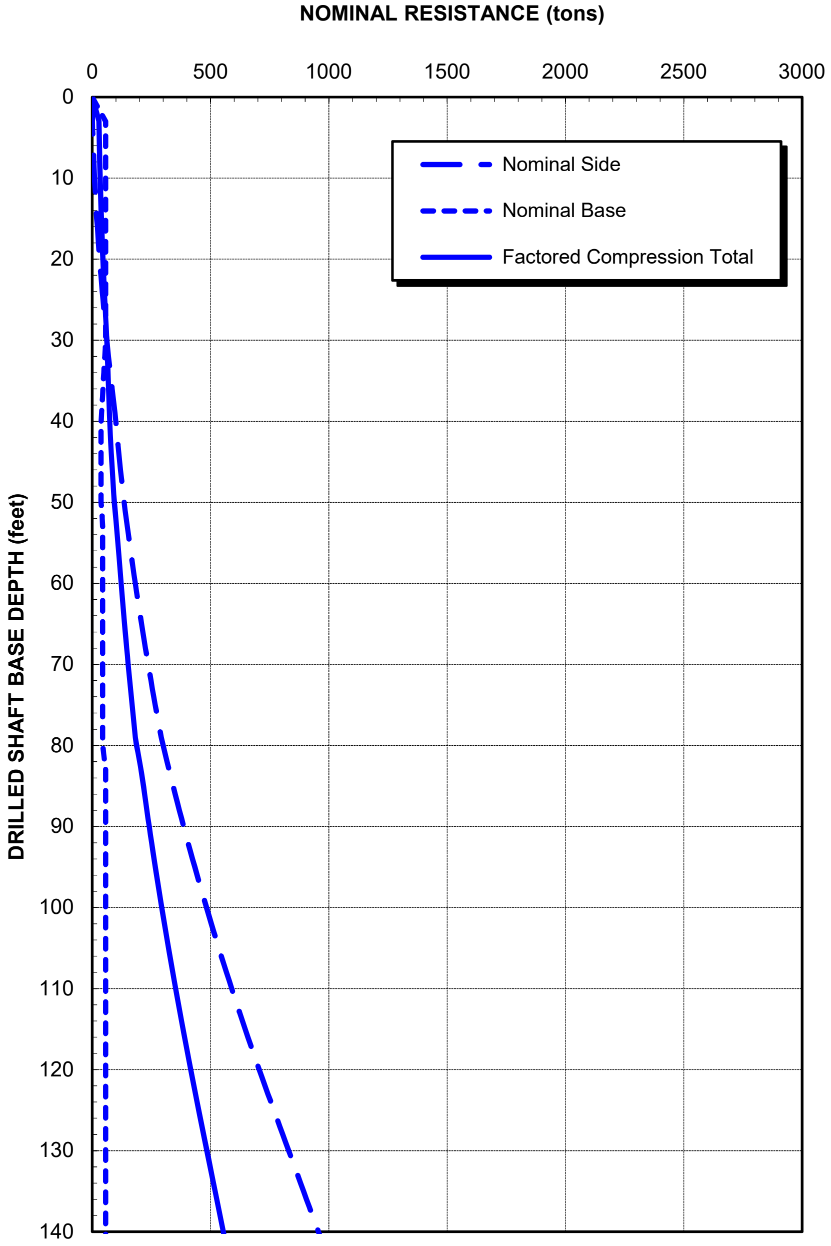
SERVICE LIMIT



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

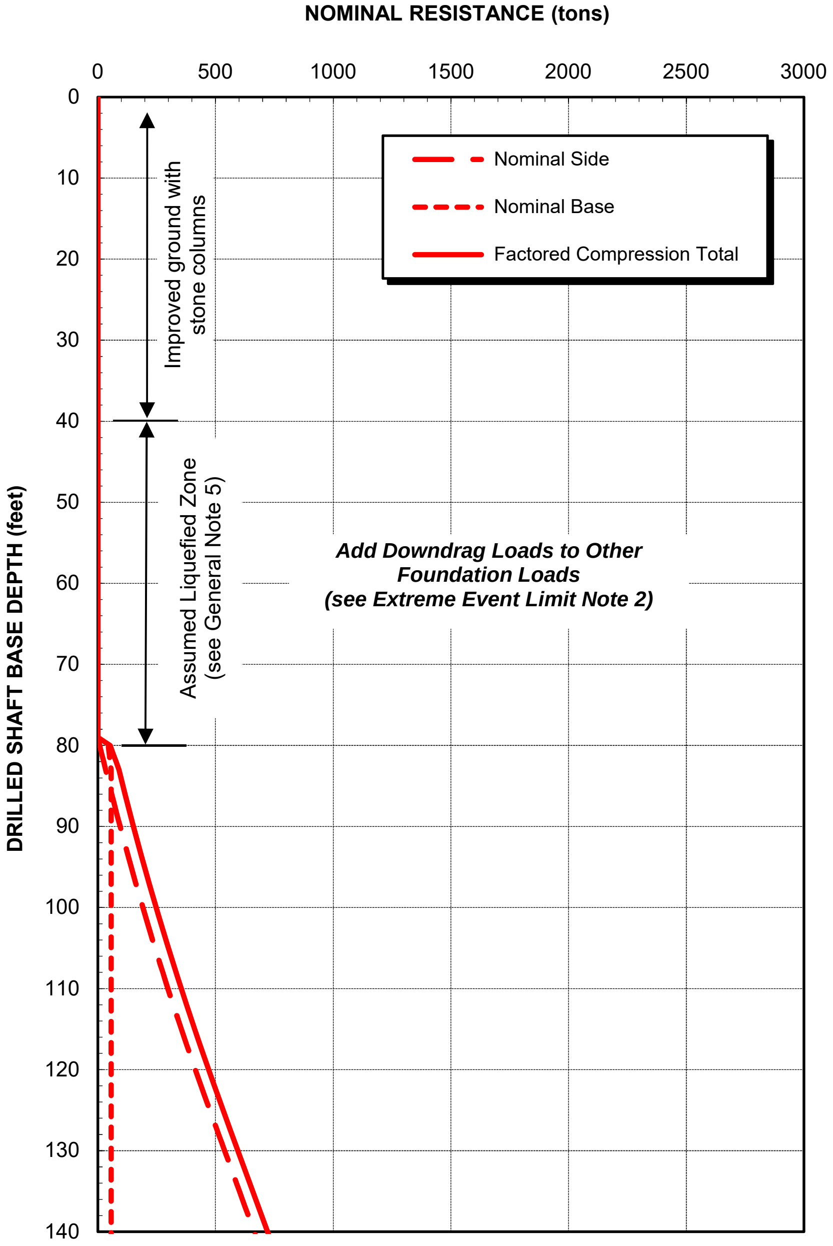
STRENGTH LIMIT



STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 190 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
2-foot Diameter Drilled Shaft
Pier 4

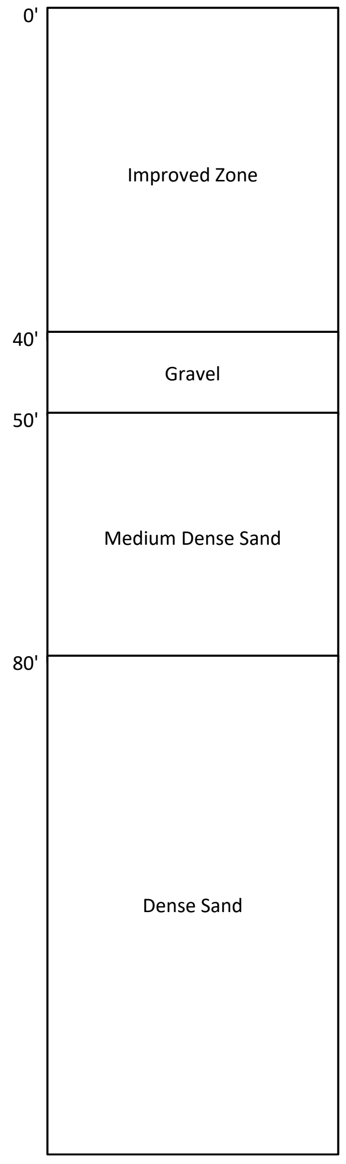
June 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

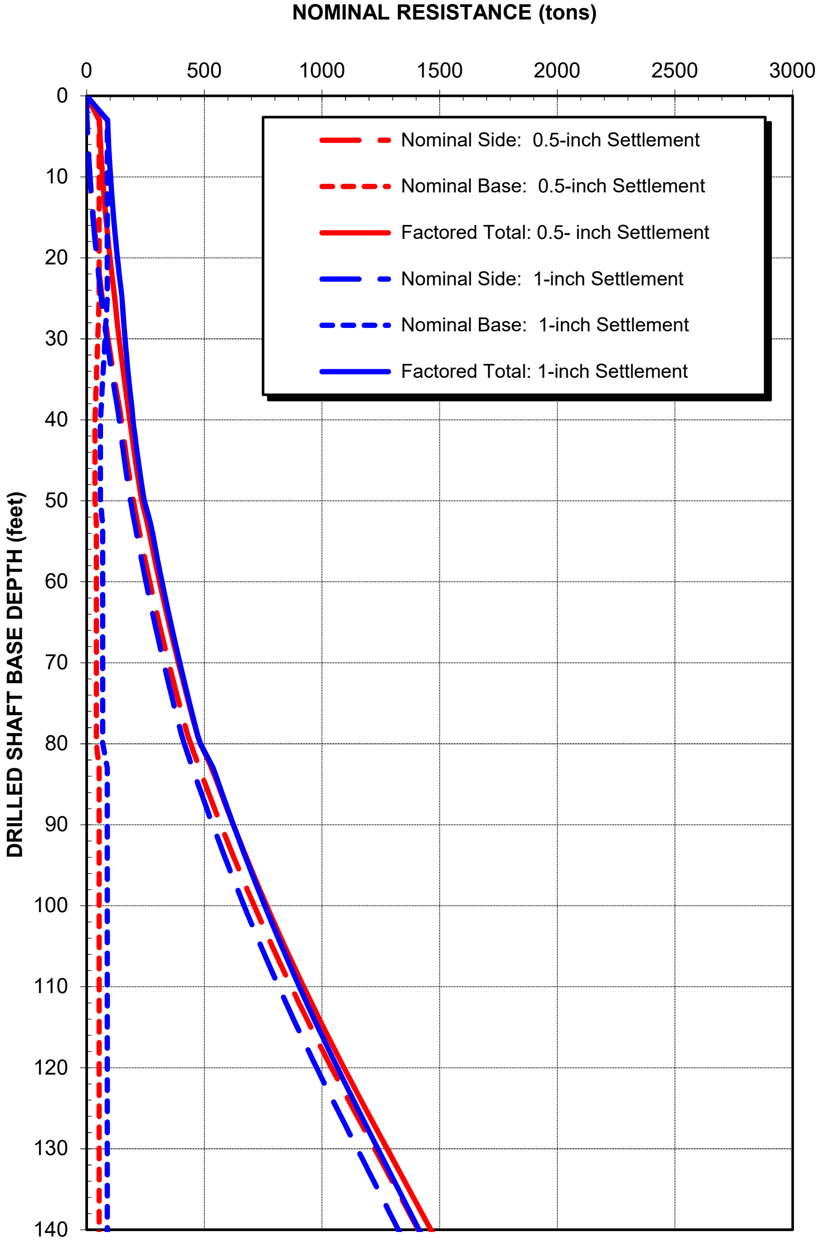
FIG. E6

ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-1



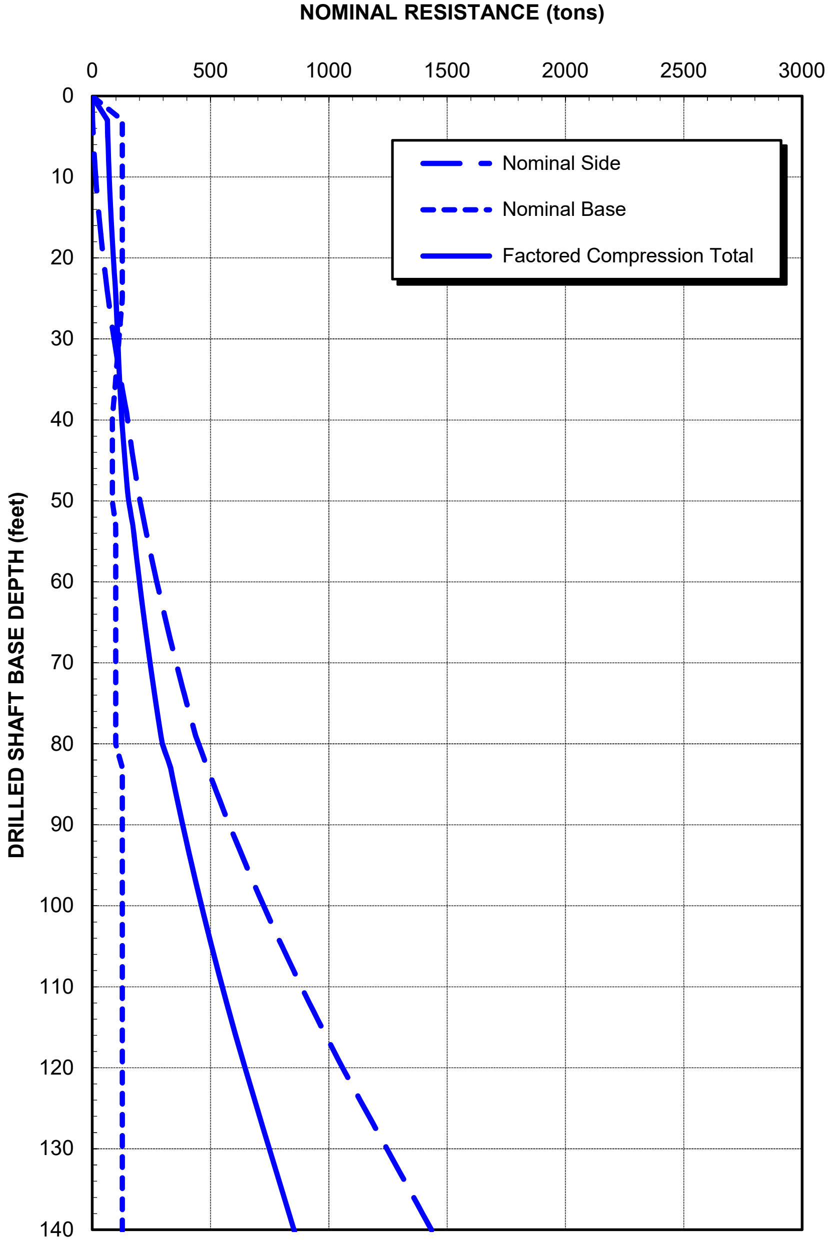
SERVICE LIMIT



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

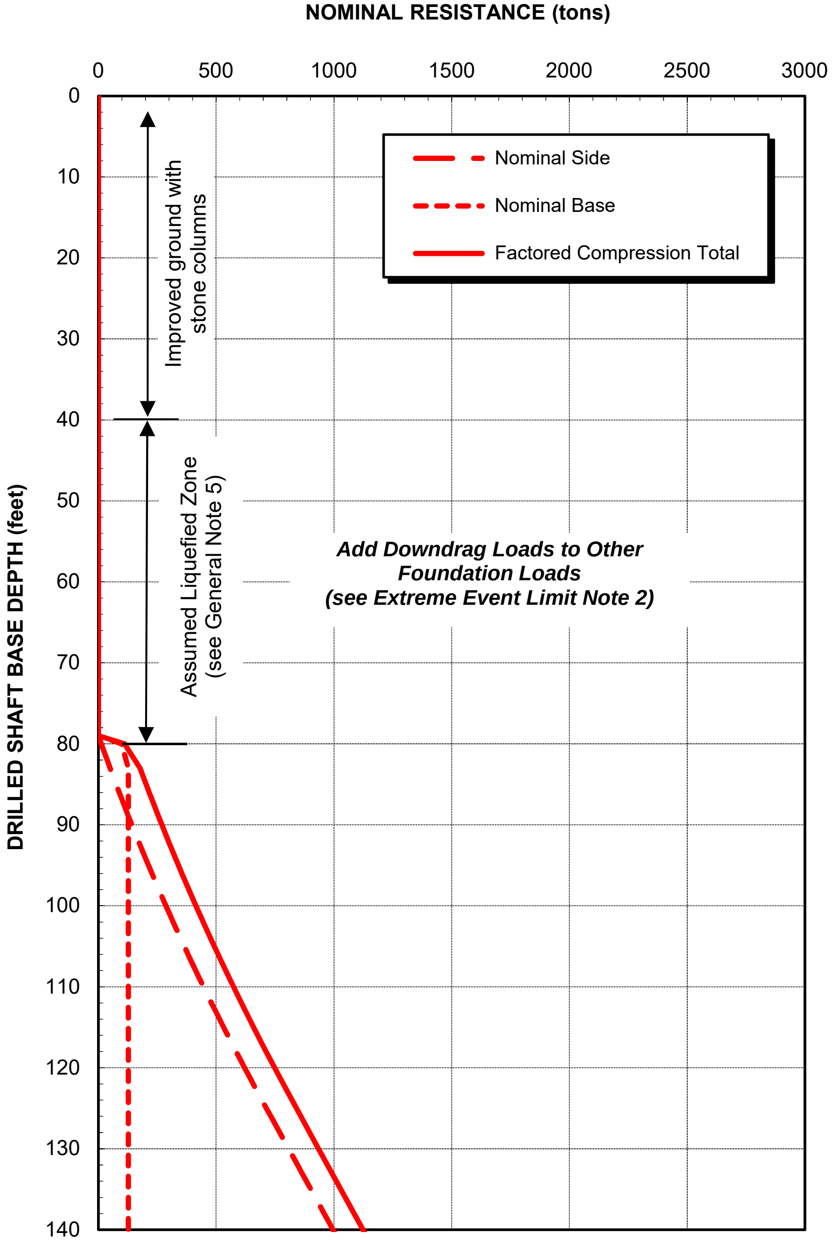
STRENGTH LIMIT



STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 290 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
3-foot Diameter Drilled Shaft
Pier 2

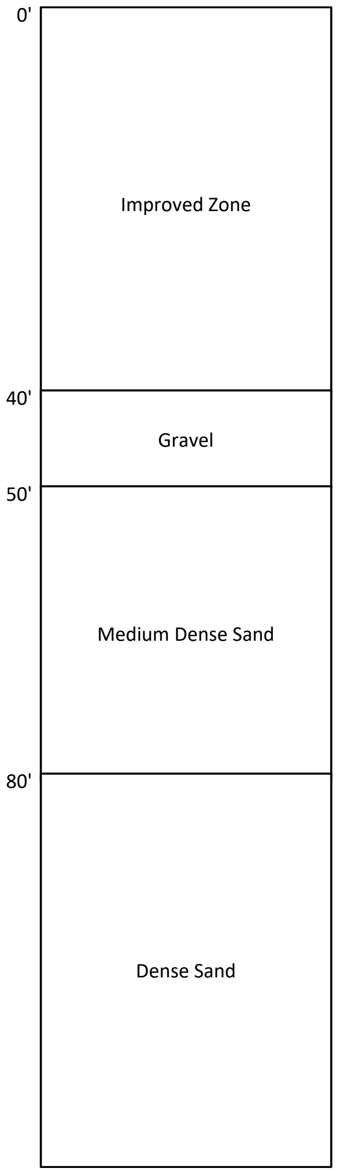
June 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E7

ASSUMED SUBSURFACE PROFILE

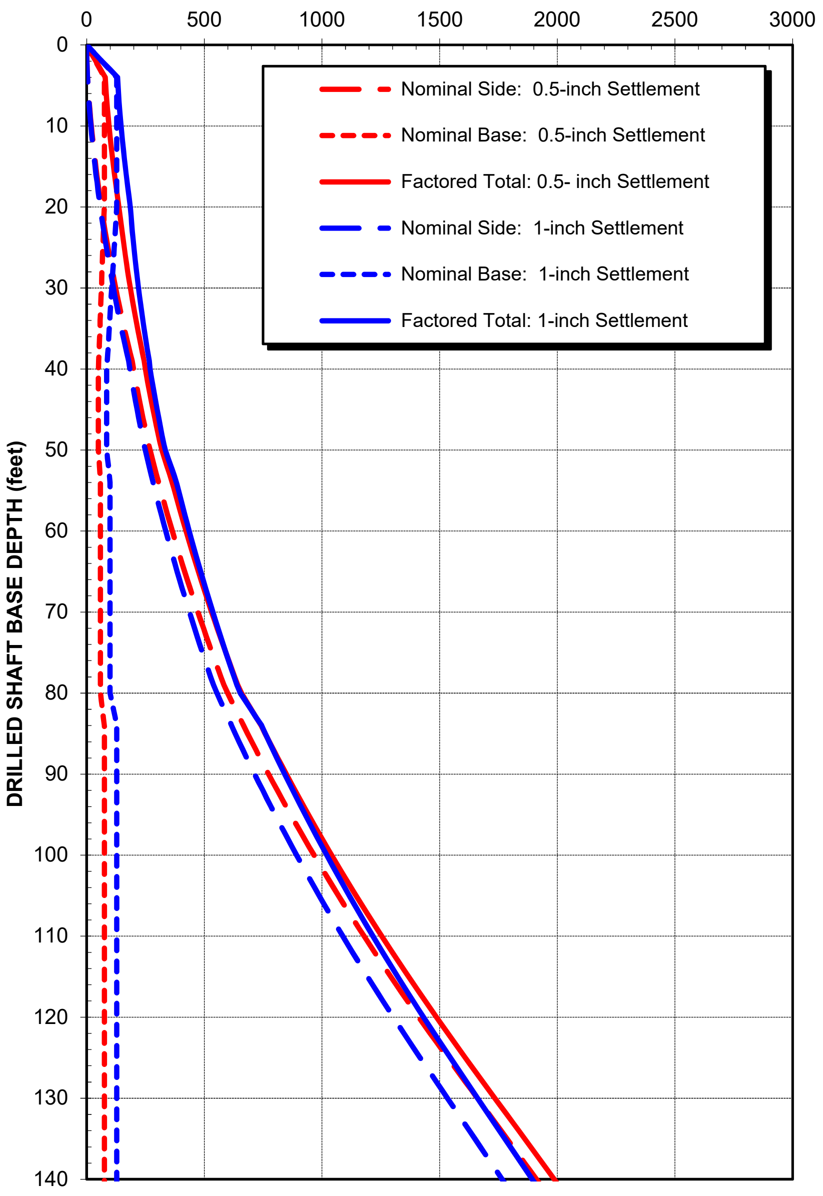
Based on Nearby Explorations:
B-1



Boring Extends to 140.0 feet

SERVICE LIMIT

NOMINAL RESISTANCE (tons)

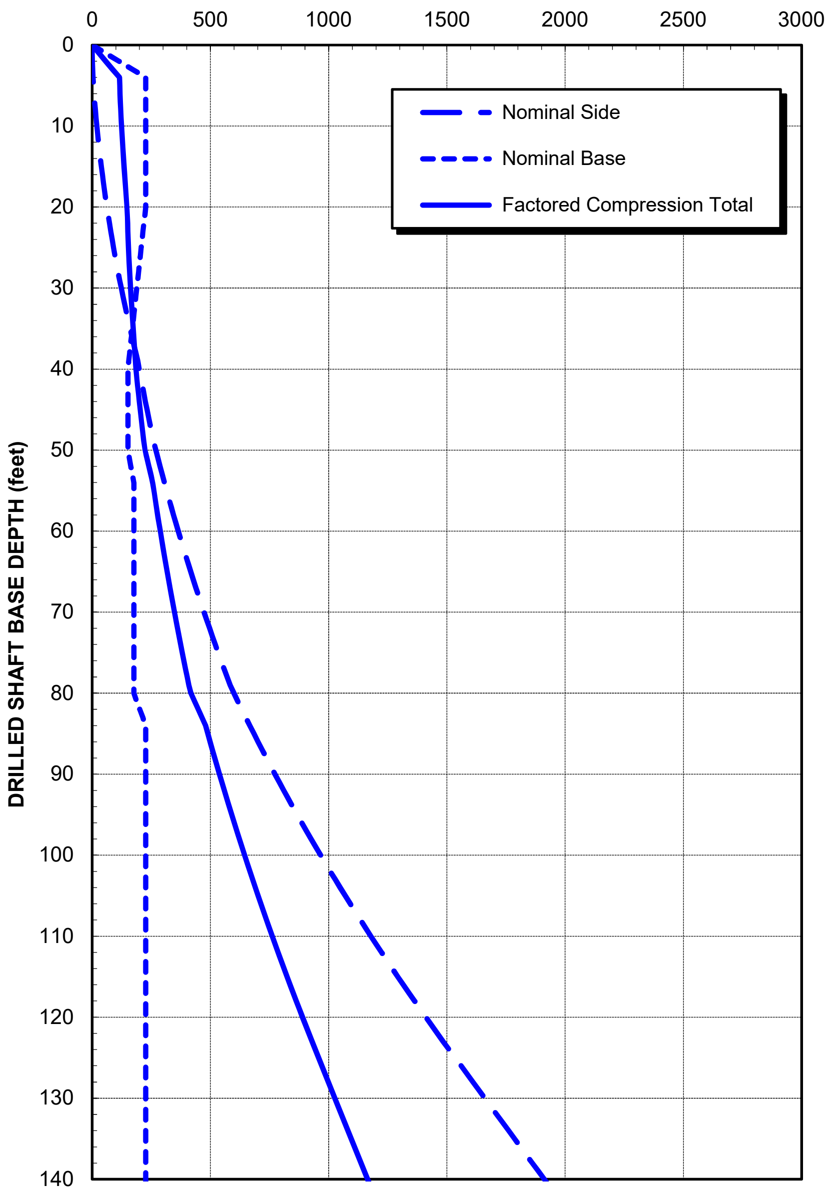


SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT

NOMINAL RESISTANCE (tons)

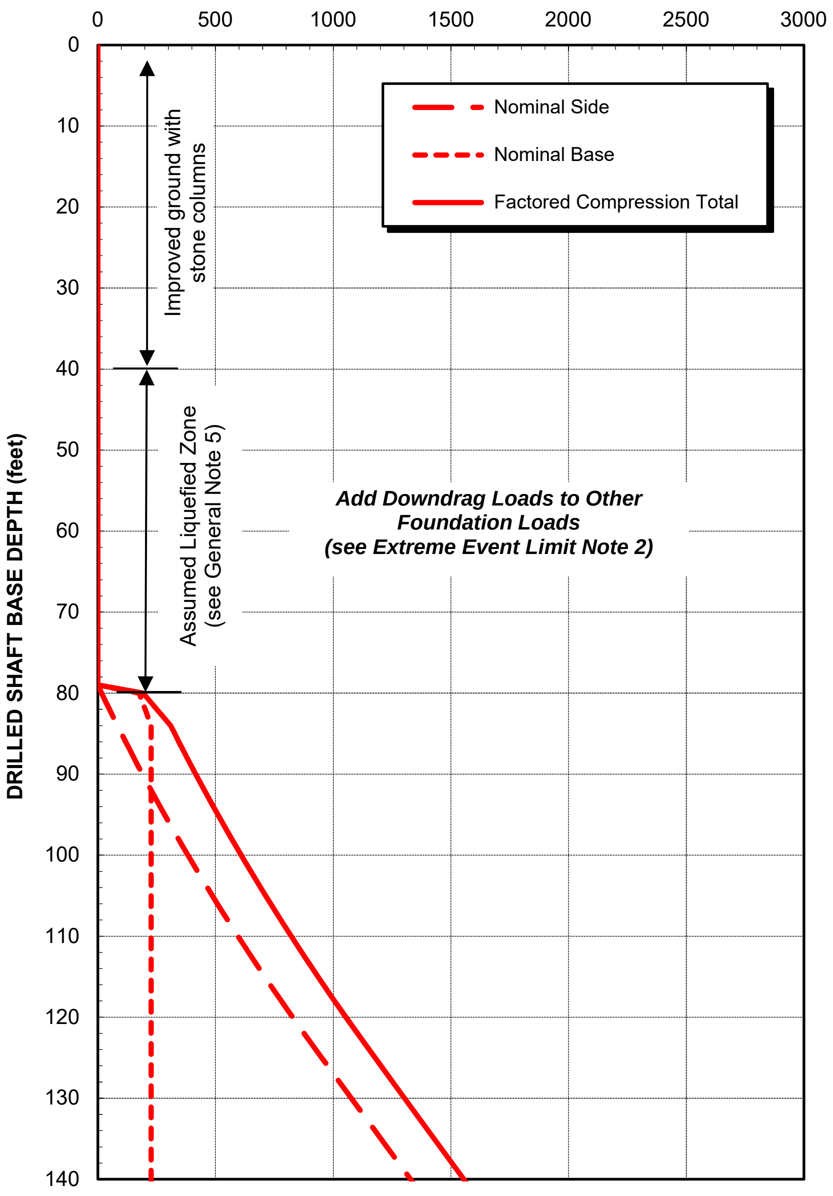


STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 390 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
4-foot Diameter Drilled Shaft
Pier 4

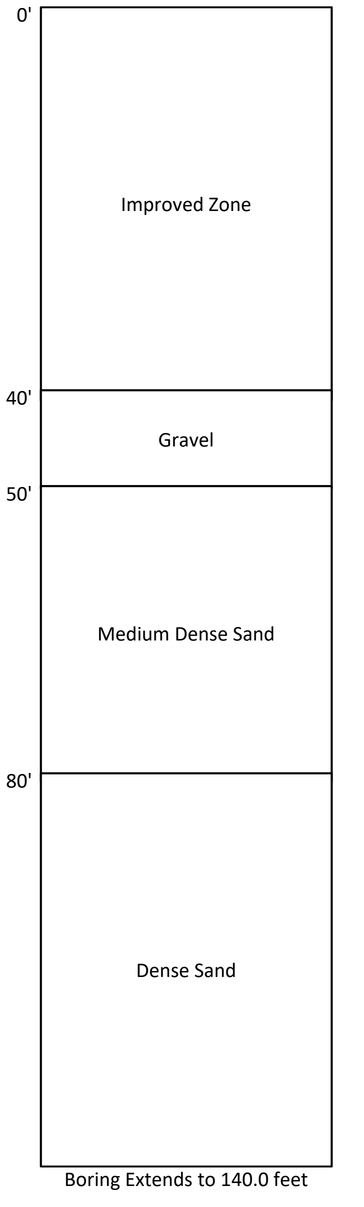
June 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E8

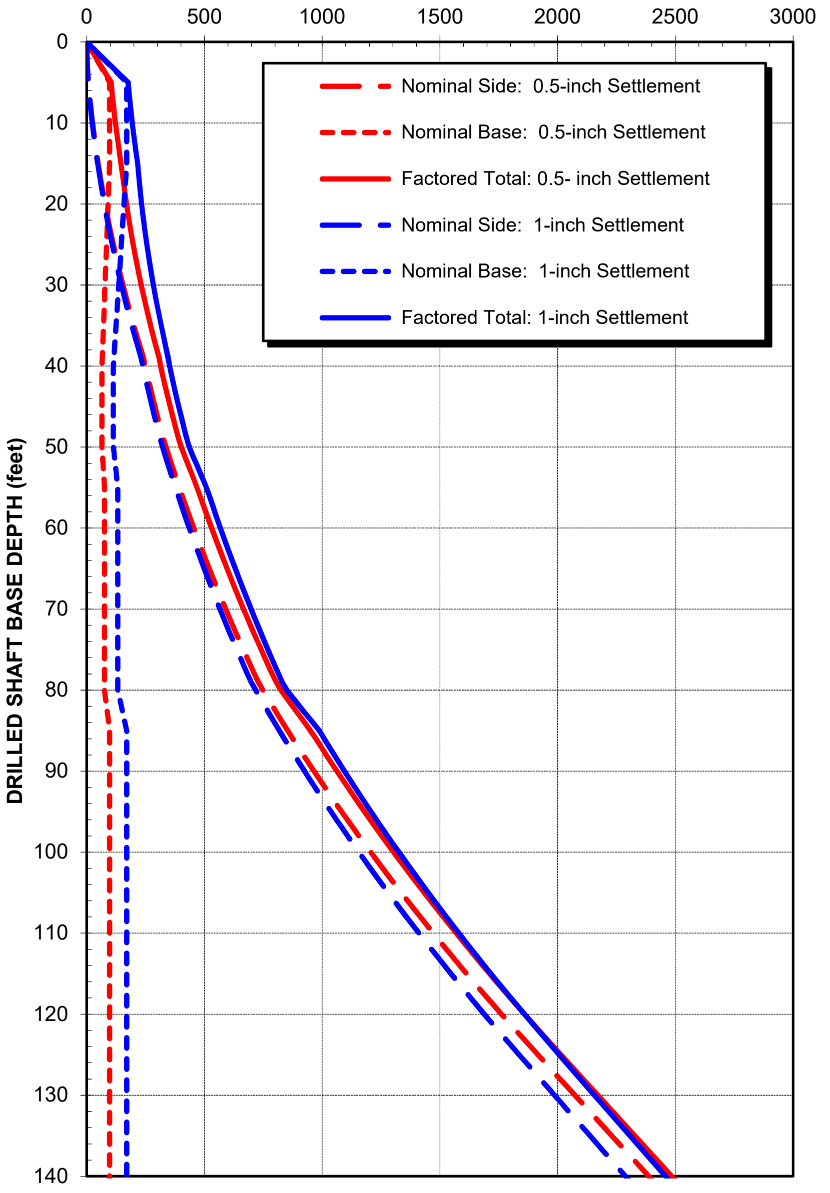
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-1



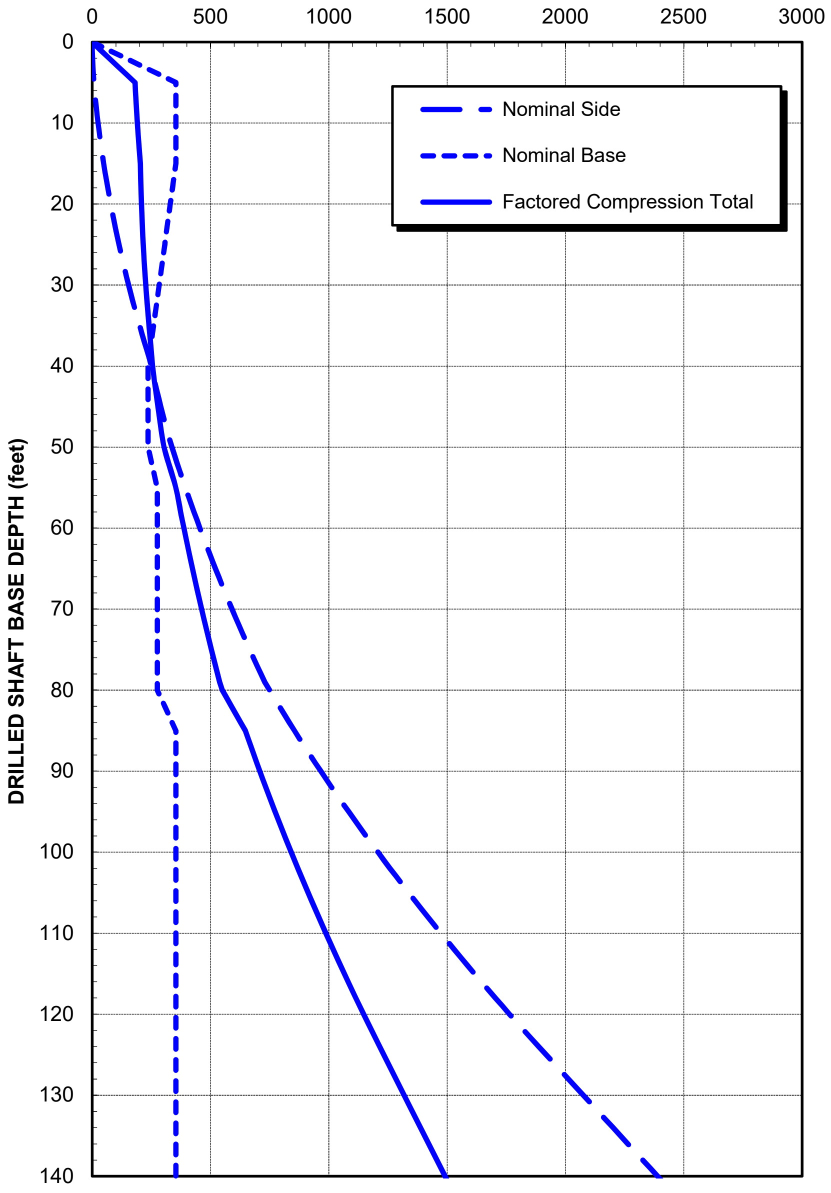
SERVICE LIMIT

NOMINAL RESISTANCE (tons)



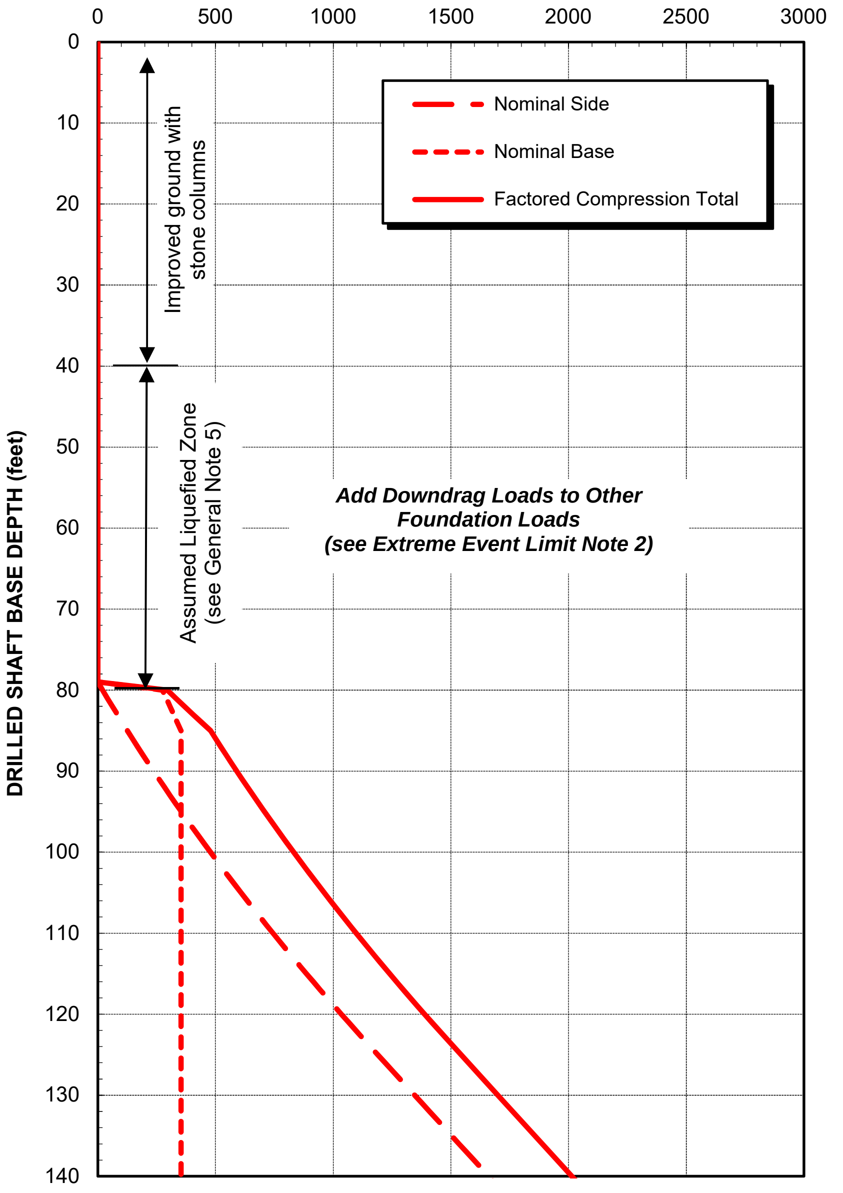
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 490 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
5-foot Diameter Drilled Shaft
Pier 4

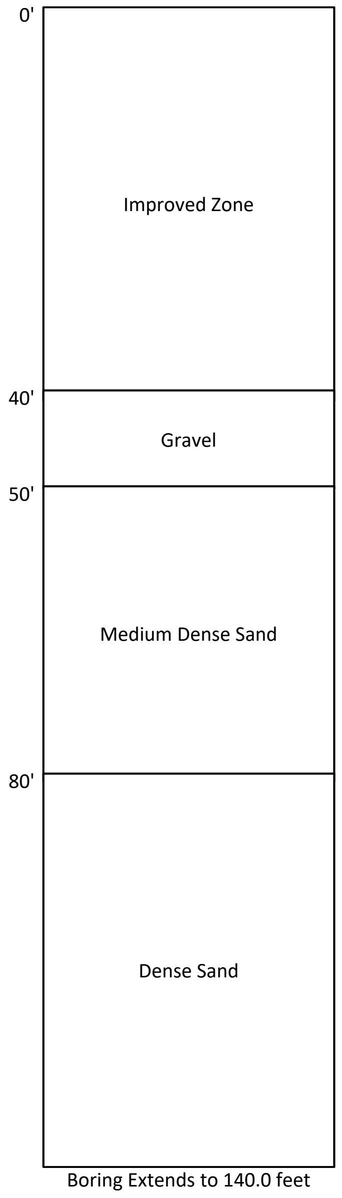
June 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

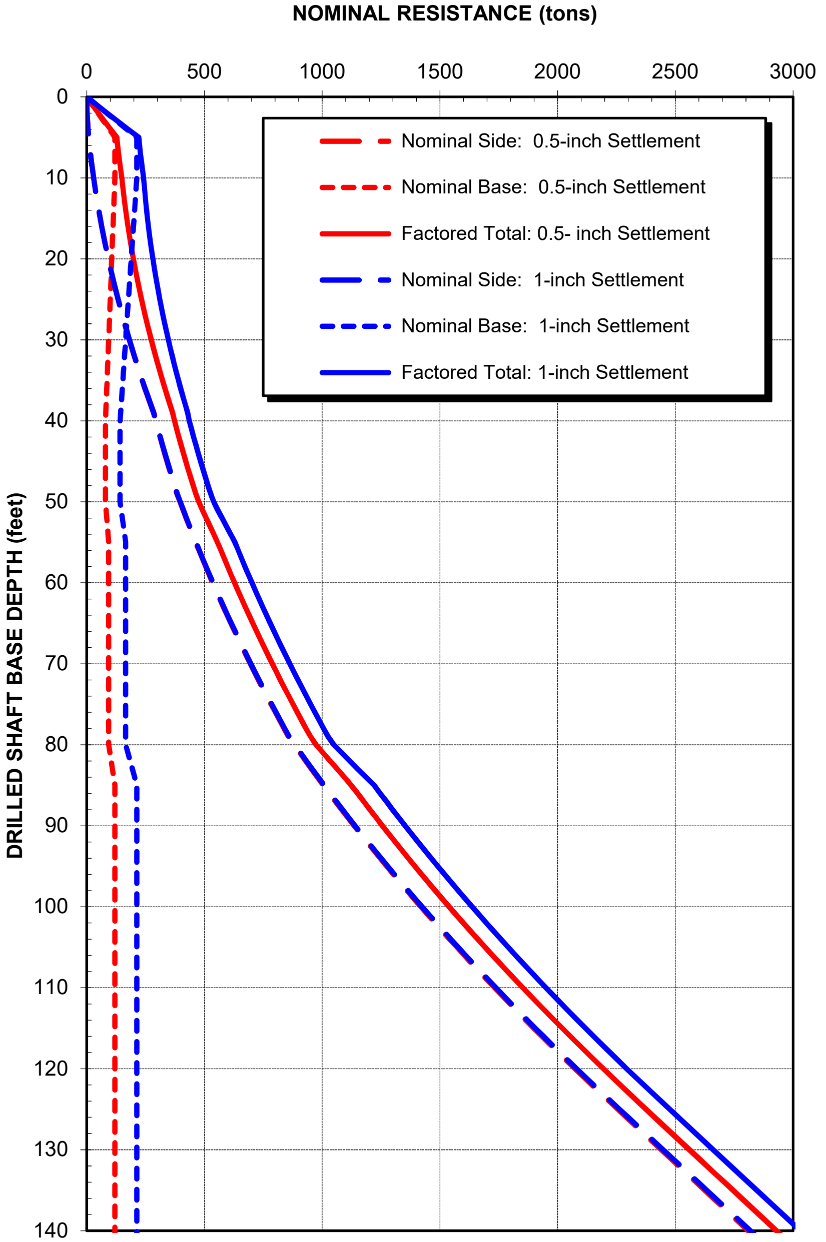
FIG. E9

ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-1



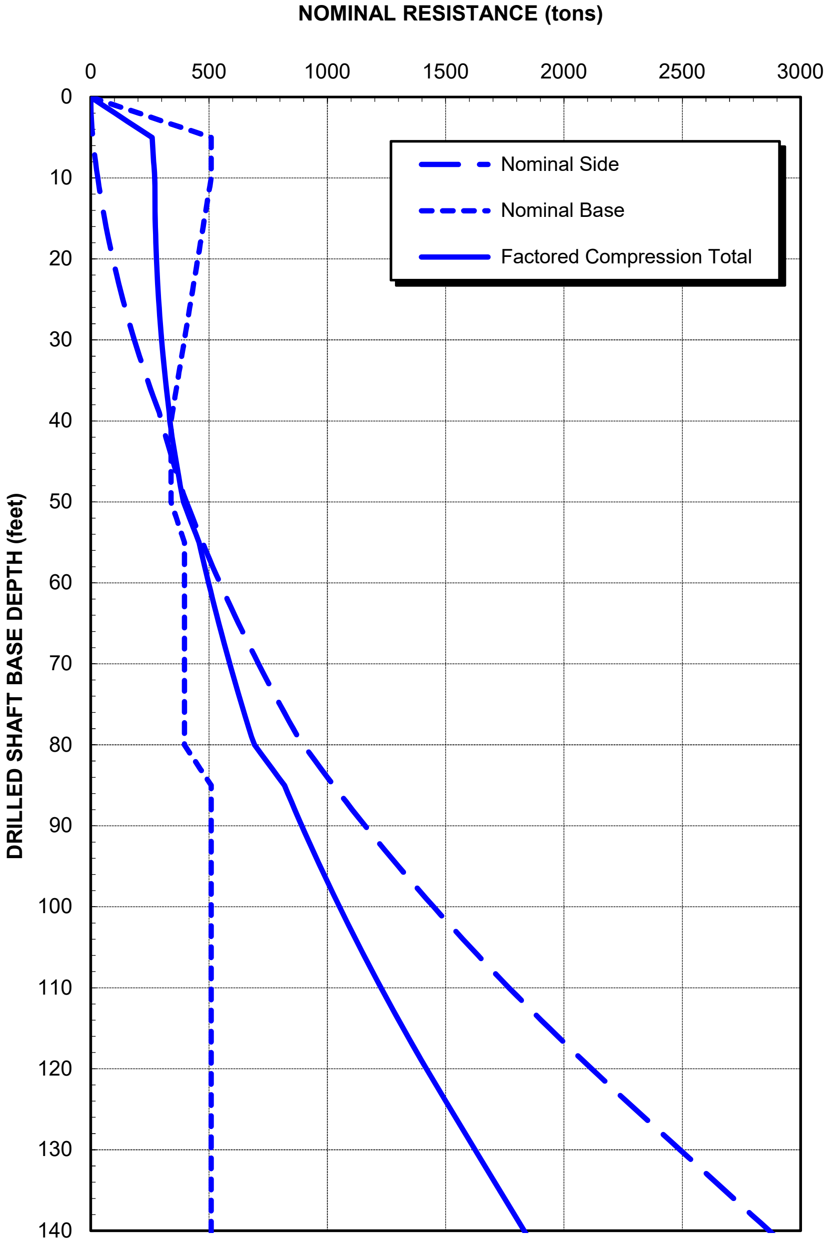
SERVICE LIMIT



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

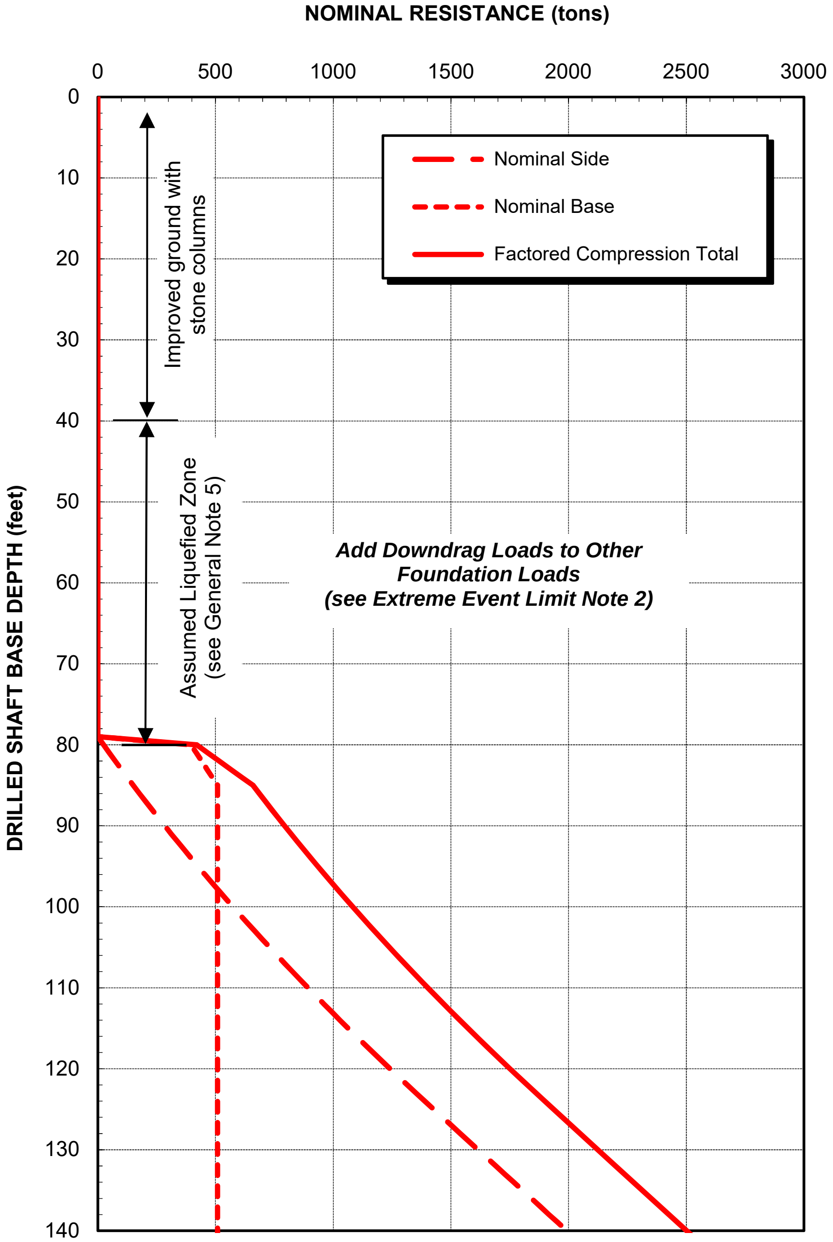
STRENGTH LIMIT



STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 580 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation
S. Kelso, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
6-foot Diameter Drilled Shaft
Pier 4

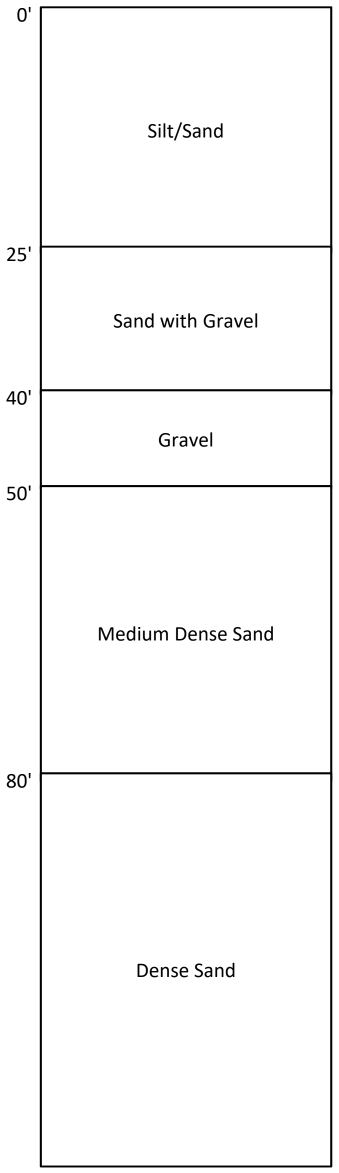
June 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

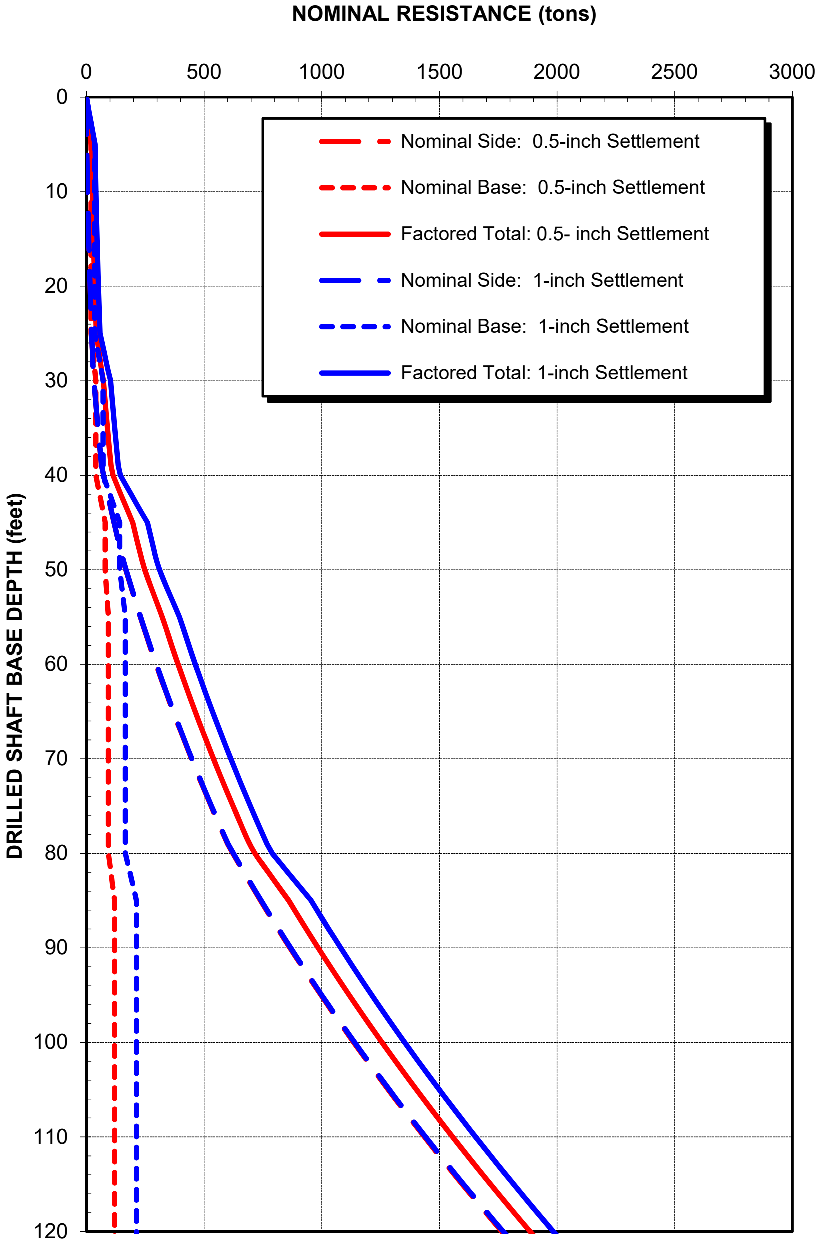
FIG. E10

ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-1



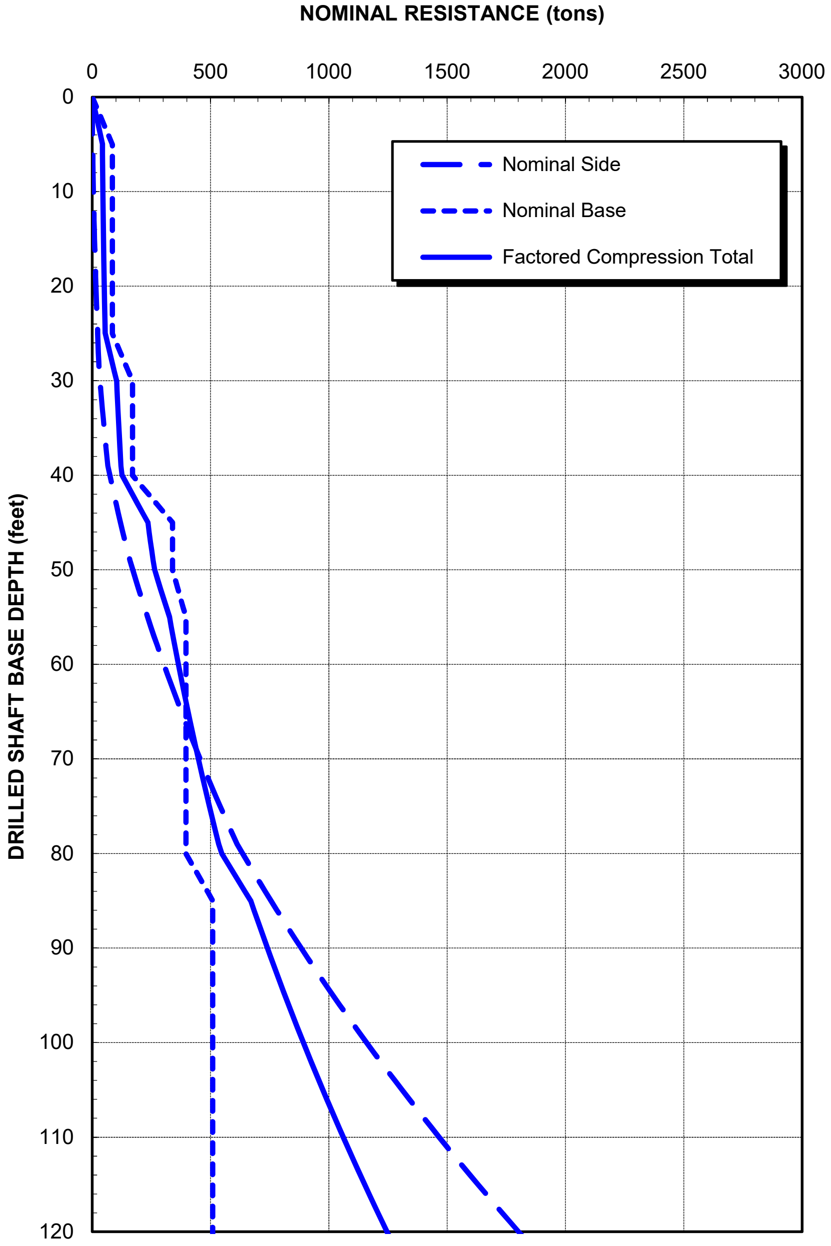
SERVICE LIMIT



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

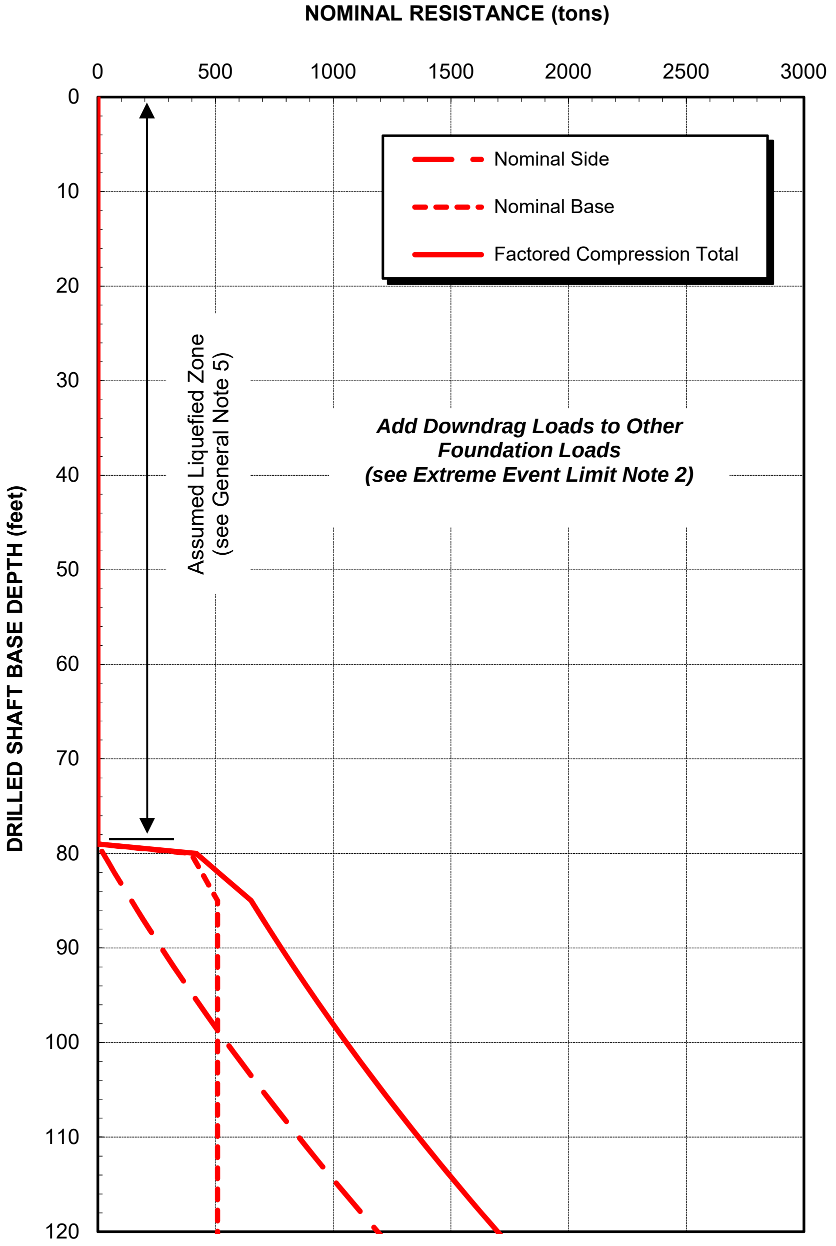
STRENGTH LIMIT



STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 310 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation

ESTIMATED AXIAL SHAFT RESISTANCE
6-foot Diameter Drilled Shaft
Piers 2 and 3

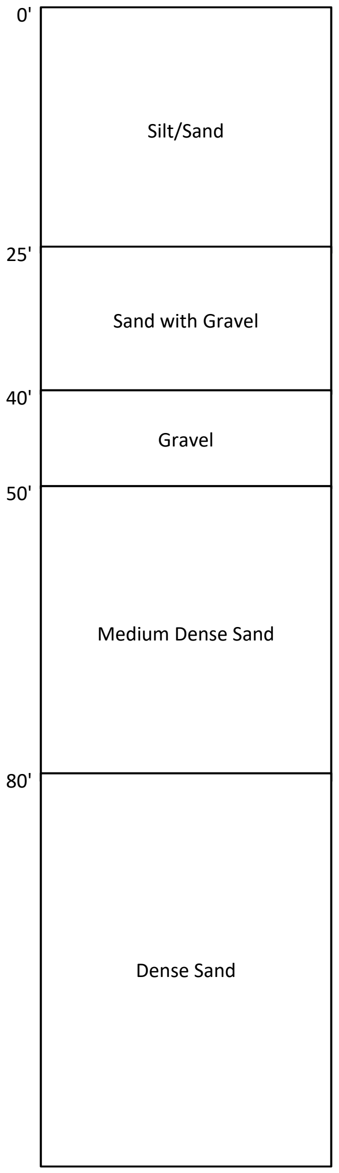
August 2018 24-1-04201-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

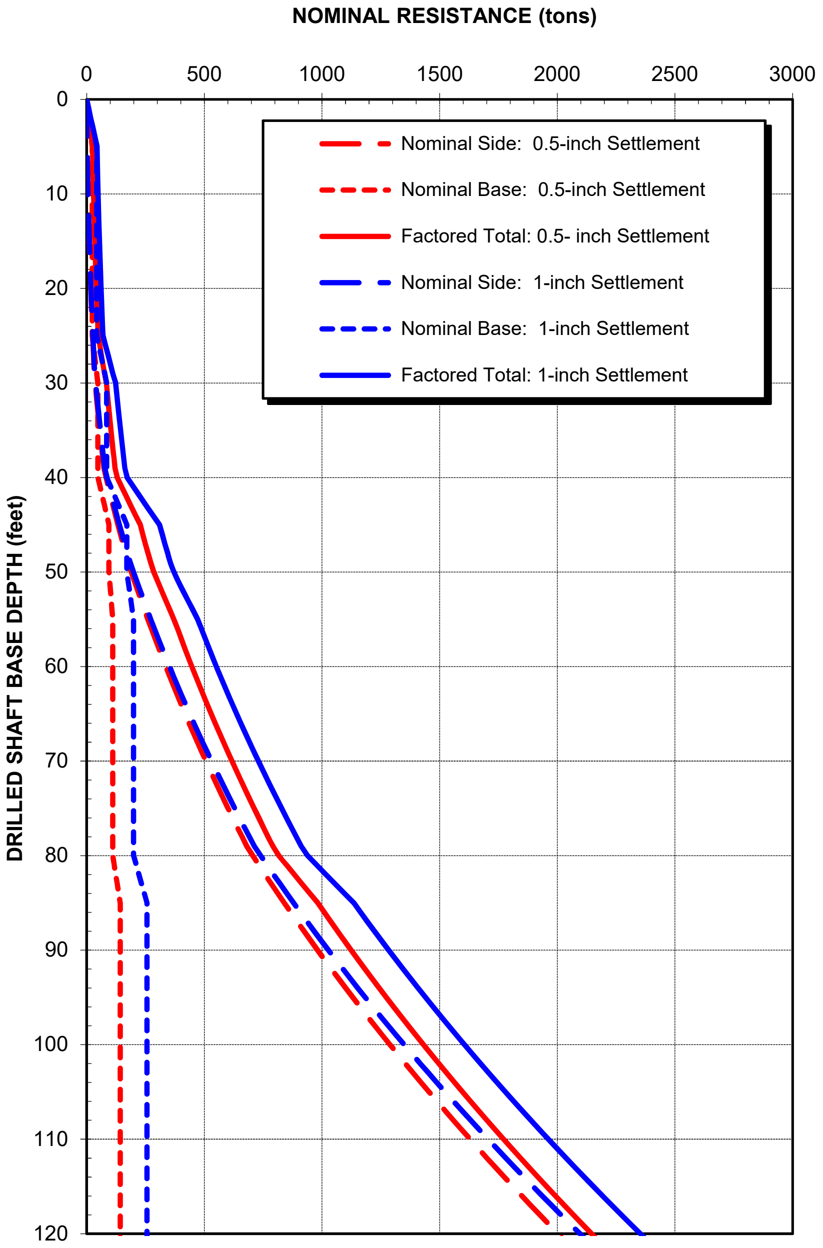
FIG. E11

ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-1



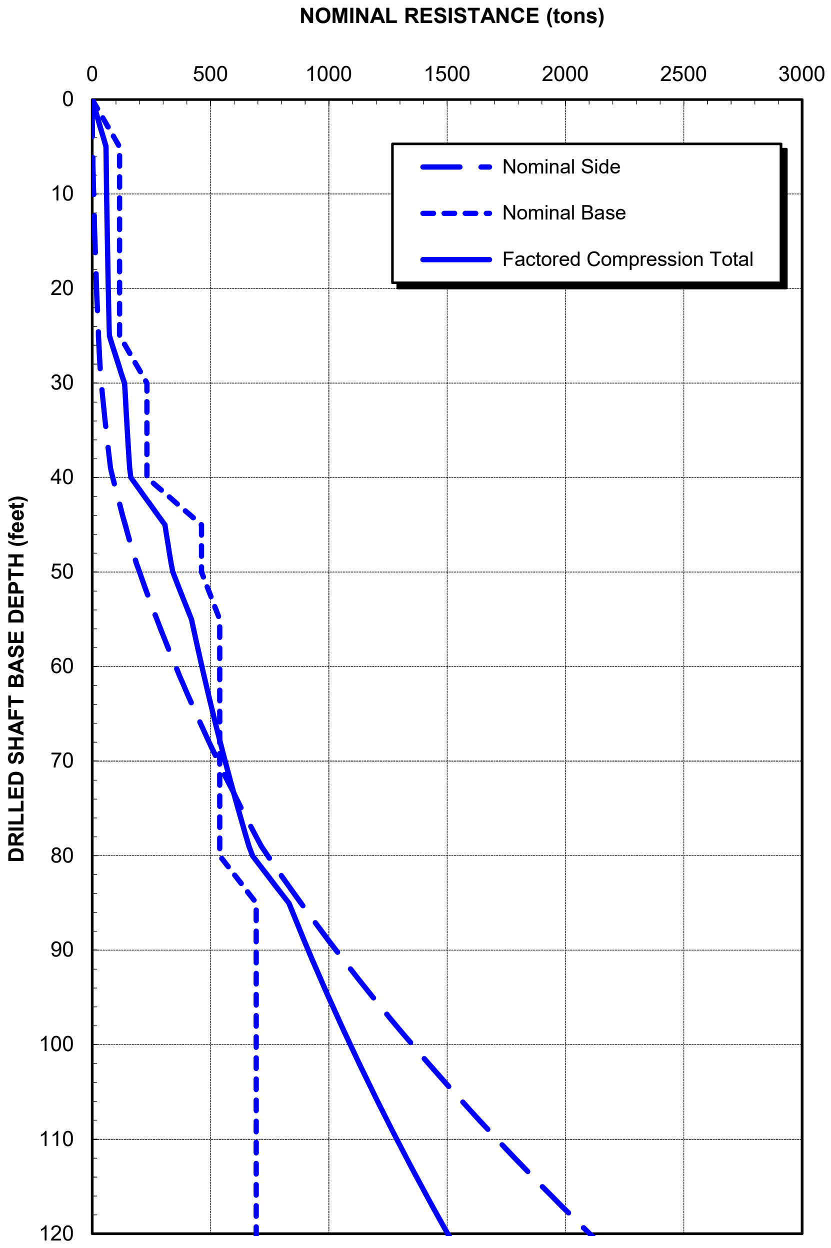
SERVICE LIMIT



SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

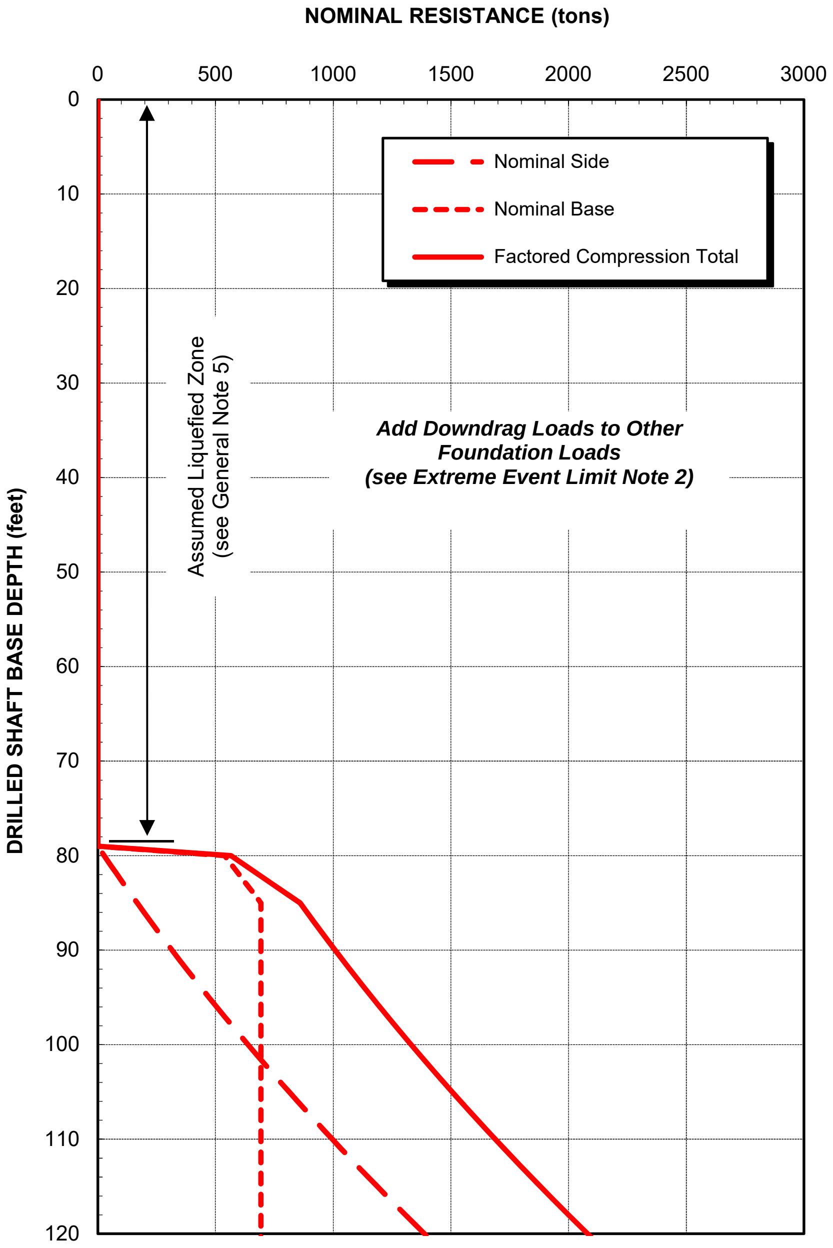
STRENGTH LIMIT



STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 360 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
4. Estimated shaft resistance assumes that the drilled shafts will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations

S. Kelso Railroad Grade Separation

ESTIMATED AXIAL SHAFT RESISTANCE
7-foot Diameter Drilled Shaft
Piers 2 and 3

August 2018 24-1-04201-00

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E12

APPENDIX F

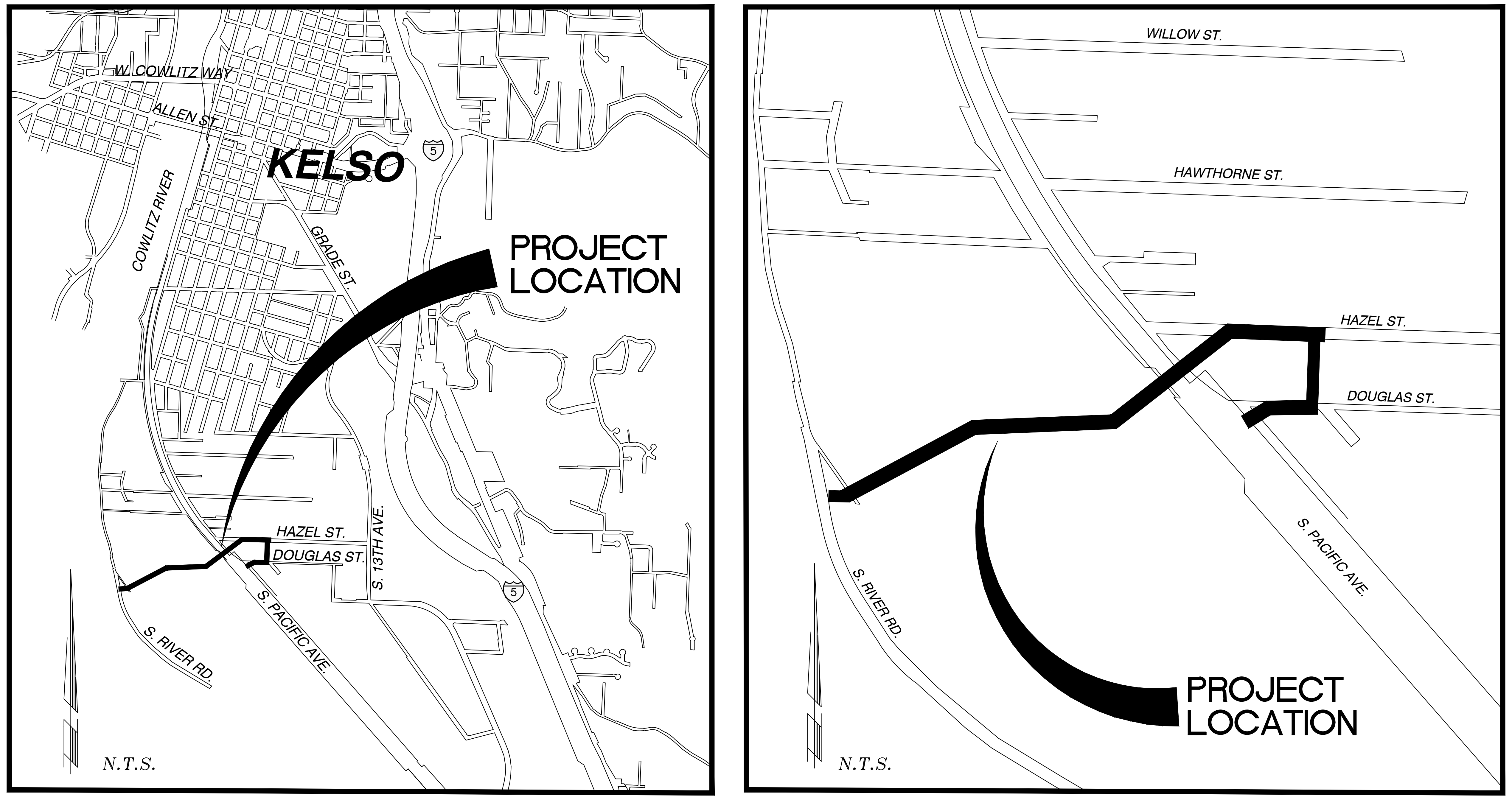
30 PERCENT DESIGN DRAWINGS

SOUTH KELSO RAILROAD CROSSING

HAZEL STREET FROM SOUTH RIVER ROAD TO 3RD AVENUE EXTENSION
INCLUDING 3RD AVE. SOUTH, DOUGLAS STREET, AND S. PACIFIC AVE.

CITY PROJECT #______

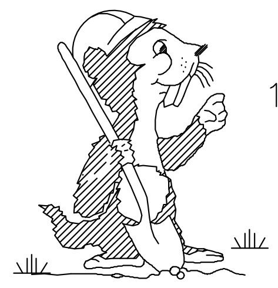
PLANS FOR THE CONSTRUCTION OF BRIDGE, RETAINING WALLS, ROADWAYS, AND STORM DRAINAGE



VICINITY MAPS

INDEX OF SHEETS

SHT.	DWG.	DESCRIPTION
1	A01	ALIGNMENT INDEX MAP
2	T01	TYPICAL SECTIONS
3	C01	HAZEL STREET PLAN AND PROFILE
4	C02	HAZEL STREET PLAN AND PROFILE
5	C03	HAZEL STREET PLAN AND PROFILE
6	C04	HAZEL STREET PLAN AND PROFILE
7	C05	HAZEL STREET PLAN AND PROFILE
8	C06	HAZEL STREET PLAN AND PROFILE
9	C07	HAZEL STREET PLAN AND PROFILE
10	C08	3RD AVE. SOUTH PLAN AND PROFILE
11	C09	S. PACIFIC AVE. PLAN AND PROFILE
12	C10	DOUGLAS STREET PLAN AND PROFILE
13	C11	S. PACIFIC AVE. (SOUTH OF DOUGLAS ST.) PLAN AND PROFILE
14	C12	HAZEL COURT PLAN AND PROFILE
15	SW01	WEST POND #1 PLAN AND PROFILE
16	SW02	WEST POND #2 PLAN AND PROFILE
17	SW03	EAST POND PLAN AND PROFILE
18	SW04	SLOUGH CROSSING PLAN AND PROFILE
19	W01	WEST ABUTMENT WALL PLAN AND ELEVATION
20	W02	EAST ABUTMENT WALL PLAN AND ELEVATION
21	W03	ROADWAY WALL PLAN AND ELEVATION
22	W04	WALL TYPICAL SECTIONS
23	W05	WALL TYPICAL SECTIONS
24	W06	WALL CAP DETAILS
25	W07	STONE COLUMN GROUND IMPROVEMENTS – WEST BRIDGE APPROACH
26	W08	STONE COLUMN GROUND IMPROVEMENTS – EAST BRIDGE APPROACH
27	W09	STONE COLUMN GROUND IMPROVEMENTS – ROADWAY WALL
28	B01	BRIDGE PLAN AND ELEVATION
29	B02	BRIDGE TYPICAL SECTIONS
30	B03	BRIDGE CONSTRUCTION SEQUENCE – 1
31	B04	BRIDGE CONSTRUCTION SEQUENCE – 2



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CITY OF KELSO
APPROVED FOR CONSTRUCTION

CITY ENGINEER DATE

30% PRELIMINARY PLANS
NOT FOR CONSTRUCTION

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VAN: GRANTE 02/04/2010 2:29pm --> VAN\Dwg\C15299-A1.DWG

Plotted: Sep 10, 2018 - 9:44am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913A01.dwg Layout Name: A01

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ALLEN THOMAS HENDY
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PRELIMINARY

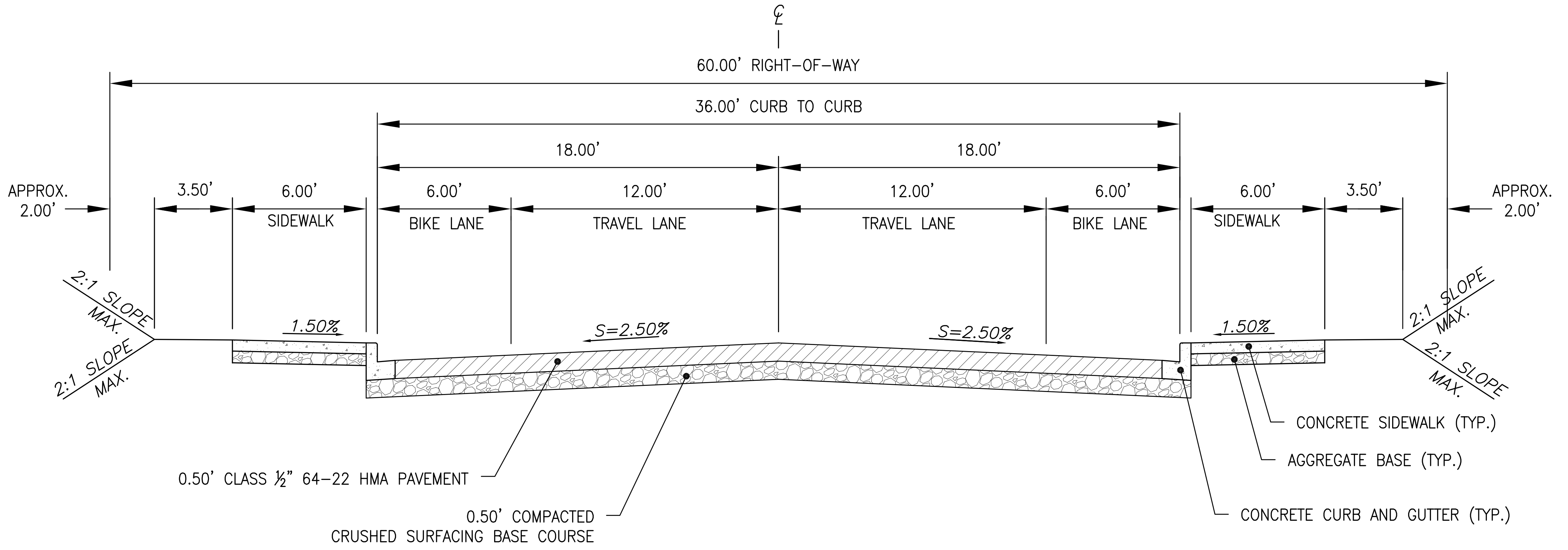
REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: A01

SOUTH KELSO RAILROAD CROSSING
ALIGNMENT INDEX MAP

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
1

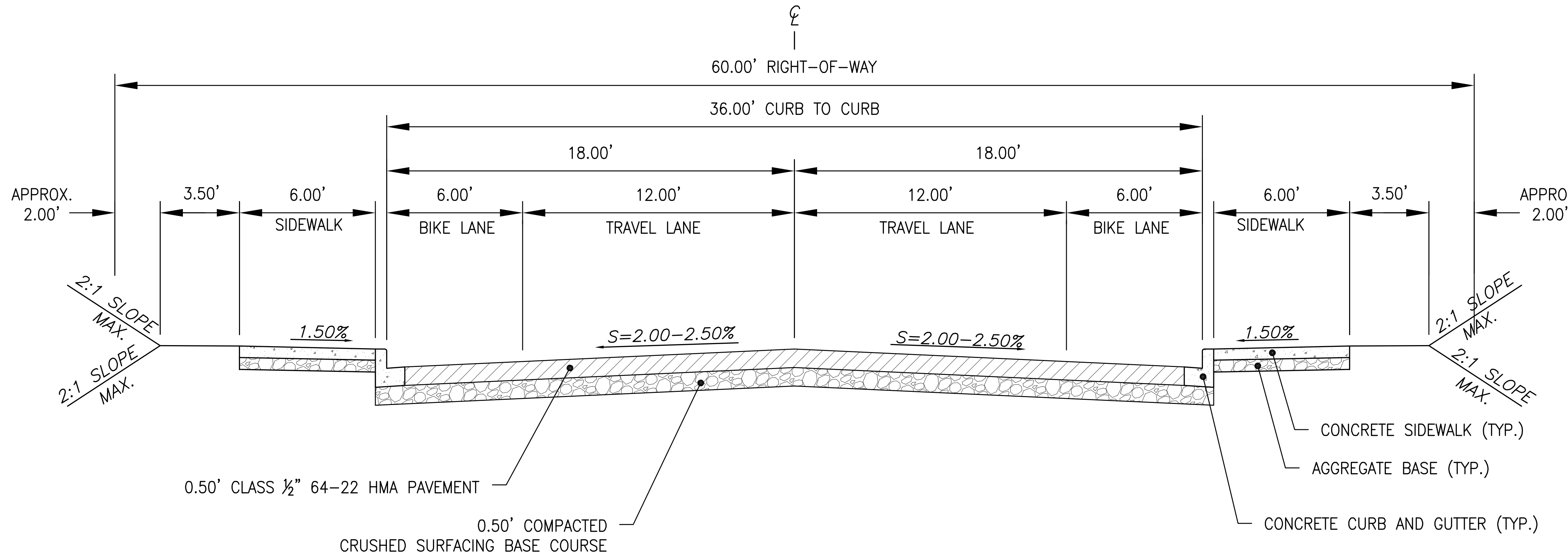


HAZEL STREET TYPICAL SECTION

FROM S. RIVER RD TO 3RD AVE.

STA: 10+50.46 TO 27+09.20
STA: 32+54.20 TO 38+93.28

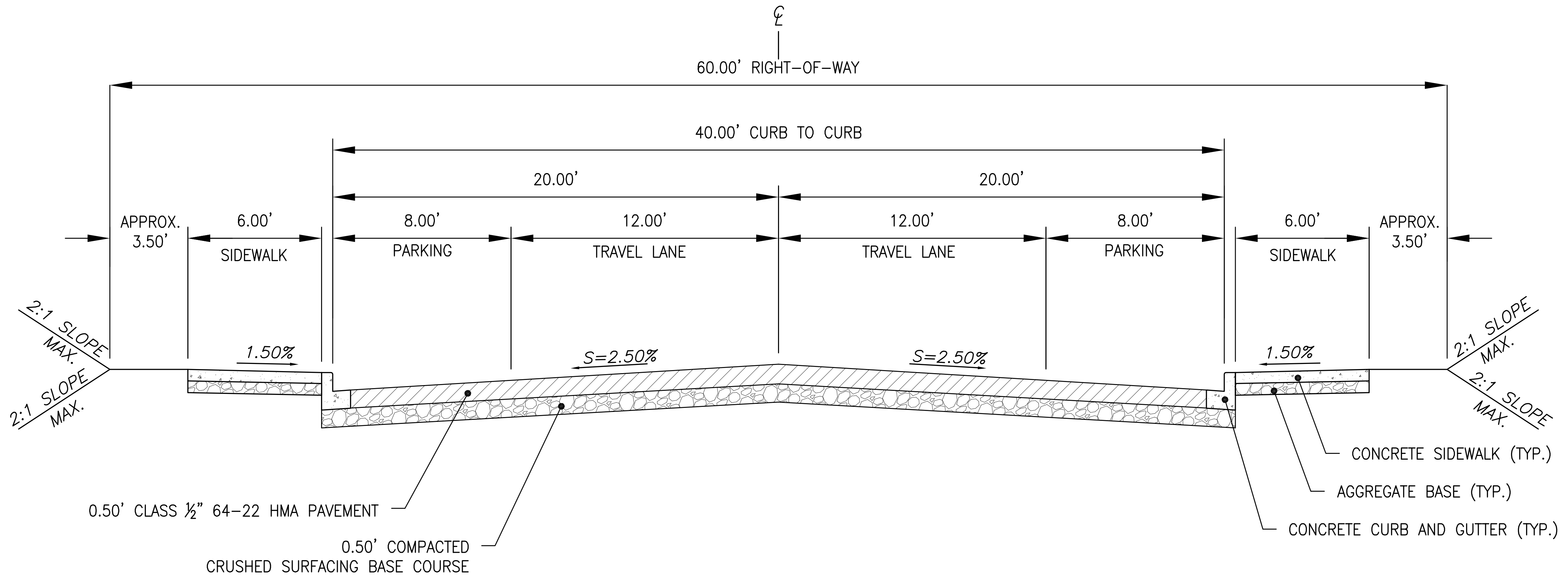
NOTE:
SEE BRIDGE PLANS, SHEET B02
FOR BRIDGE TYPICAL SECTIONS



HAZEL STREET TYPICAL SECTION

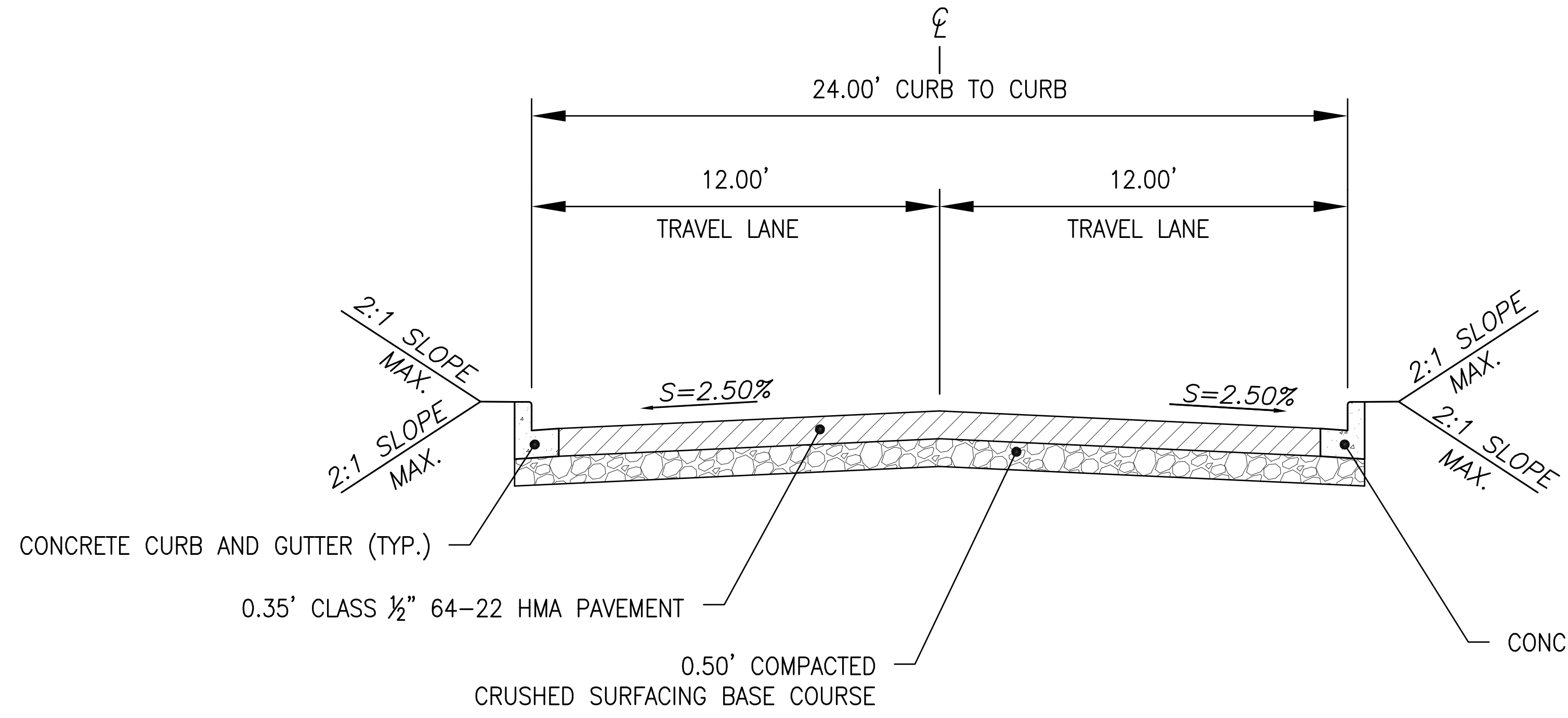
FROM S. RIVER RD TO 3RD AVE.

STA: 27+09.20 TO 27+59.20 TRANSITION ROAD CROSS SLOPE FROM 2.50% TO 2.00%
STA: 32+04.20 TO 32+54.20 TRANSITION ROAD CROSS SLOPE FROM 2.00% TO 2.50%



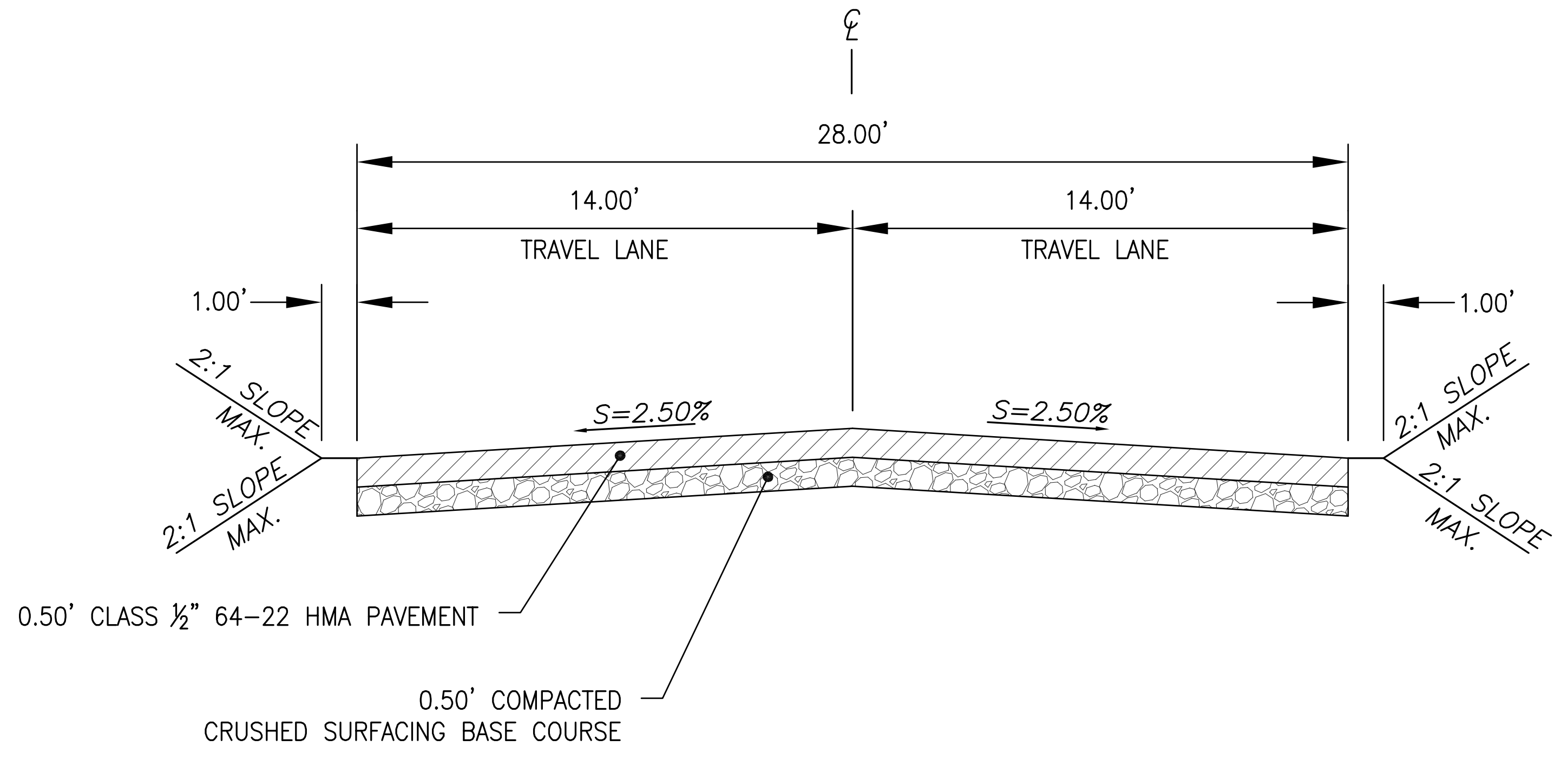
DOUGLAS STREET & 3RD AVENUE TYPICAL SECTION

DOUGLAS STREET STA: 105+40.77 TO 110+26.48
3RD AVENUE STA: 200+76.72 TO 204+13.91



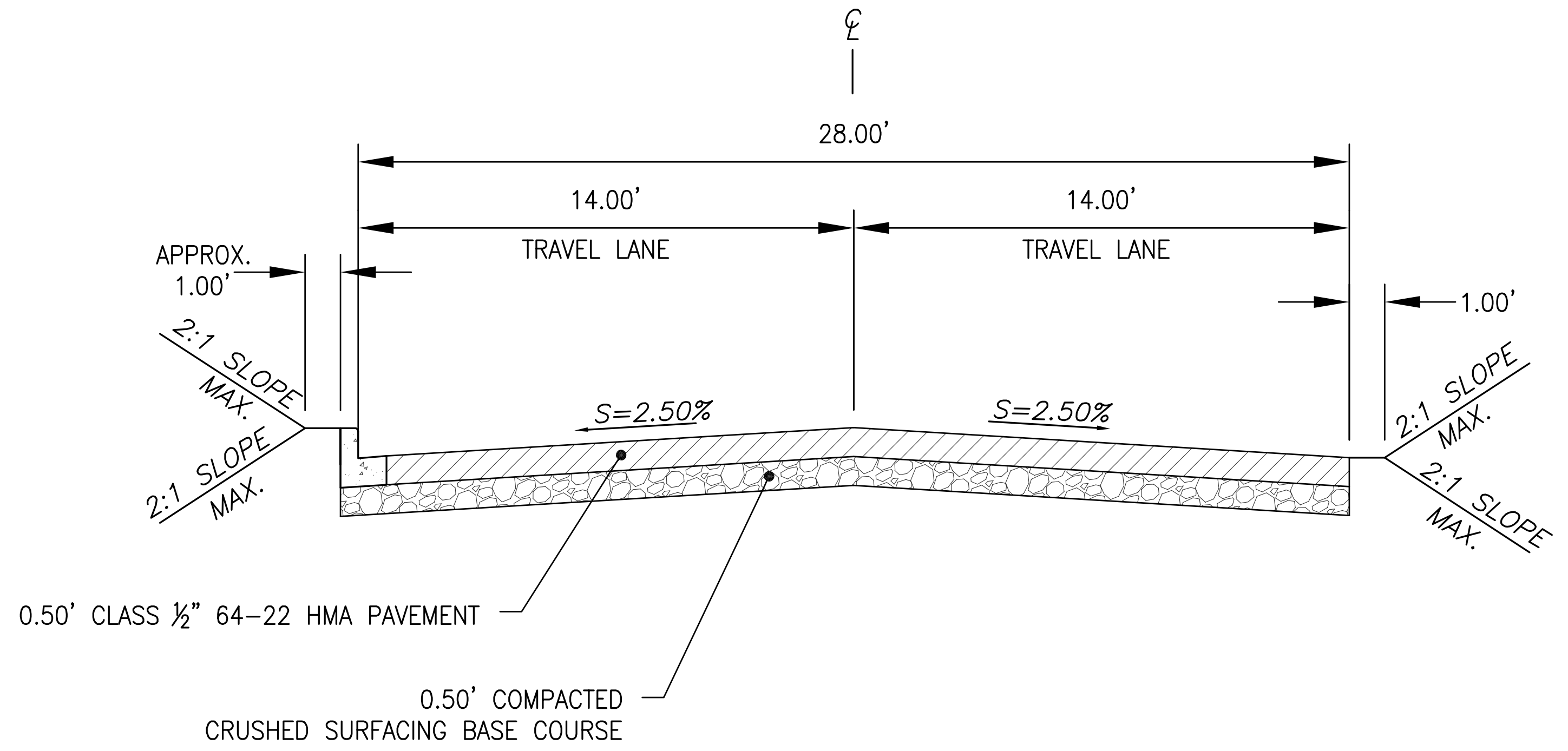
HAZEL COURT TYPICAL SECTION

STA: 400+20.12 TO 402+64.64



S. PACIFIC AVENUE (SOUTH) TYPICAL SECTION

STA: 300+50.82 TO 301+55.71



S. PACIFIC AVENUE TYPICAL SECTION

STA: 100+22.91 TO 105+26.55

5 0 5 10 HORIZ
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Checked By: ATH
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SCALE: AS NOTED
DWG NO.: T01

SOUTH KELSO RAILROAD CROSSING
TYPICAL SECTIONS

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
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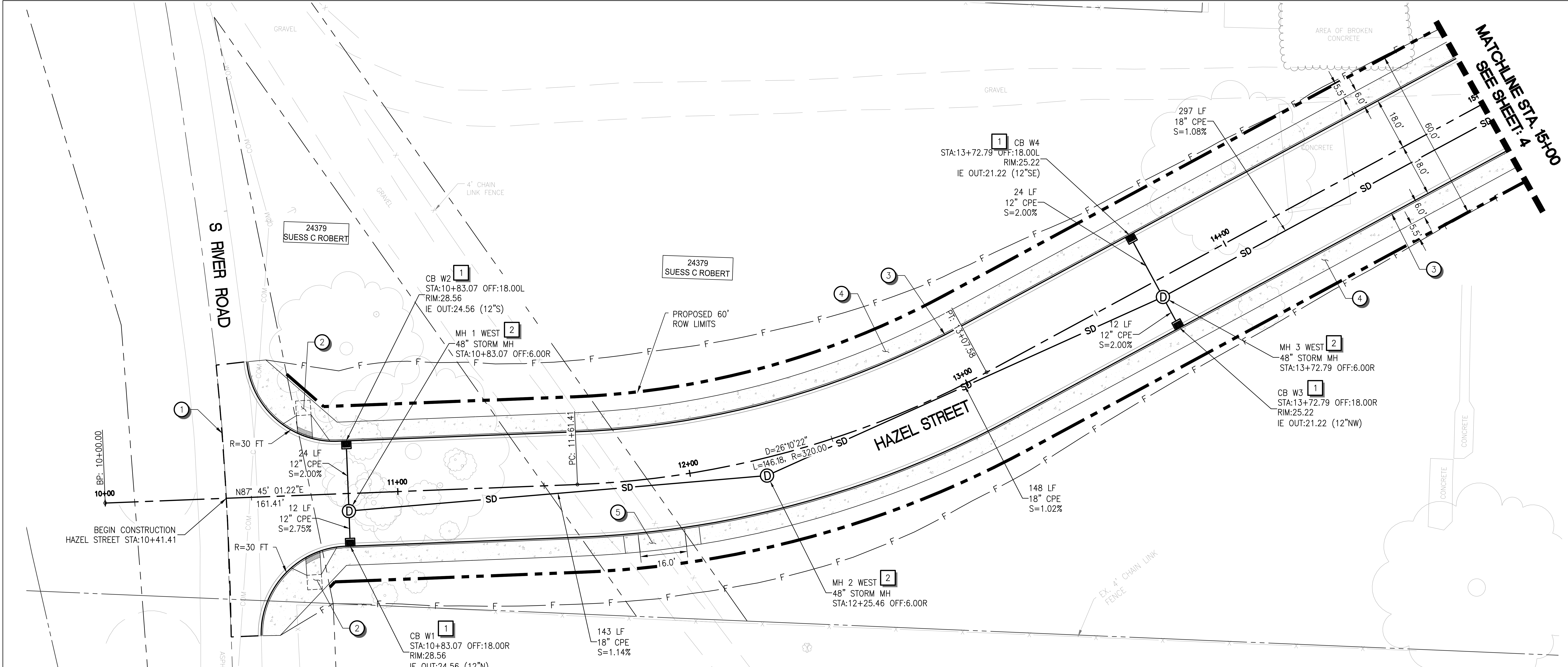
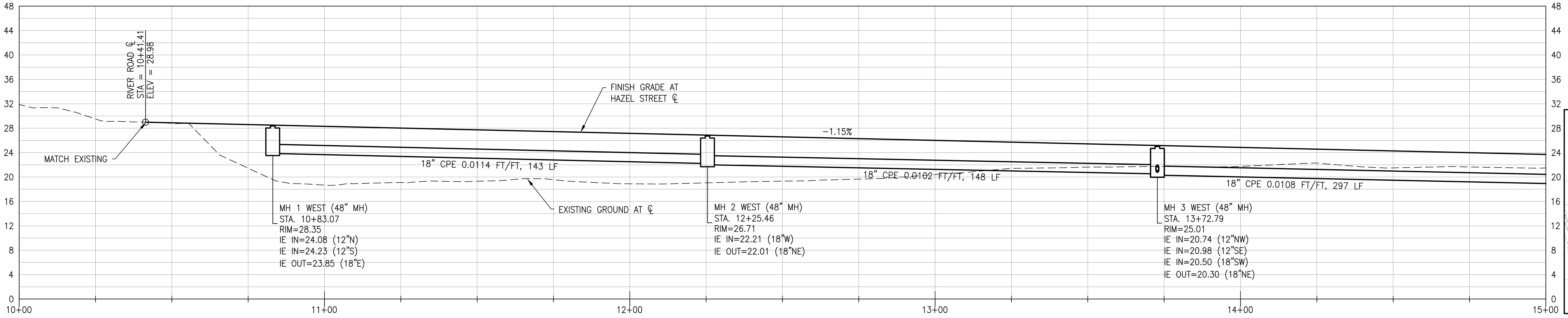
SHEET NUMBER
2

Plotted: Sep 10, 2018 - 3:06pm V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913T01.dwg Layout Name: T01



PLAN



PROFILE

CONSTRUCTION NOTES

① SAWCUT, TYP

② INSTALL CURB RAMP C PER CITY OF KELSO STANDARD PLAN ST-130 AND ST-140

③ INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST-110

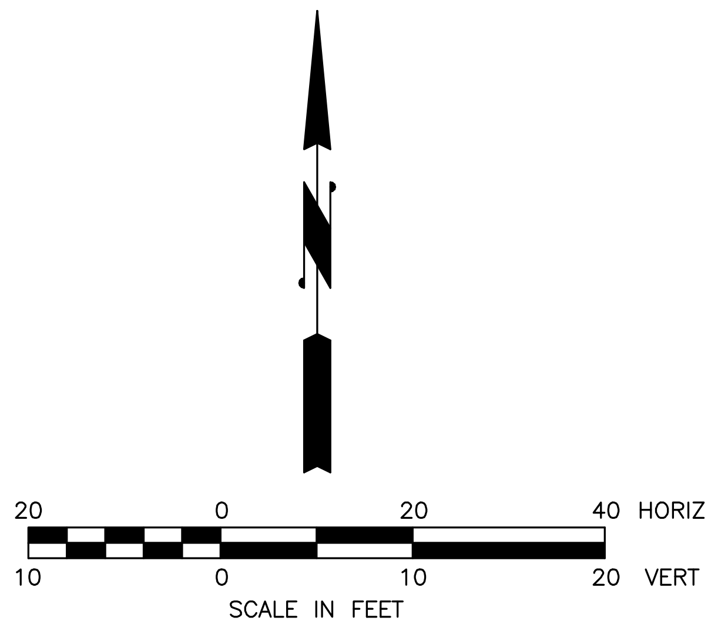
④ INSTALL SIDEWALK PER CITY OF KELSO STANDARD PLAN ST-150

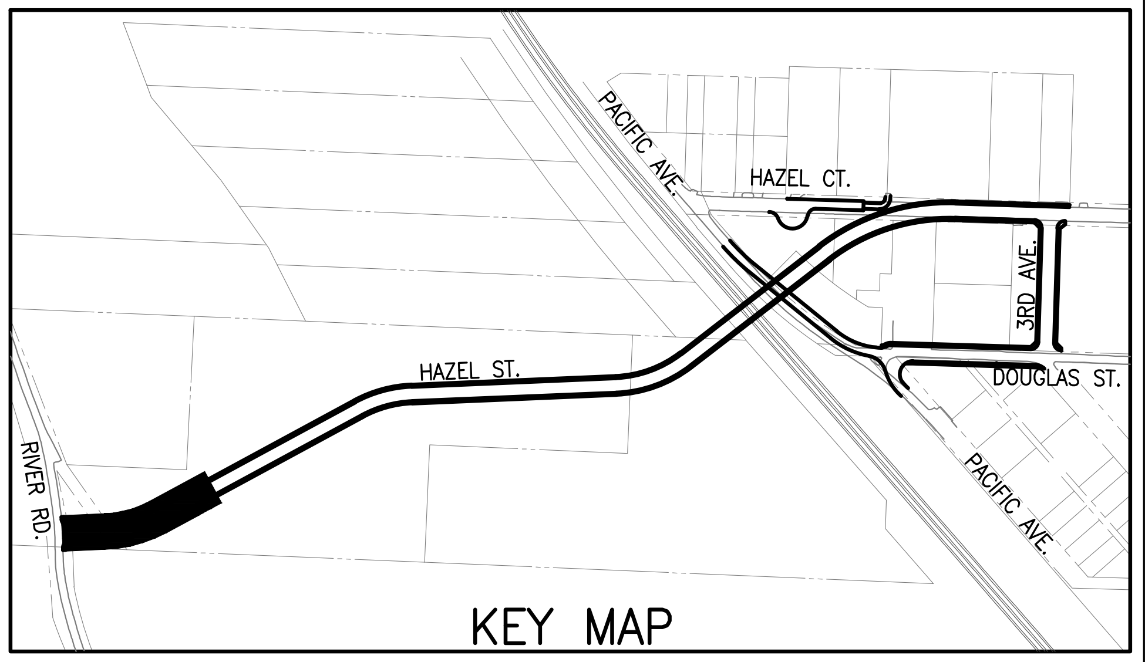
⑤ INSTALL DRIVEWAY APPROACH PER CITY OF KELSO STANDARD PLAN ST-160

STORMWATER NOTES

1 INSTALL WSDOT B-25.20-02 COMBINATION INLET TYPE 1

2 INSTALL WSDOT B-15.60-02 MANHOLE TYPE 3





KEY MAP

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REVISION	DATE	DESCRIPTION	BY

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SCALE: AS NOTED
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SOUTH KELSO RAILROAD CROSSING
HAZEL STREET PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
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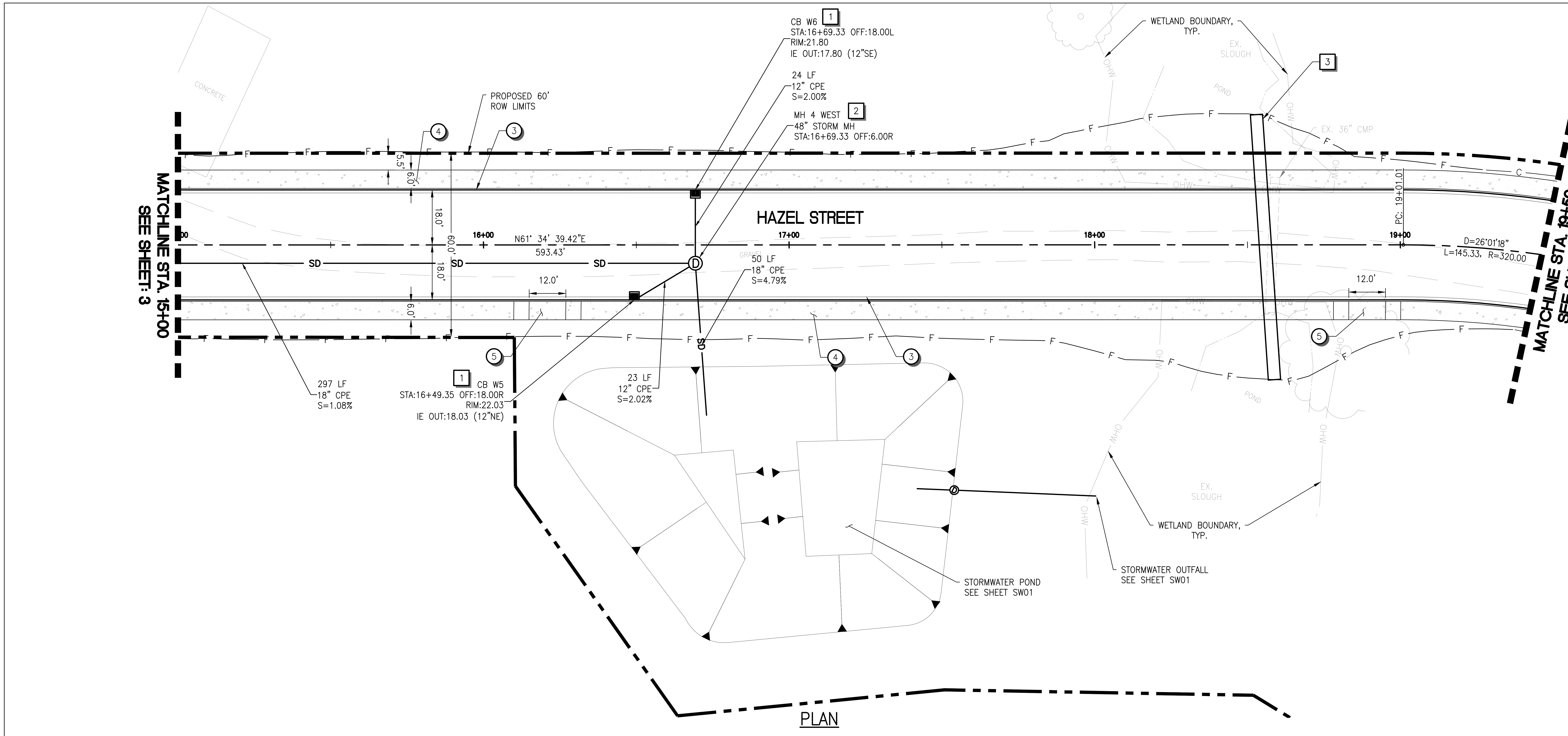
SHEET NUMBER
3

Plotted: Sep 10, 2018 - 9:45am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913C01.dwg Layout Name: C01



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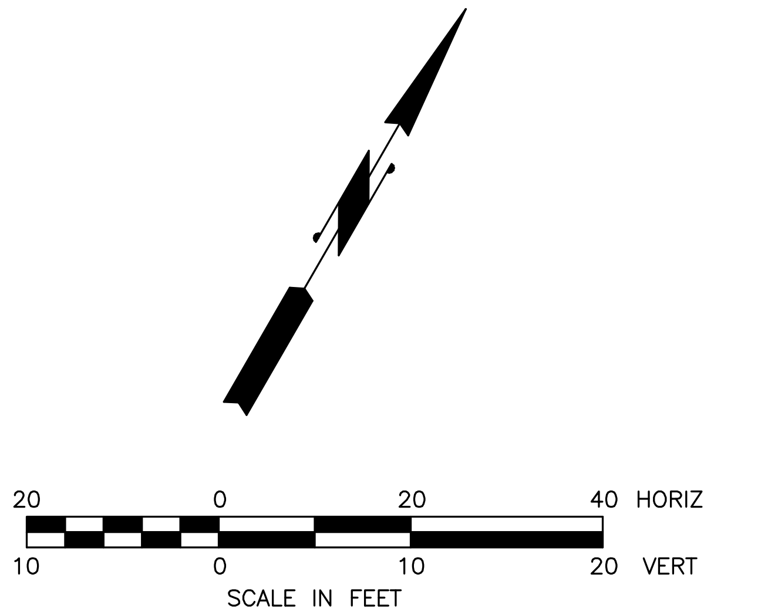


CONSTRUCTION NOTES

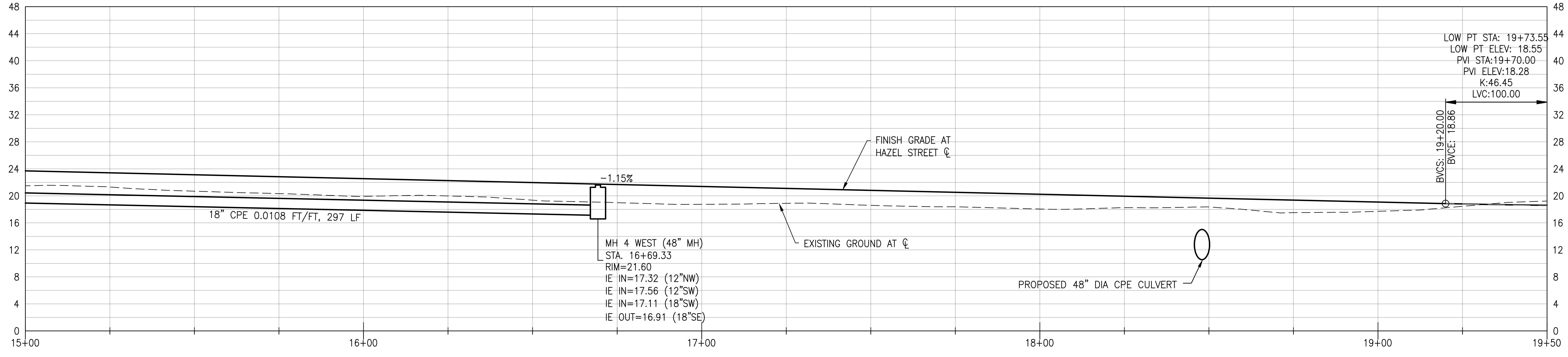
③ INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST-110

④ INSTALL SIDEWALK PER CITY OF KELSO STANDARD PLAN ST-150

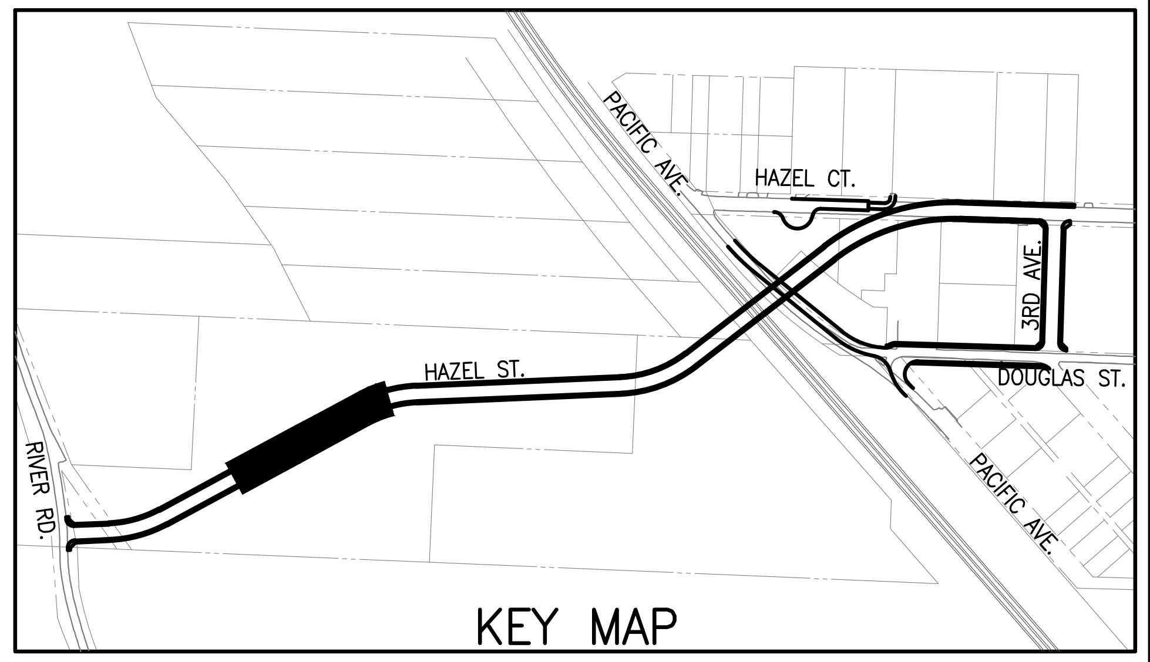
⑤ INSTALL DRIVEWAY APPROACH PER CITY OF KELSO STANDARD PLAN ST-160

STORMWATER NOTES

1. INSTALL WSDOT B-25.20-02 COMBINATION INLET TYPE 1
2. INSTALL WSDOT B-15.60-02 MANHOLE TYPE 3
3. PROPOSED 48" DIA. CPE CULVERT, SEE SHEET SW04



PROFILE



KEY MAP

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DATE: 09/2018
SCALE: AS NOTED
DWG NO.: C02

SOUTH KELSO RAILROAD CROSSING
HAZEL STREET PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
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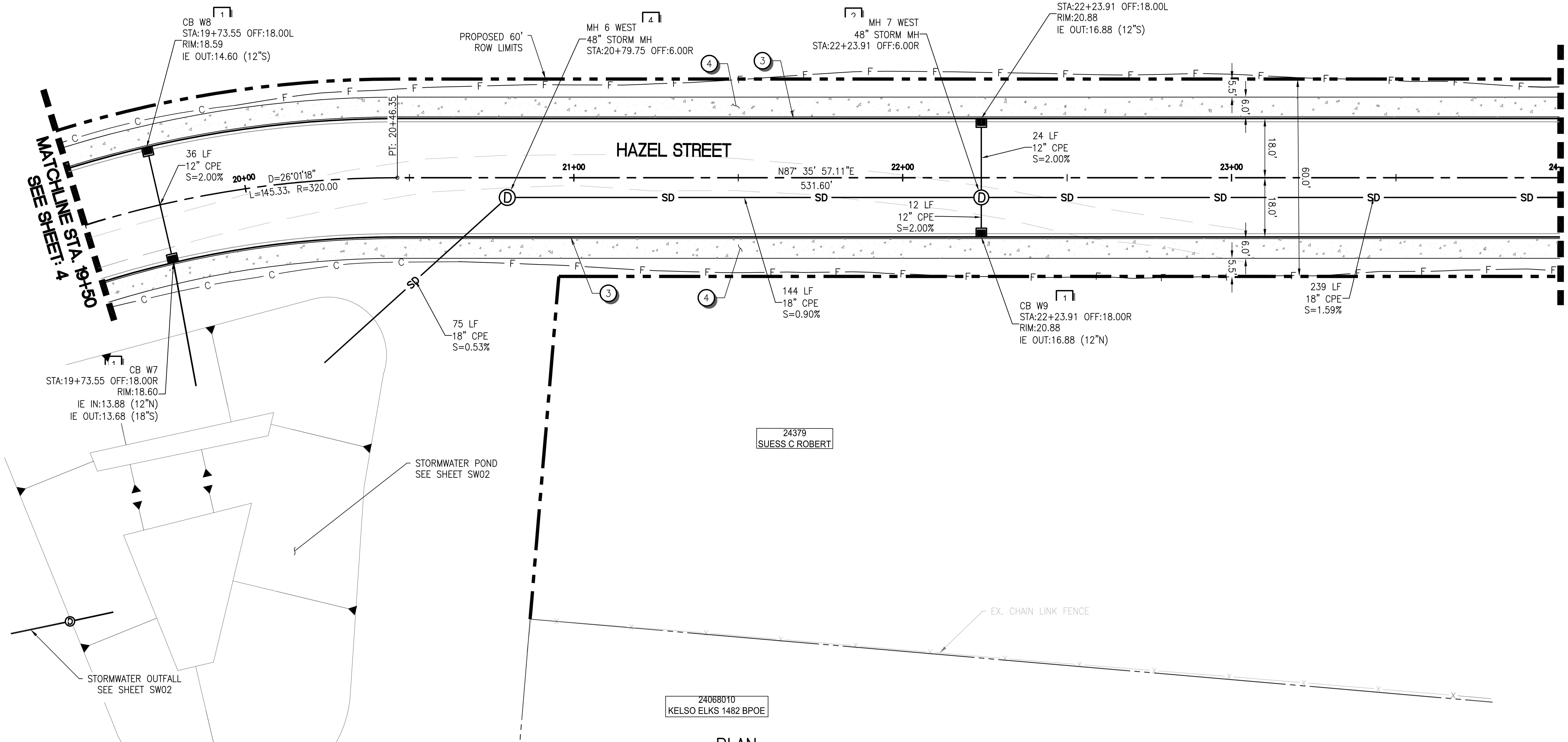
SHEET NUMBER
4



PLAN

CONSTRUCTION NOTES

③ INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST–110

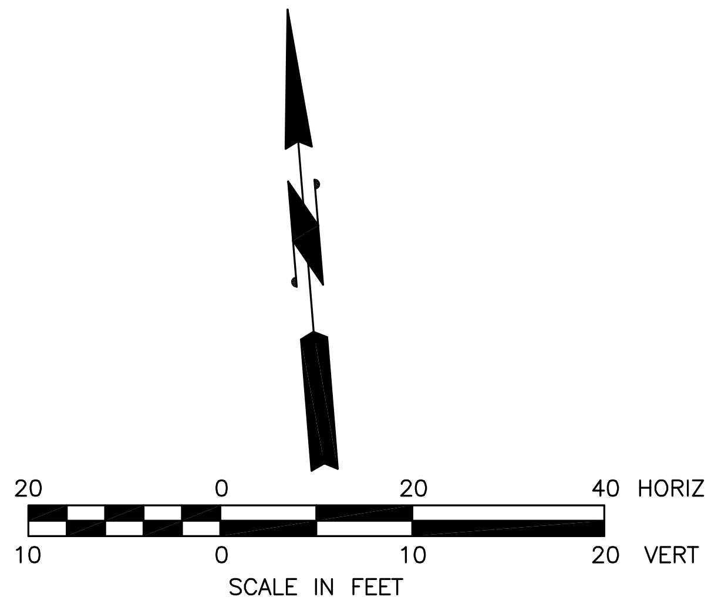
④ INSTALL SIDEWALK PER CITY OF KELSO STANDARD PLAN ST–150

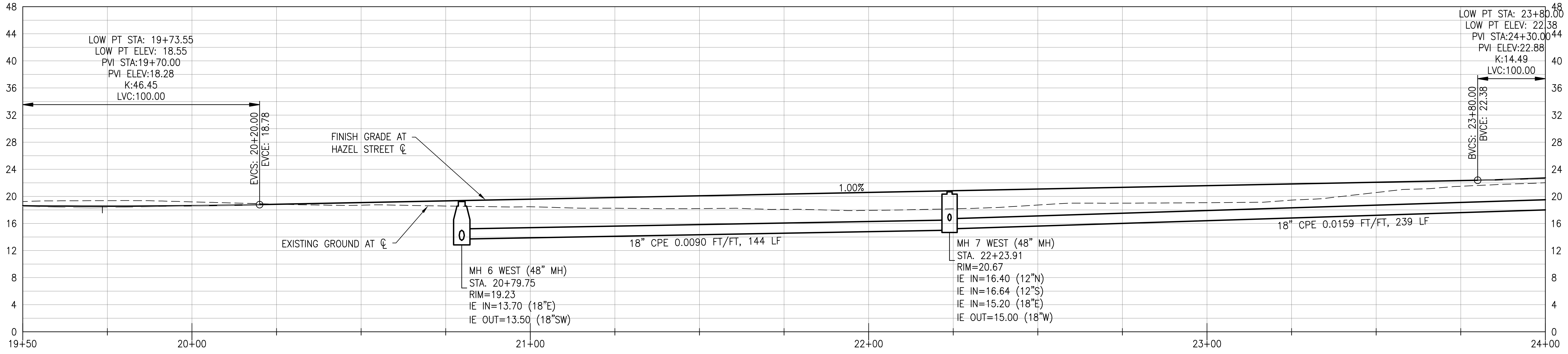
STORMWATER NOTES

1 INSTALL WSDOT B–25.20–02 COMBINATION INLET TYPE 1

2 INSTALL WSDOT B–15.60–02 MANHOLE TYPE 3

4 INSTALL WSDOT B–15.20–01 MANHOLE TYPE 1





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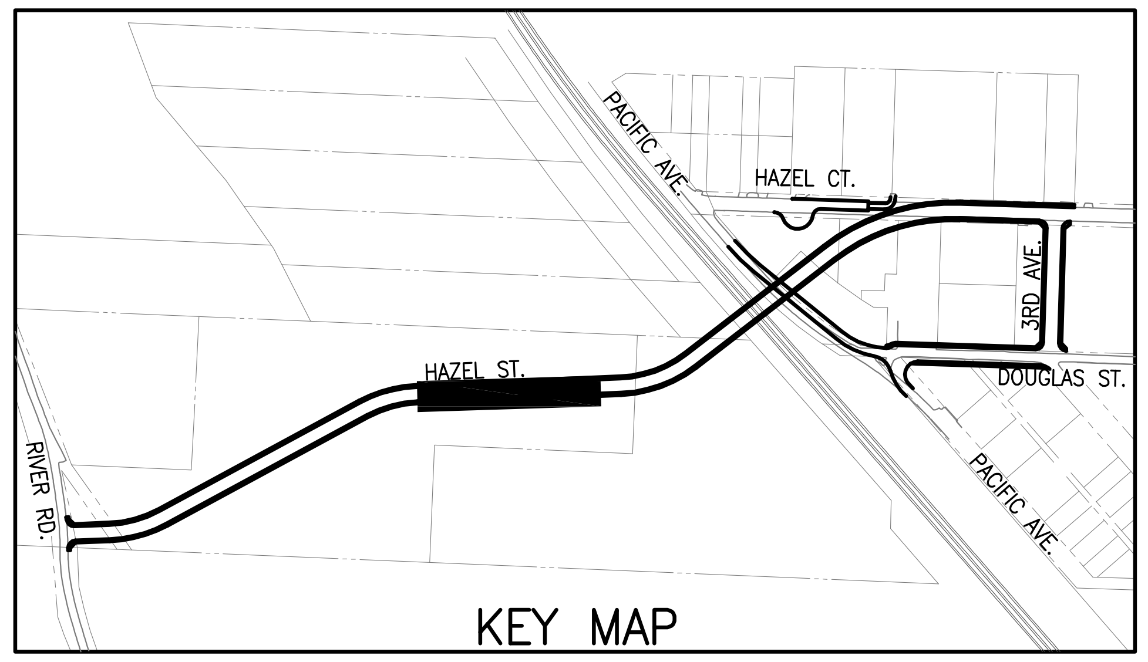

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SOUTH KELSO RAILROAD CROSSING
HAZEL STREET PLAN AND PROFILE

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SHEET NUMBER
5

Plotted: Sep 10, 2018 – 2:56pm V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913C01.dwg Layout Name: C03

CONSTRUCTION NOTES

③ INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST-110

④ INSTALL SIDEWALK PER CITY OF KELSO STANDARD PLAN ST-150

⑥ BEAM GUARDRAIL TYPE 31 SEE WSDOT DETAIL C-24.10-01, C-20.10-04, C-22.45-03, C-25.30-00

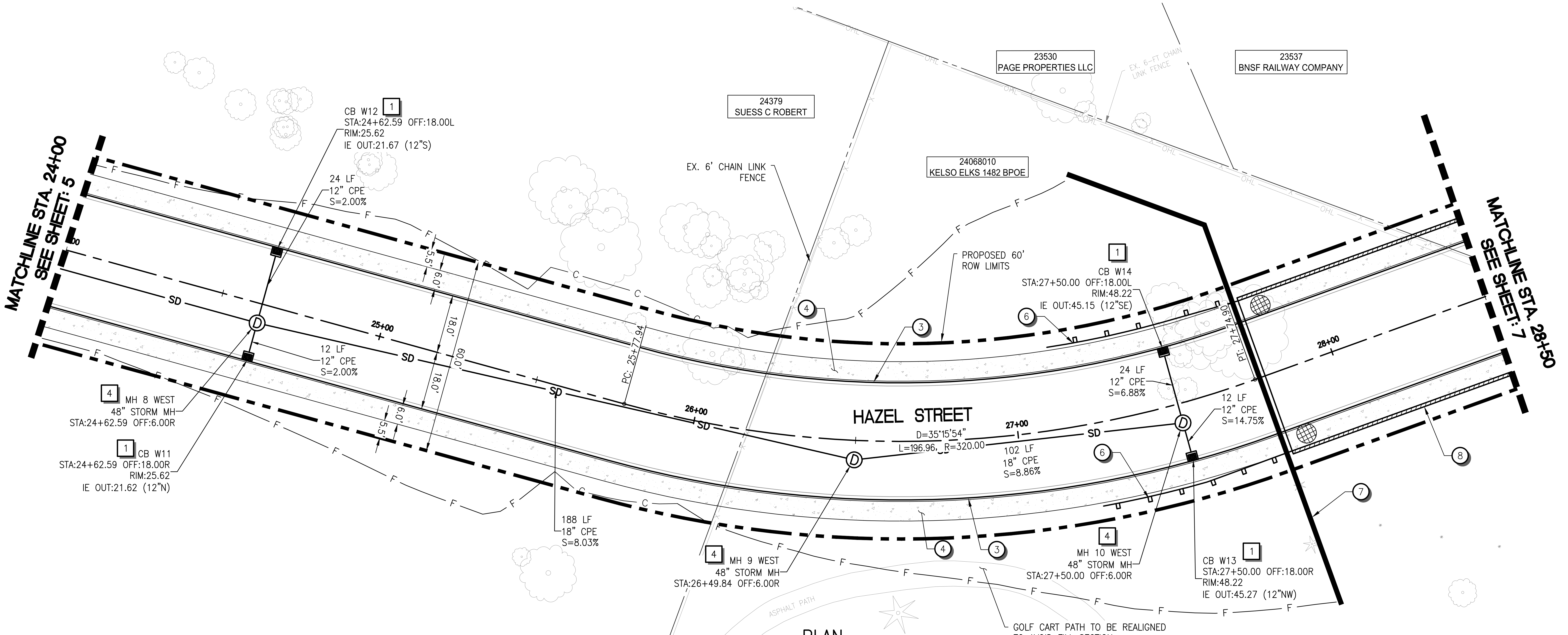
⑦ BRIDGE APPROACH ABUTMENT. SEE STRUCTURAL SHEETS FOR DETAILS.

⑧ HAZEL STREET BRIDGE CROSSING. SEE STRUCTURAL SHEETS

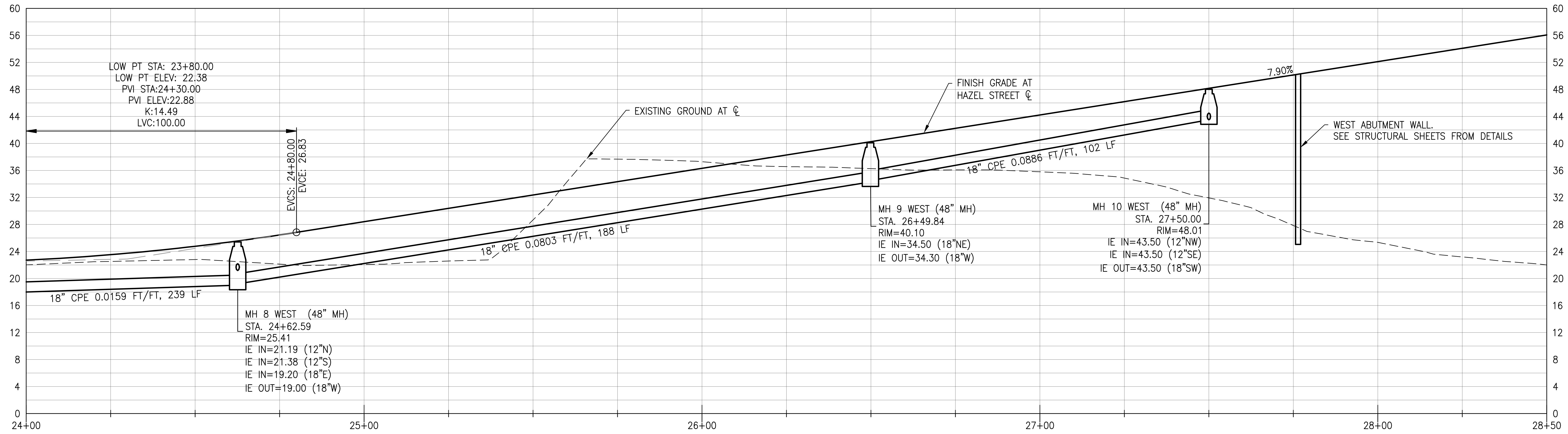
STORMWATER NOTES

1 INSTALL WSDOT B-25.20-02 COMBINATION INLET TYPE 1

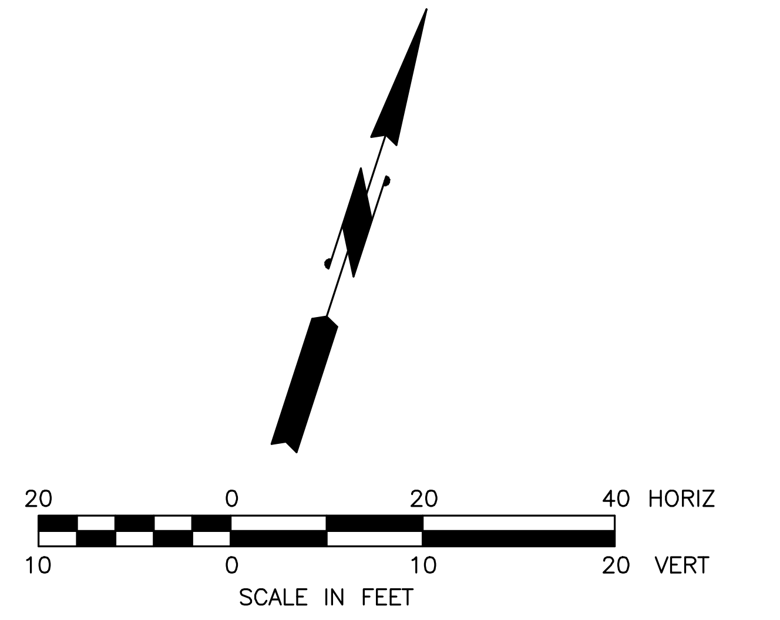
4 INSTALL WSDOT B-15.20-01 MANHOLE TYPE 1

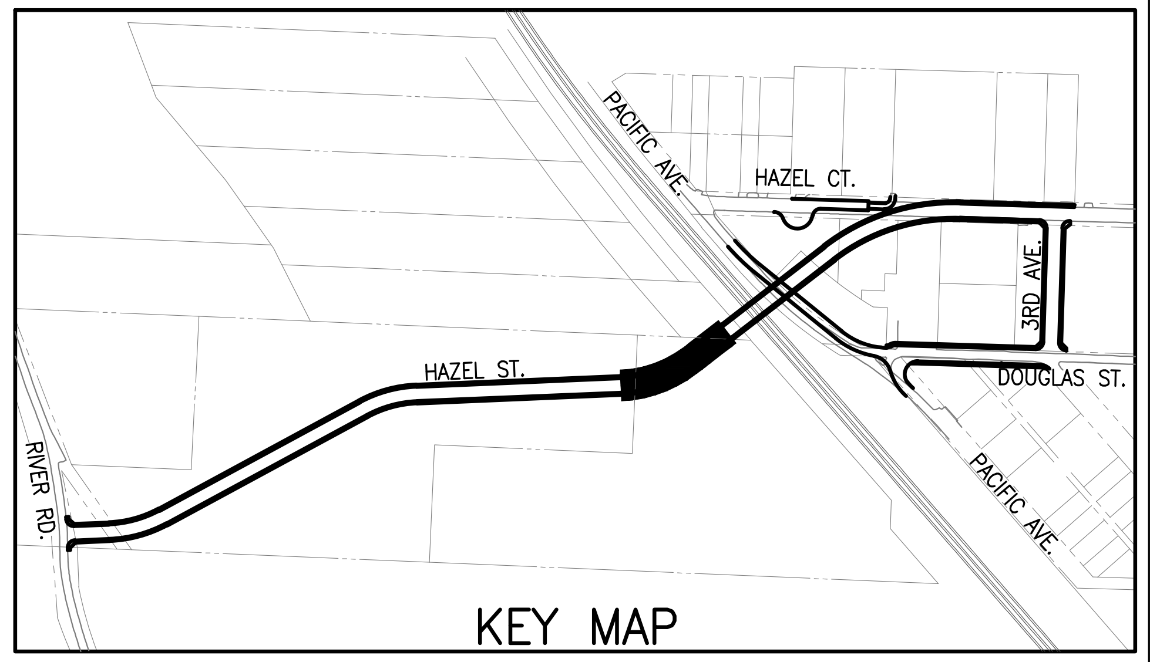


PLAN



PROFILE





KEY MAP

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DATE: 09/2018
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SOUTH KELSO RAILROAD CROSSING
HAZEL STREET PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
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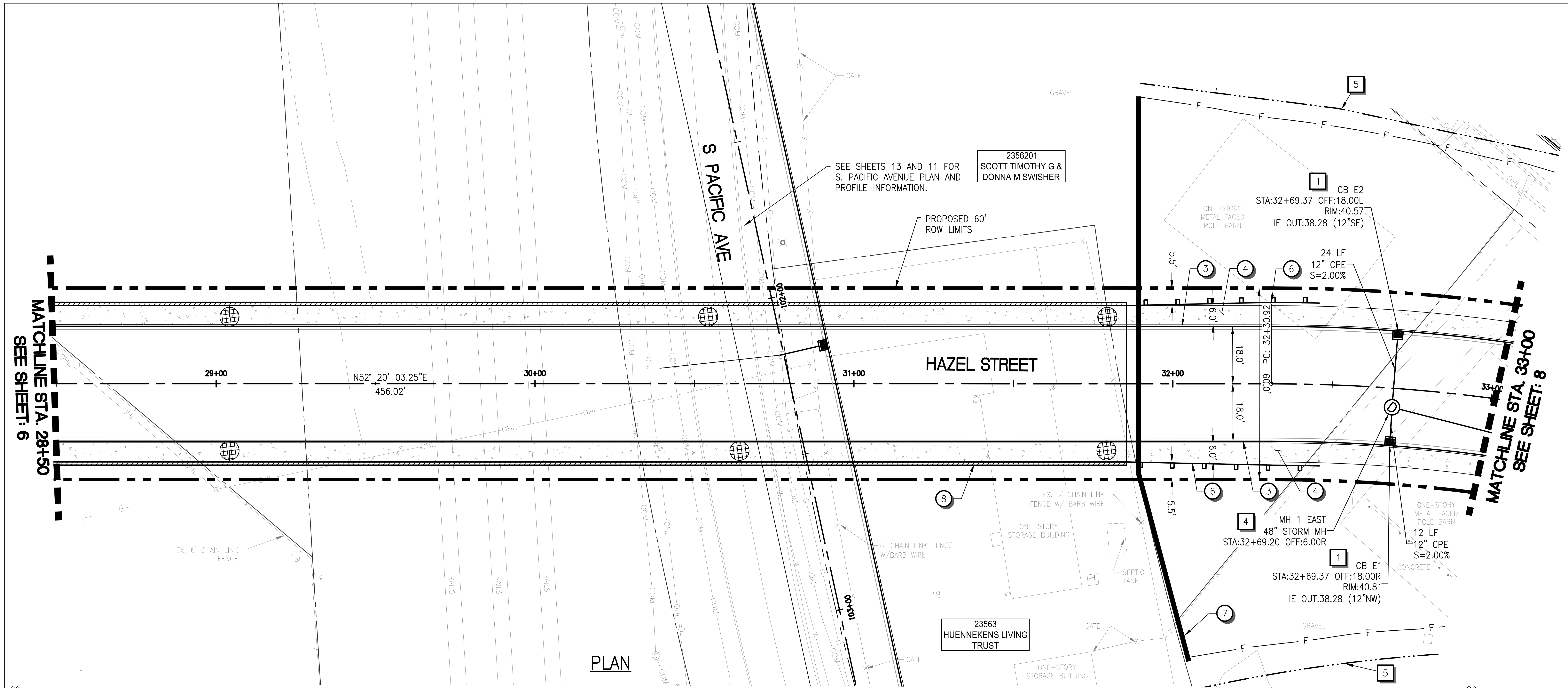
SHEET NUMBER
6



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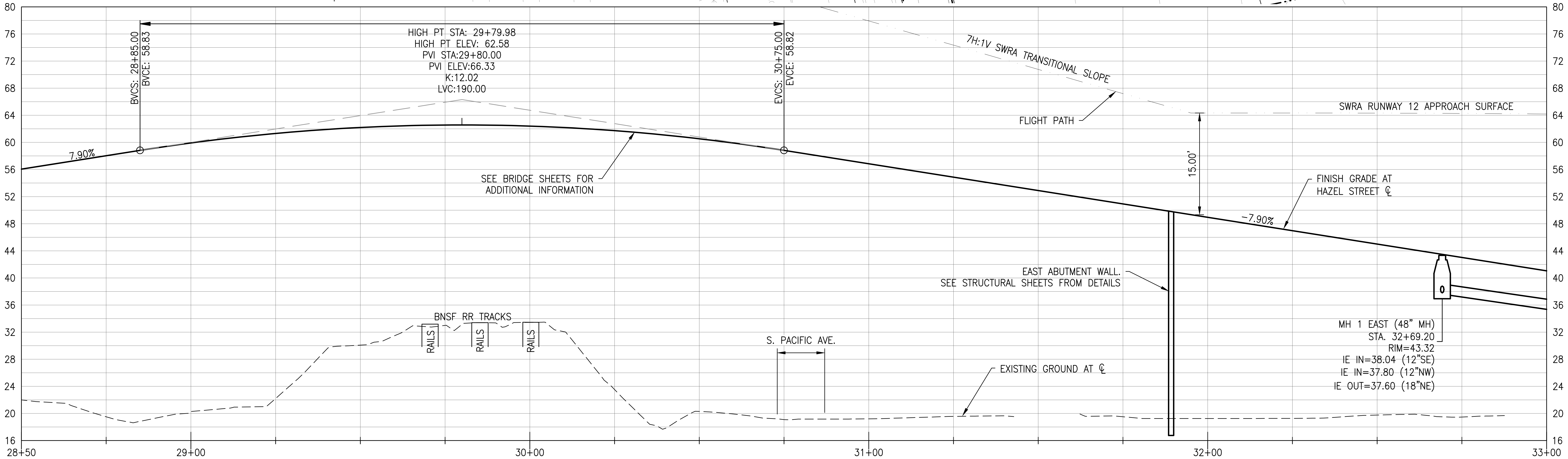


PROFILE

CONSTRUCTION NOTES

③ INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST-110

④ INSTALL SIDEWALK PER CITY OF KELSO STANDARD PLAN ST-150

⑥ BEAM GUARDRAIL TYPE 31
SEE WSDOT DETAIL C-24.10-01, C-20.10-04, C-22.45-03, C-25.30-00

⑦ BRIDGE APPROACH ABUTMENT. SEE STRUCTURAL SHEETS FOR DETAILS.

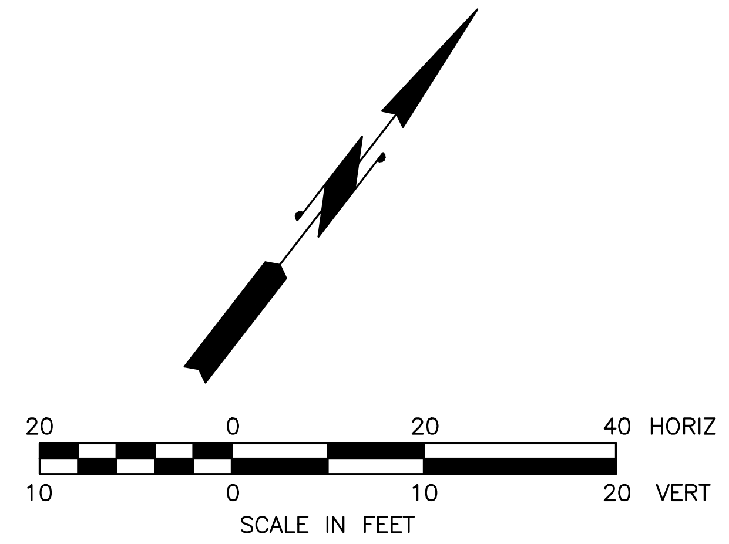
⑧ HAZEL STREET BRIDGE CROSSING. SEE STRUCTURAL SHEETS

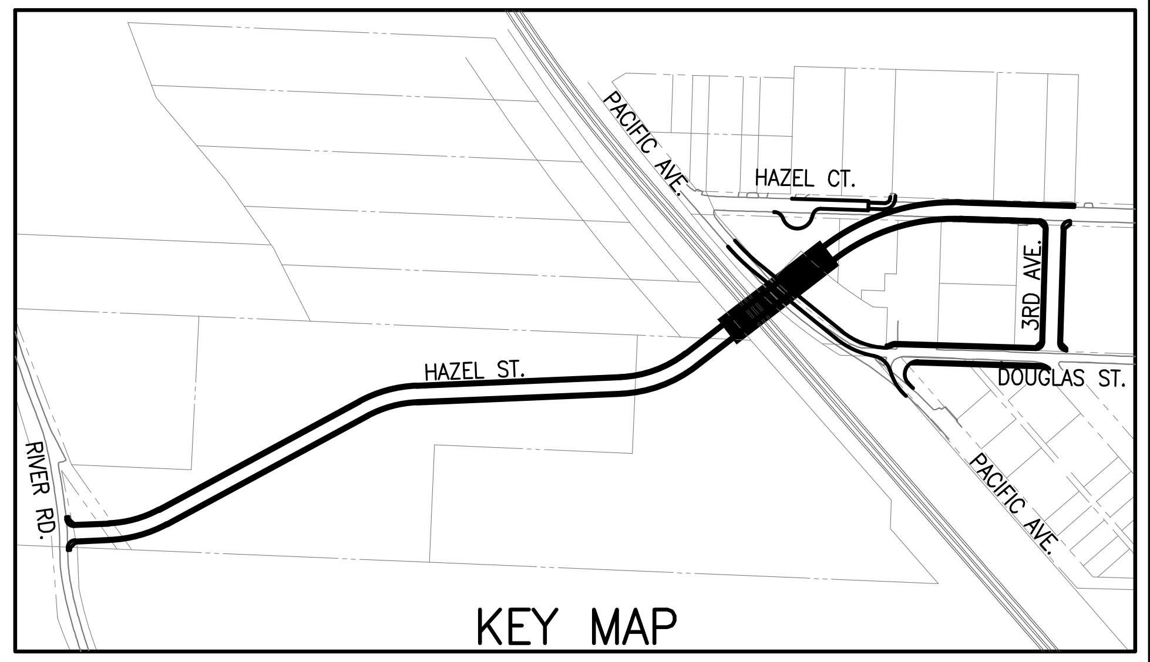
STORMWATER NOTES

1 INSTALL WSDOT B-25.20-02 COMBINATION INLET TYPE 1

4 INSTALL WSDOT B-15.20-01 MANHOLE TYPE 1

5 CONSTRUCT DITCH





KEY MAP

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DATE: 09/2018
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SOUTH KELSO RAILROAD CROSSING
HAZEL STREET PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
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SHEET NUMBER
7

Plotted: Sep 10, 2018 - 9:46am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913C01.dwg Layout Name: C05

CONSTRUCTION NOTES

① SAWCUT, TYP

③ INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST-110

④ INSTALL SIDEWALK PER CITY OF KELSO STANDARD PLAN ST-150

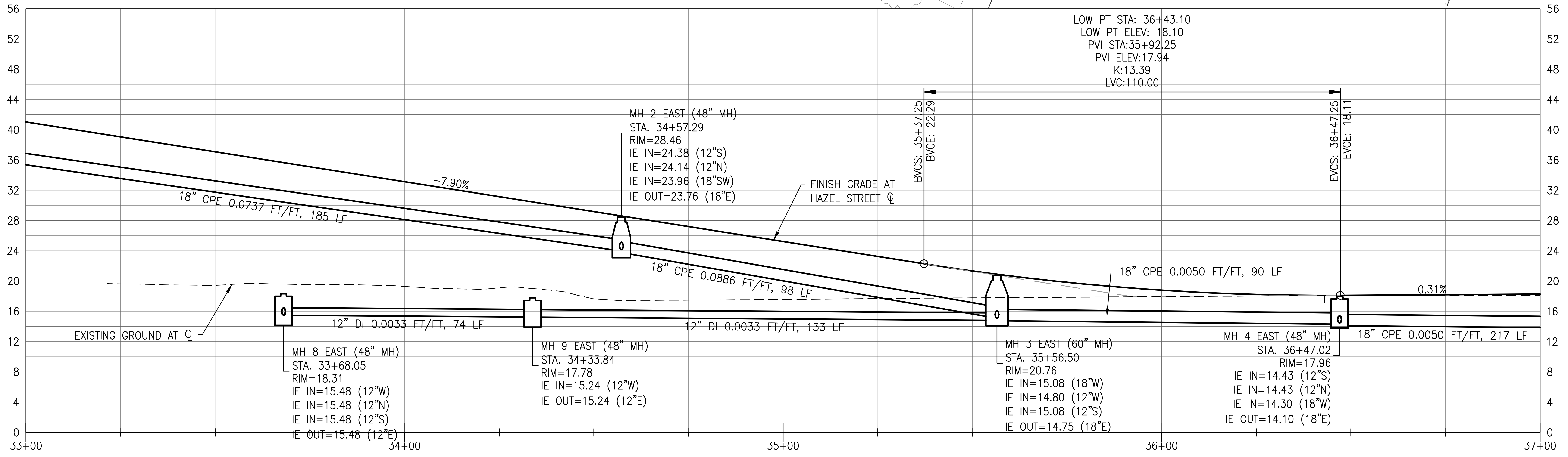
⑤ INSTALL DRIVEWAY APPROACH PER CITY OF KELSO STANDARD PLAN ST-160

⑨ INSTALL RETAINING WALL, SEE STRUCTURAL SHEETS FOR DETAILS.

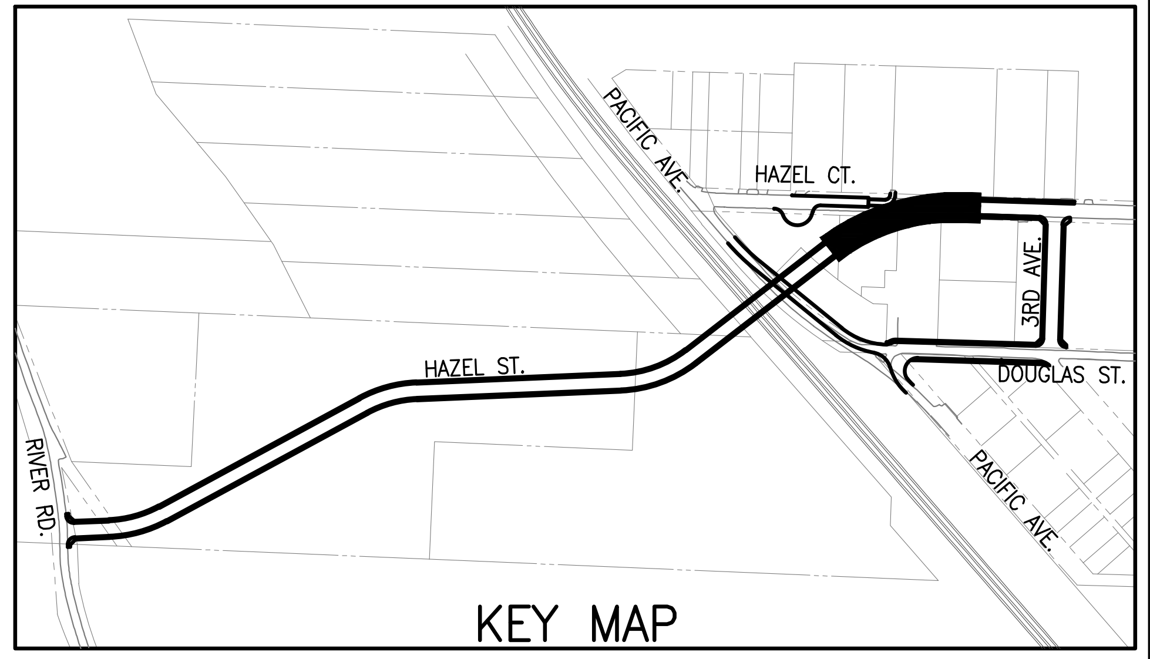
STORMWATER NOTES

1	INSTALL WSDOT B-25.20-02 COMBINATION INLET TYPE 1
2	INSTALL WSDOT B-15.60-02 MANHOLE TYPE 3
4	INSTALL WSDOT B-15.20-01 MANHOLE TYPE 1
5	CONSTRUCT DITCH
6	INSTALL FIELD INLET

PLAN



PROFILE



KEY MAP

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Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: C06

SOUTH KELSO RAILROAD CROSSING
HAZEL STREET PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
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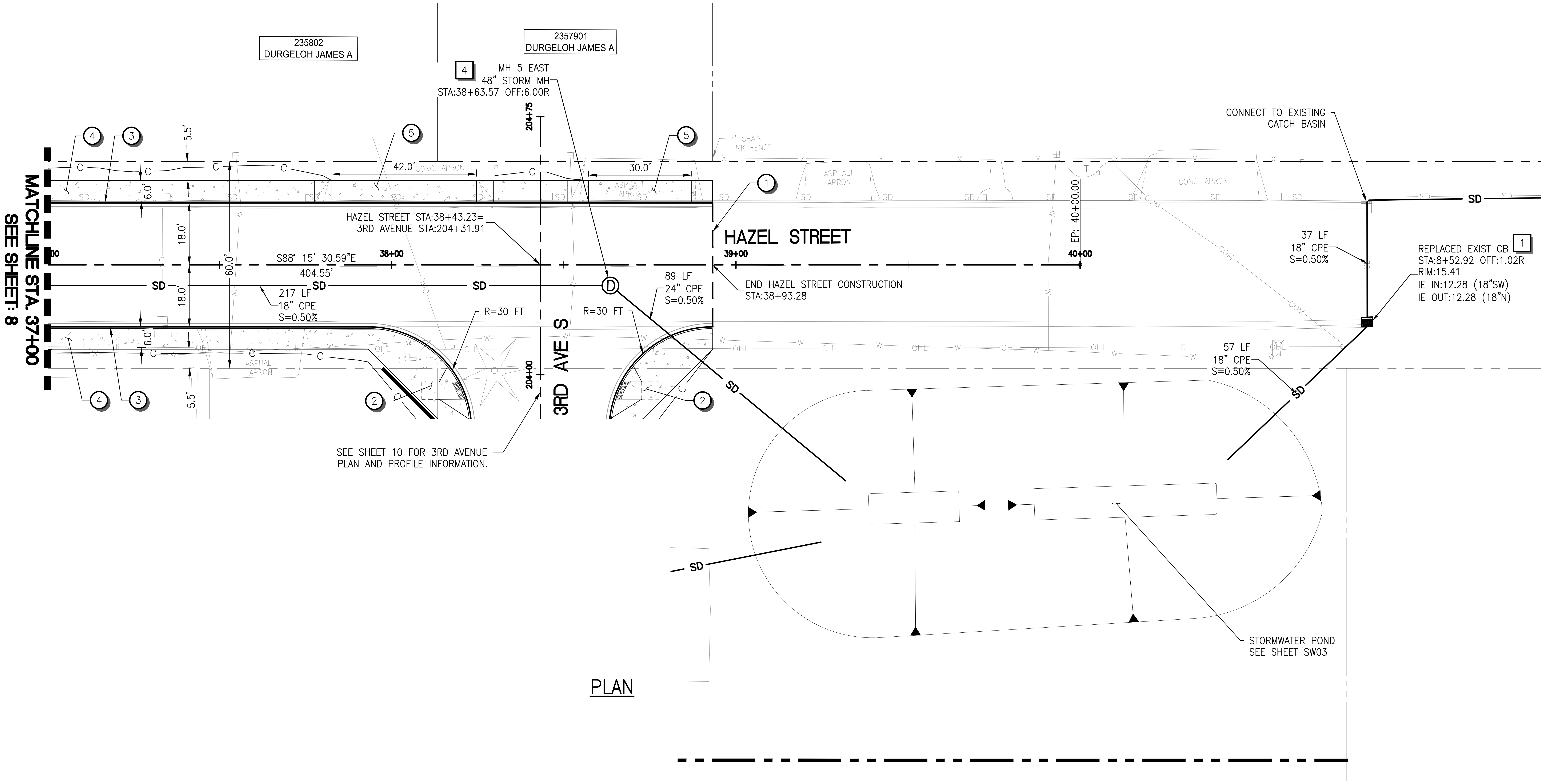
SHEET NUMBER
8

CONSTRUCTION NOTES

1. SAWCUT, TYP
2. INSTALL CURB RAMP C PER CITY OF KELSO STANDARD PLAN ST–130 AND ST–140
3. INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST–110
4. INSTALL SIDEWALK PER CITY OF KELSO STANDARD PLAN ST–150
5. INSTALL DRIVEWAY APPROACH PER CITY OF KELSO STANDARD PLAN ST–160

STORMWATER NOTES

1. INSTALL WSDOT B–25.20–02 COMBINATION INLET TYPE 1
4. INSTALL WSDOT B–15.60–01 MANHOLE TYPE 1



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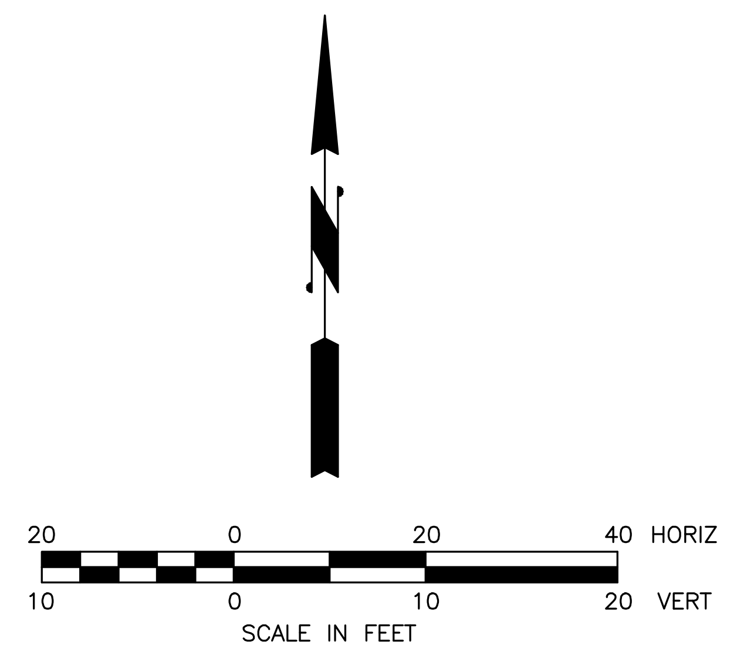

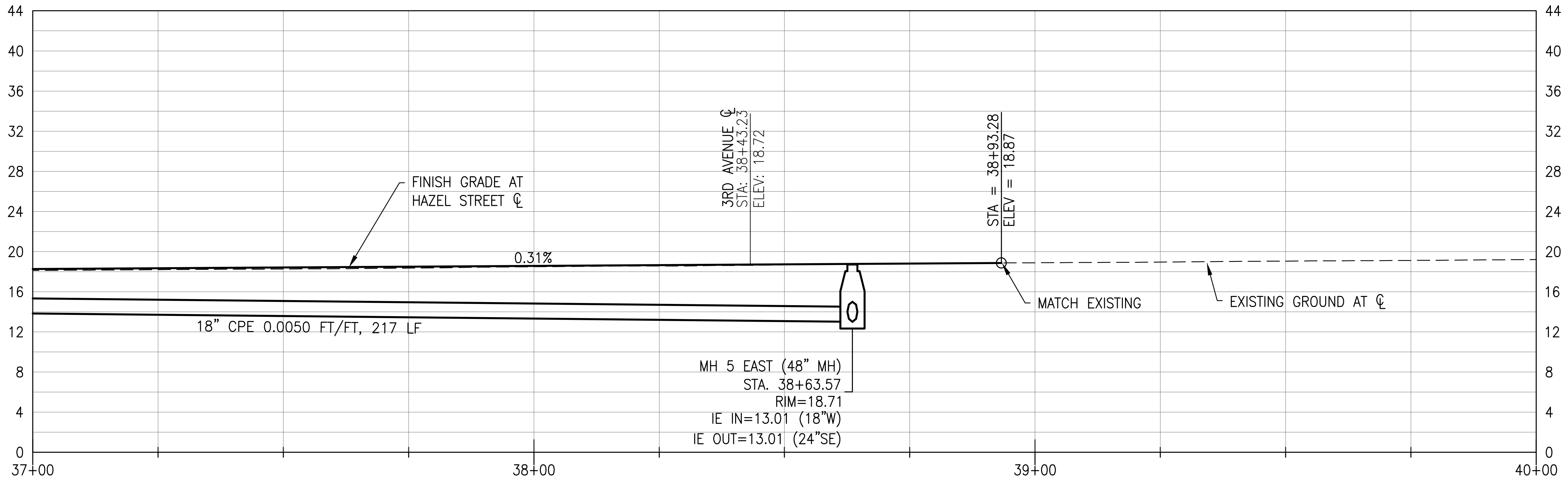

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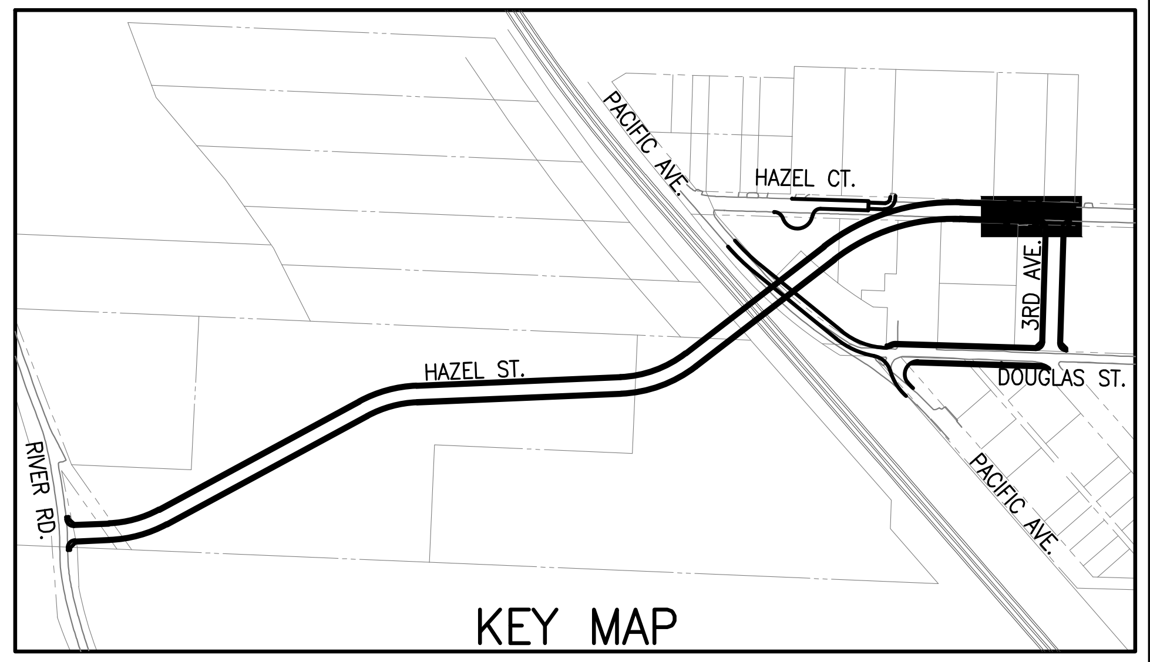

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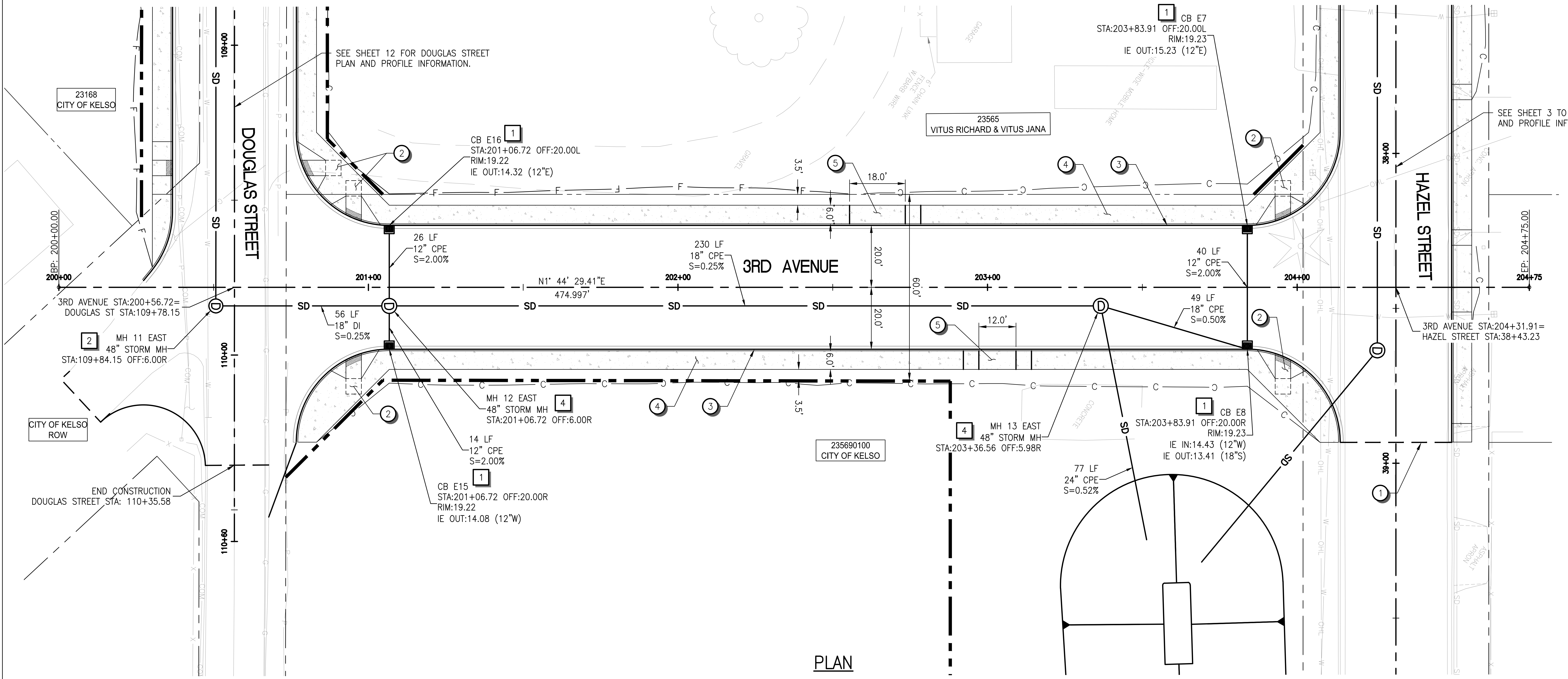
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: C07

SOUTH KELSO RAILROAD CROSSING
HAZEL STREET PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER 9

Plotted: Sep 10, 2018 - 9:46am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913C01.dwg Layout Name: C07



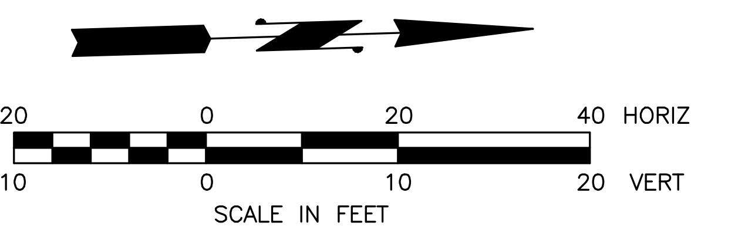
PLAN

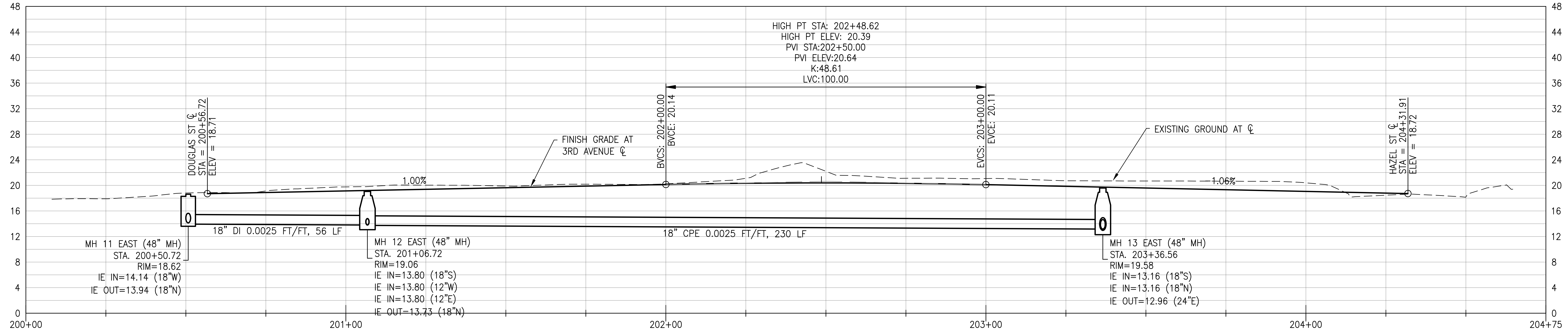
CONSTRUCTION NOTES

1. SAWCUT, TYP
2. INSTALL CURB RAMP C PER CITY OF KELSO STANDARD PLAN ST-130 AND ST-140
3. INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST-110
4. INSTALL SIDEWALK PER CITY OF KELSO STANDARD PLAN ST-150
5. INSTALL DRIVEWAY APPROACH PER CITY OF KELSO STANDARD PLAN ST-160

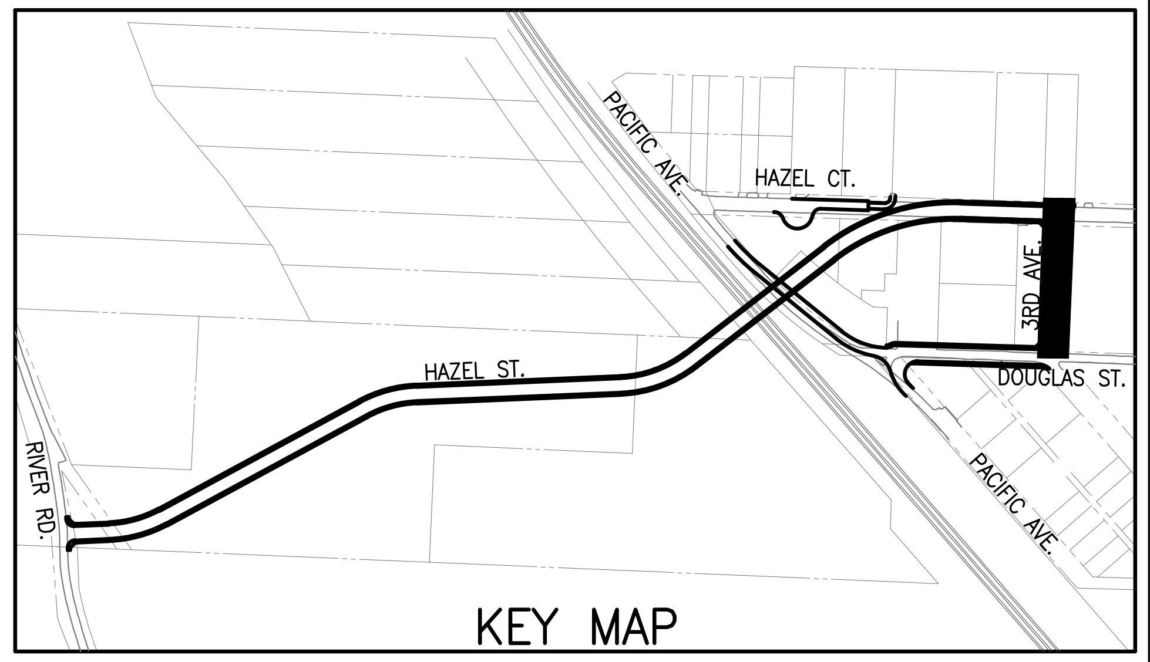
STORMWATER NOTES

1. INSTALL WSDOT B-25.20-02 COMBINATION INLET TYPE 1
2. INSTALL WSDOT B-15.60-02 MANHOLE TYPE 3
4. INSTALL WSDOT B-15.20-01 MANHOLE TYPE 1





PROFILE



KEY MAP

Plotted: Sep 10, 2018 - 10:14am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913C08.dwg Layout Name: 3RD AVE

Otak
700 Washington Street
Suite 300
Vancouver, WA 98660
Phone: (360) 737-9613
FAX: (360) 737-9651
www.otak.com

PRELIMINARY
ALLEN THOMAS HENDY
42636

REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: C08

SOUTH KELSO RAILROAD CROSSING
3RD AVE. SOUTH PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

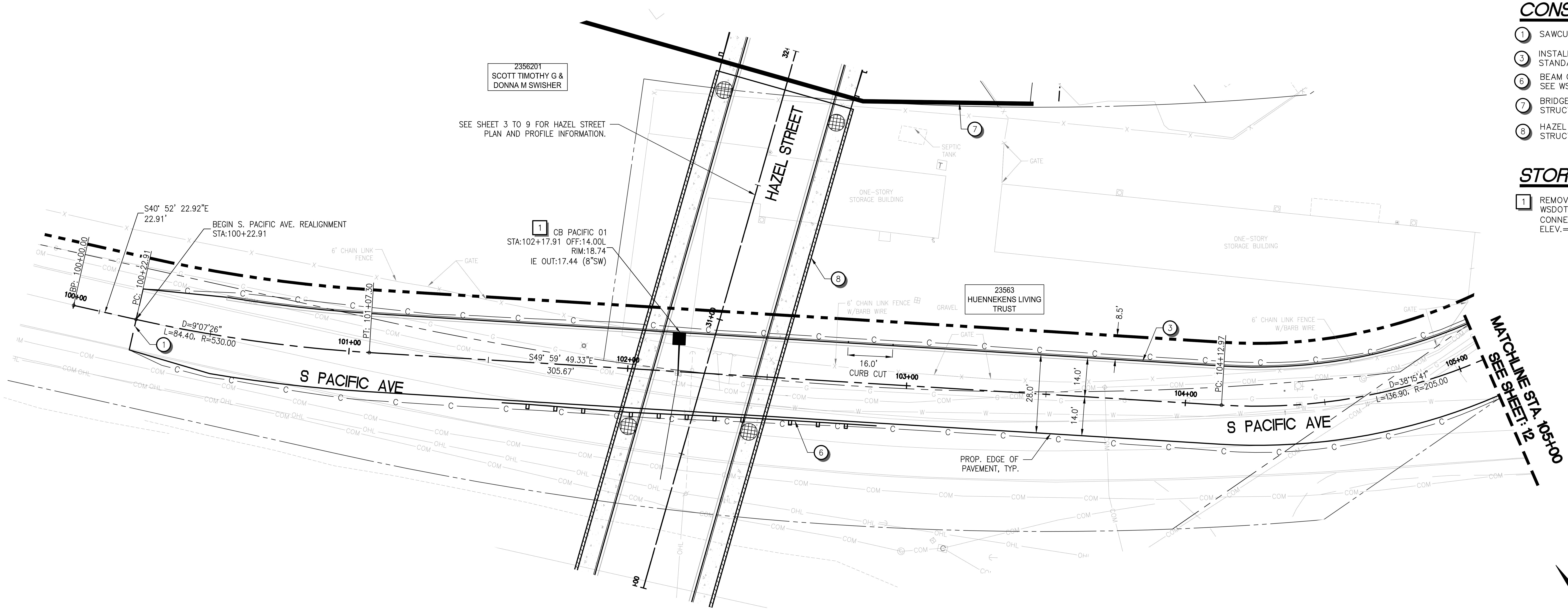
SHEET NUMBER
10

CONSTRUCTION NOTES

1. SAWCUT, TYP
3. INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST-110
6. BEAM GUARDRAIL TYPE 31 SEE WSDOT DETAIL C-20.10-04 & C-22.45-03
7. BRIDGE APPROACH ABUTMENT. SEE STRUCTURAL SHEETS FOR DETAILS.
8. HAZEL STREET BRIDGE CROSSING. SEE STRUCTURAL SHEETS

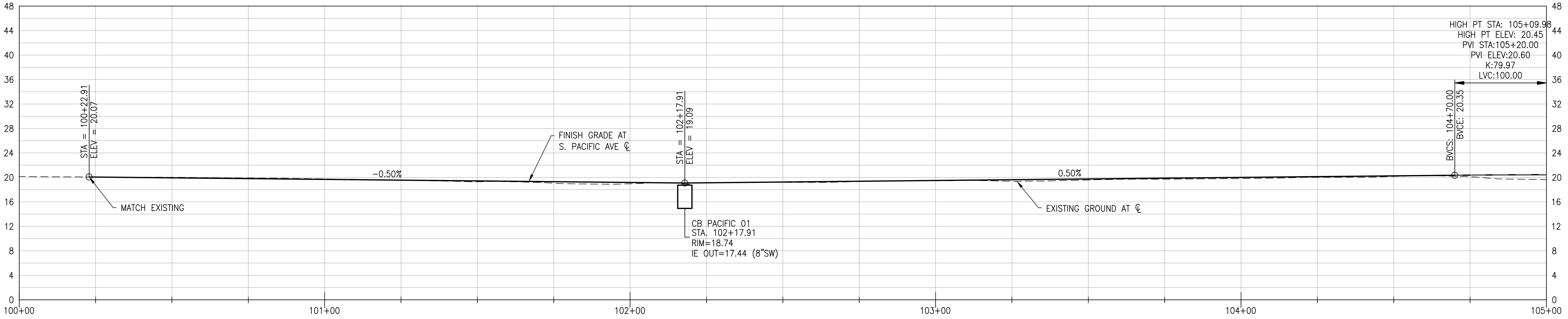
STORMWATER NOTES

1. REMOVE EX. CATCH BASIN AND INSTALL WSDOT 25.20-02 COMBINATION INLET TYPE 1. CONNECT TO EXISTING 8" PIPE AT ELEV.=17.17.

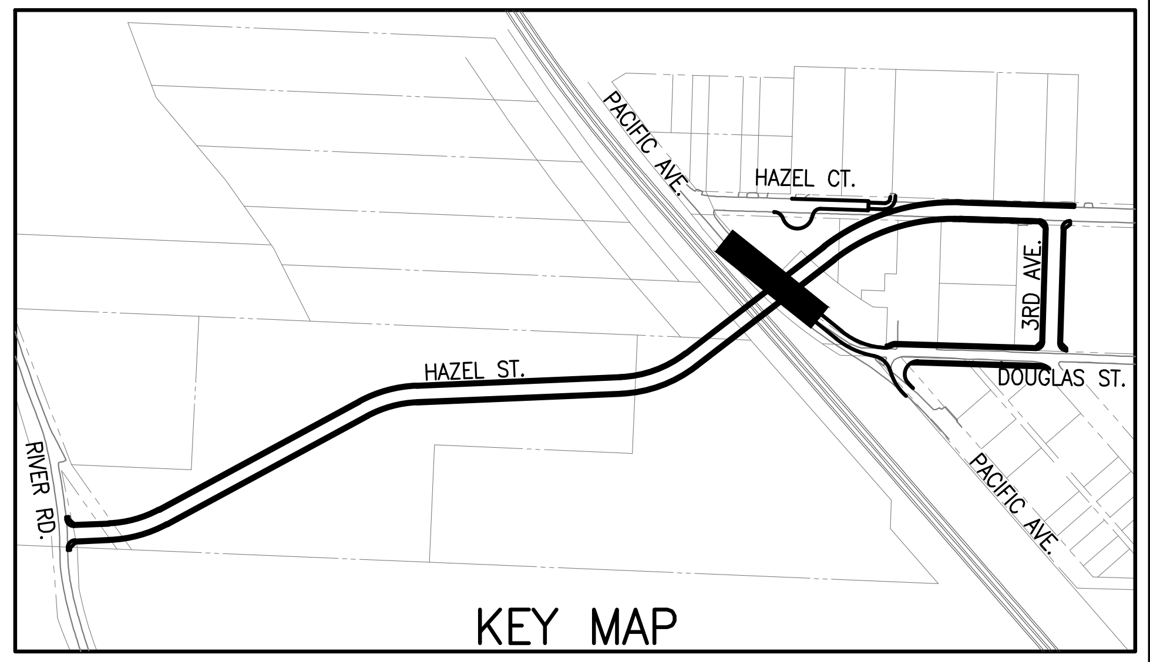


PLAN

20 0 20 40 HORIZ
10 0 10 20 VERT
SCALE IN FEET



PROFILE



KEY MAP

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PRELIMINARY
ALLEN THOMAS HENDY
42636

REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: C09

SOUTH KELSO RAILROAD CROSSING
S. PACIFIC AVE. PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
11

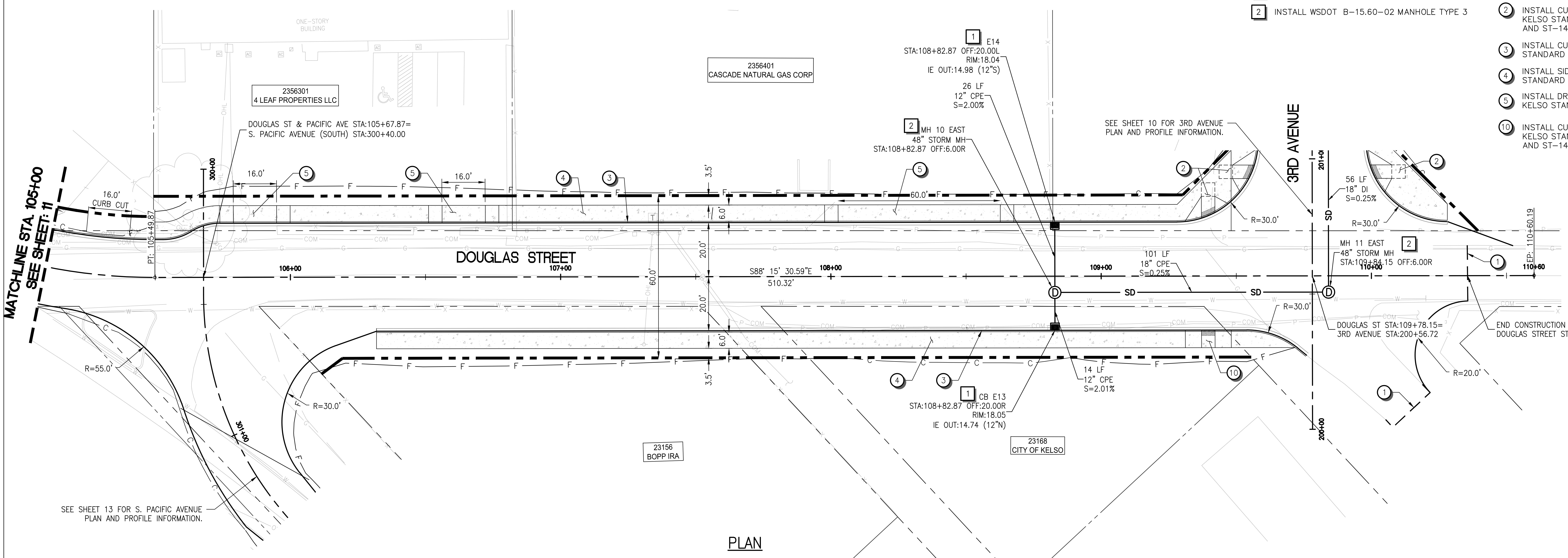
Plotted: Sep 10, 2018 - 9:47am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913C08.dwg Layout Name: PACIFIC

STORMWATER NOTES

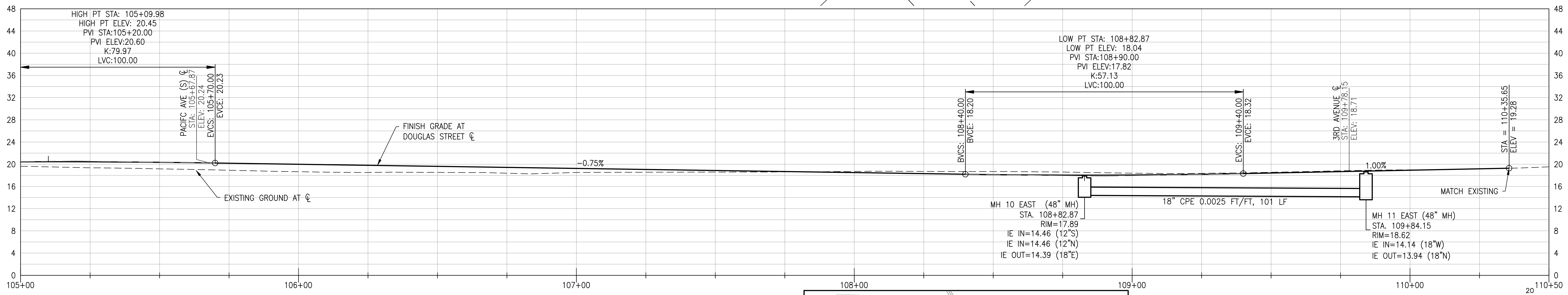
- [1] INSTALL WSDOT B-25.20-02 COMBINATION INLET TYPE 1
- [2] INSTALL WSDOT B-15.60-02 MANHOLE TYPE 3

CONSTRUCTION NOTES

- ① SAWCUT, TYP
- ② INSTALL CURB RAMP C PER CITY OF KELSO STANDARD PLAN ST-130 AND ST-140
- ③ INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST-110
- ④ INSTALL SIDEWALK PER CITY OF KELSO STANDARD PLAN ST-150
- ⑤ INSTALL DRIVEWAY APPROACH PER CITY OF KELSO STANDARD PLAN ST-160
- ⑩ INSTALL CURB RAMP D PER CITY OF KELSO STANDARD PLAN ST-130 AND ST-140

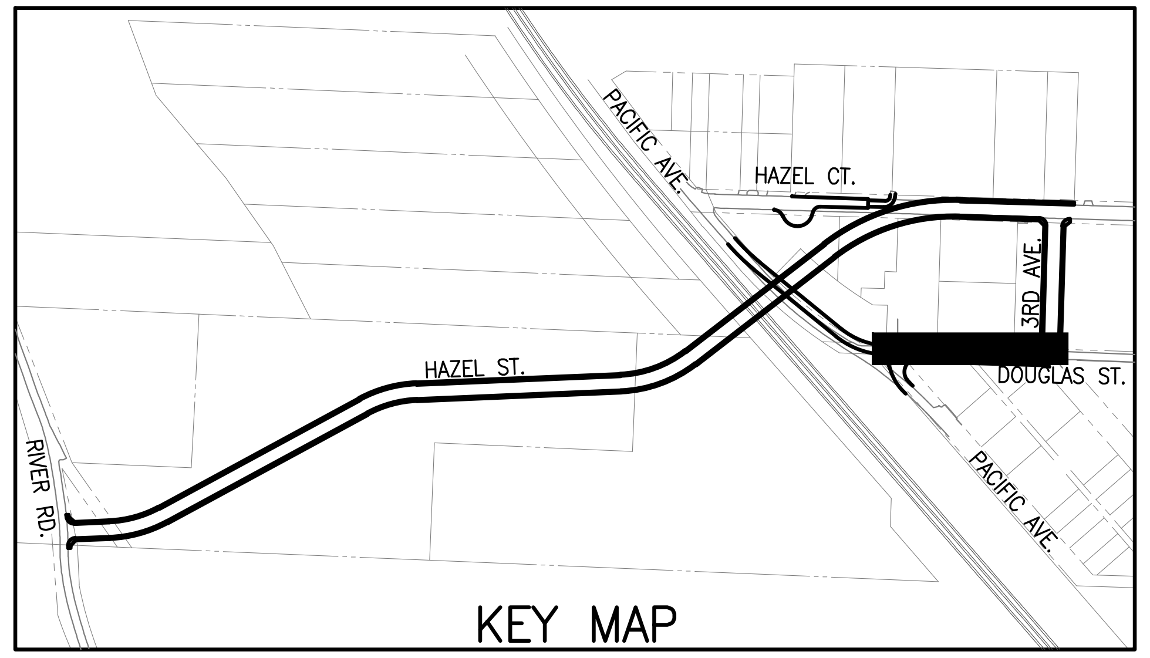


PLAN



PROFILE

20 0 20 40 HORIZ
10 0 10 20 VERT
SCALE IN FEET



KEY MAP

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PRELIMINARY
ALLEN THOMAS HENDY
42636

REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: C10

SOUTH KELSO RAILROAD CROSSING
DOUGLAS STREET PLAN AND PROFILE

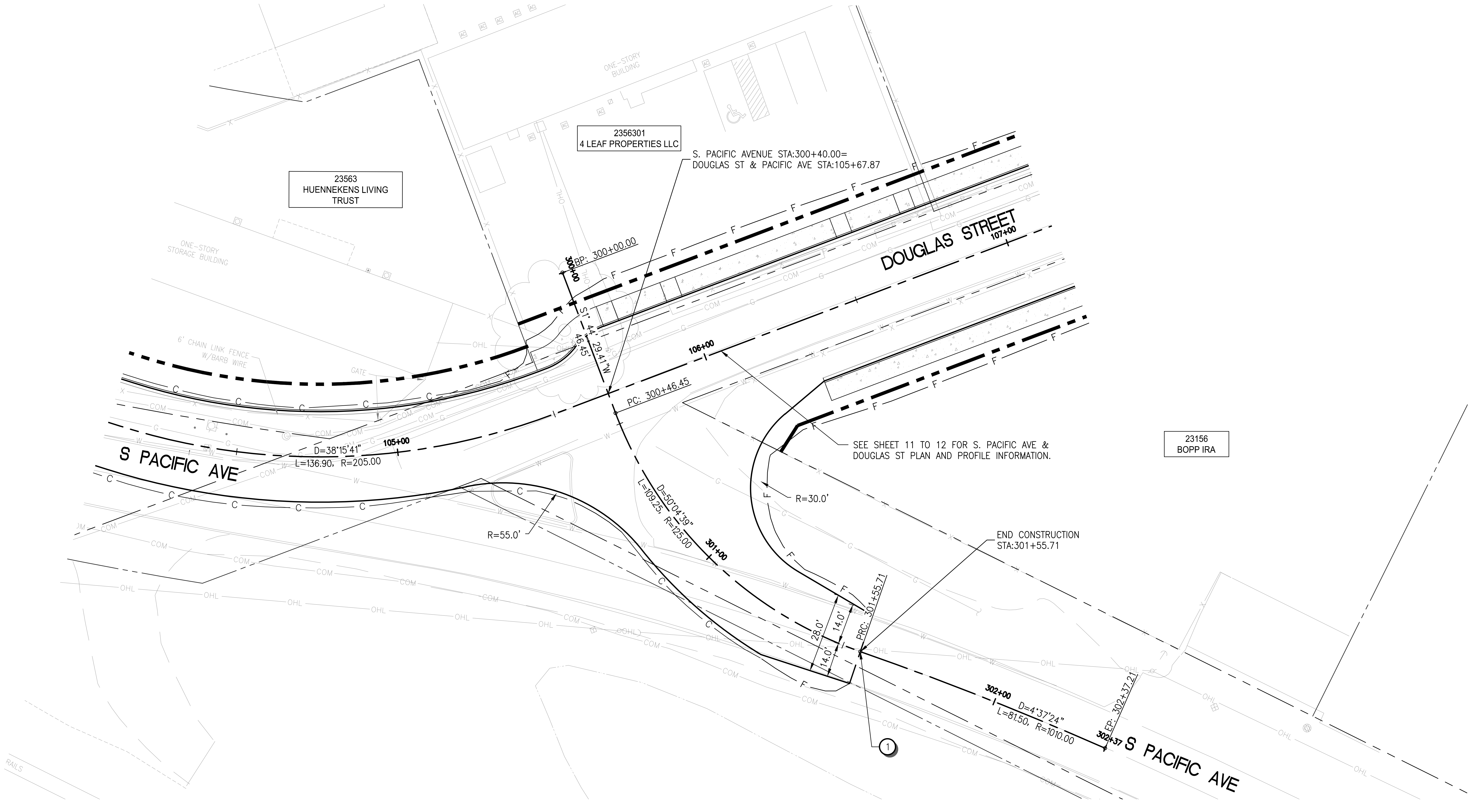
CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
12

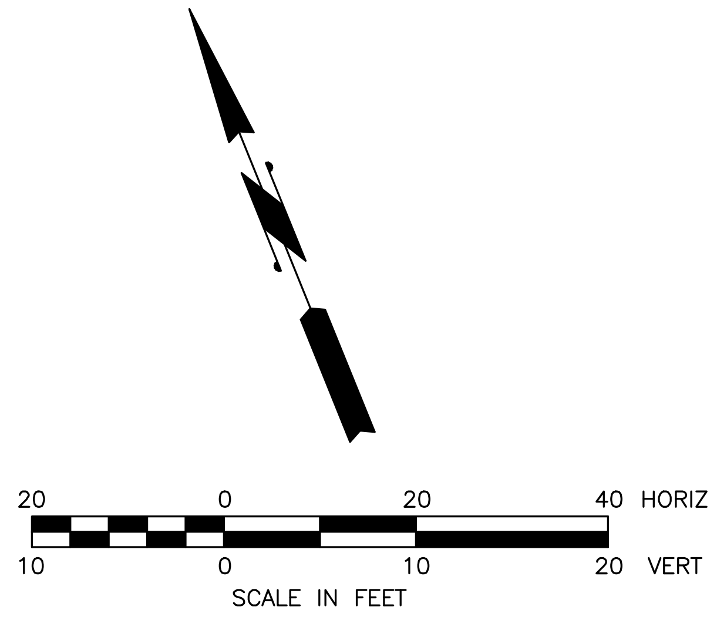
Plotted: Sep 10, 2018 - 9:47am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913C08.dwg Layout Name: DOUGLAS

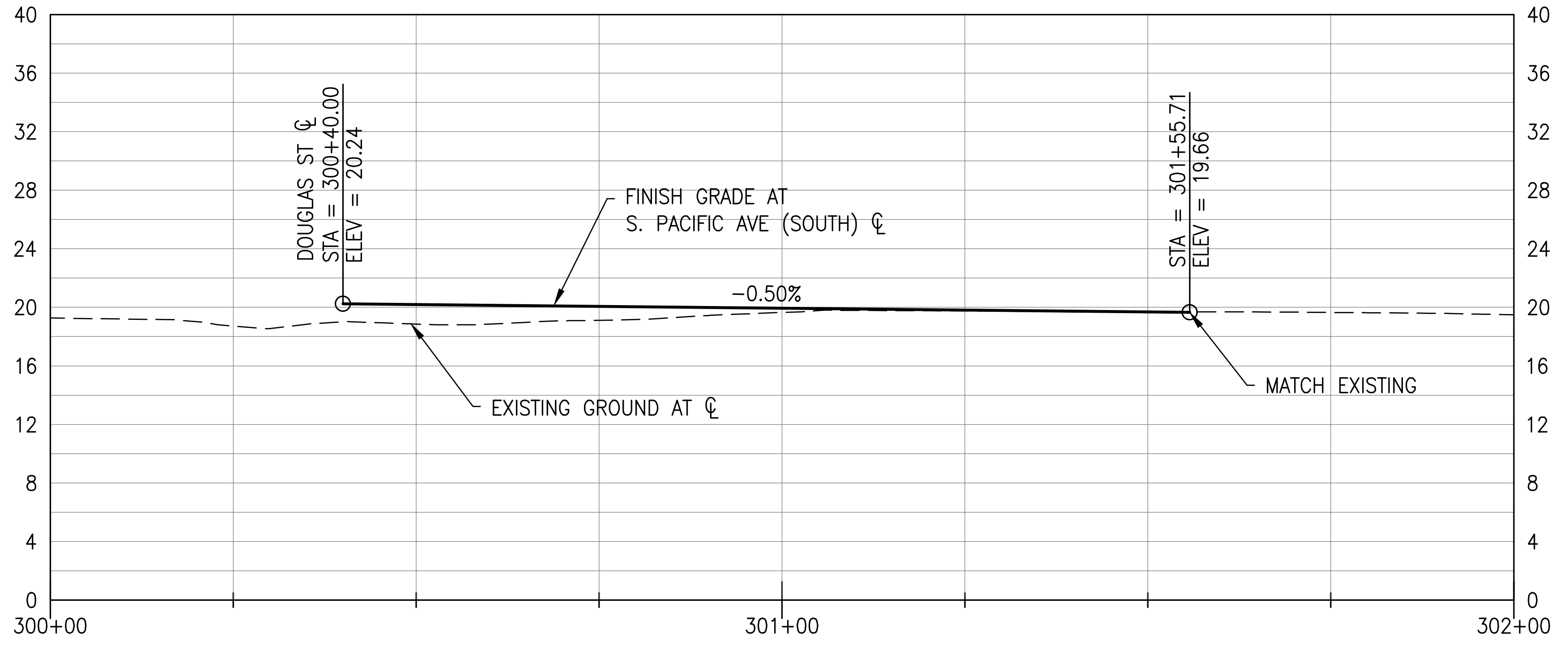
CONSTRUCTION NOTES

① SAWCUT, TYP

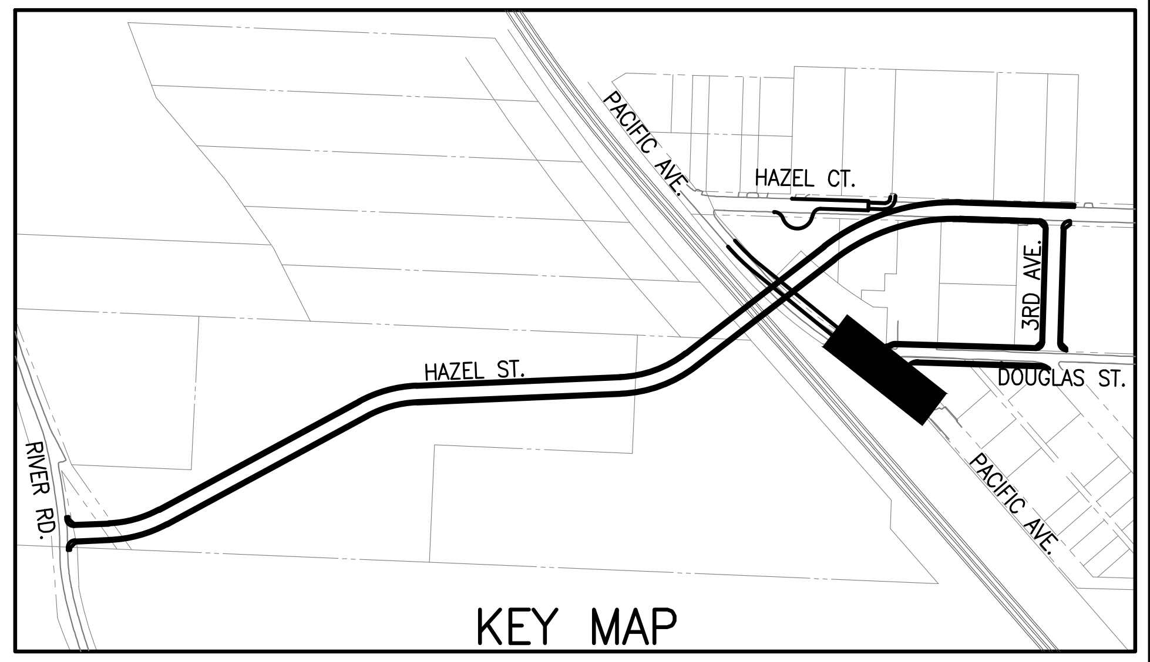


PLAN





PROFILE



KEY MAP

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ALLEN THOMAS HENDY
42636

REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: C11

SOUTH KELSO RAILROAD CROSSING
S. PACIFIC AVE. (SOUTH OF DOUGLAS ST.) PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
13

Plotted: Sep 10, 2018 - 9:48am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913C08.dwg Layout Name: PACIFIC 02

PLAN

CONSTRUCTION NOTES

① SAWCUT, TYP.

③ INSTALL CURB & GUTTER PER CITY OF KELSO STANDARD PLAN ST-110.

⑨ INSTALL RETAINING WALL, SEE STRUCTURAL SHEETS FOR DETAILS.

⑪ INSTALL CONCRETE CURB PER CITY OF KELSO STANDARD PLAN ST-090.

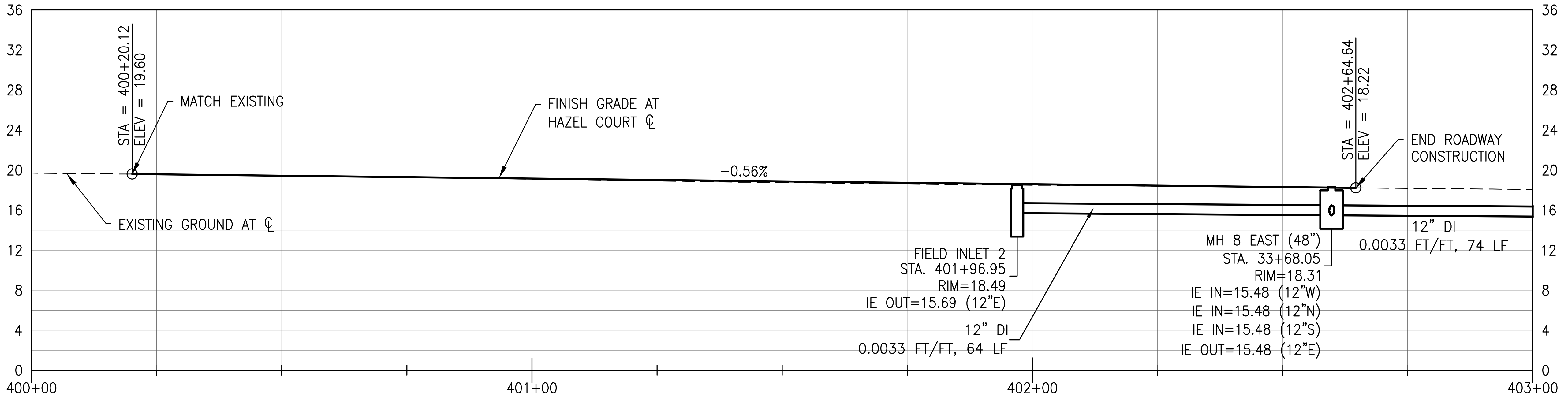
⑫ INSTALL CONCRETE VALLEY GUTTER.

STORMWATER NOTES

1 INSTALL WSDOT B-25.20-02 COMBINATION INLET TYPE 1

2 INSTALL WSDOT B-15.60-02 MANHOLE TYPE 3

6 INSTALL FIELD INLET



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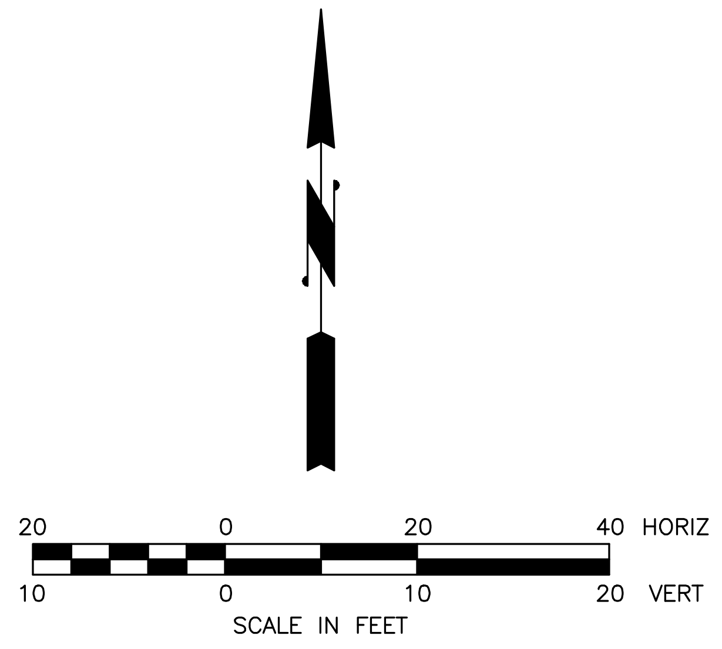

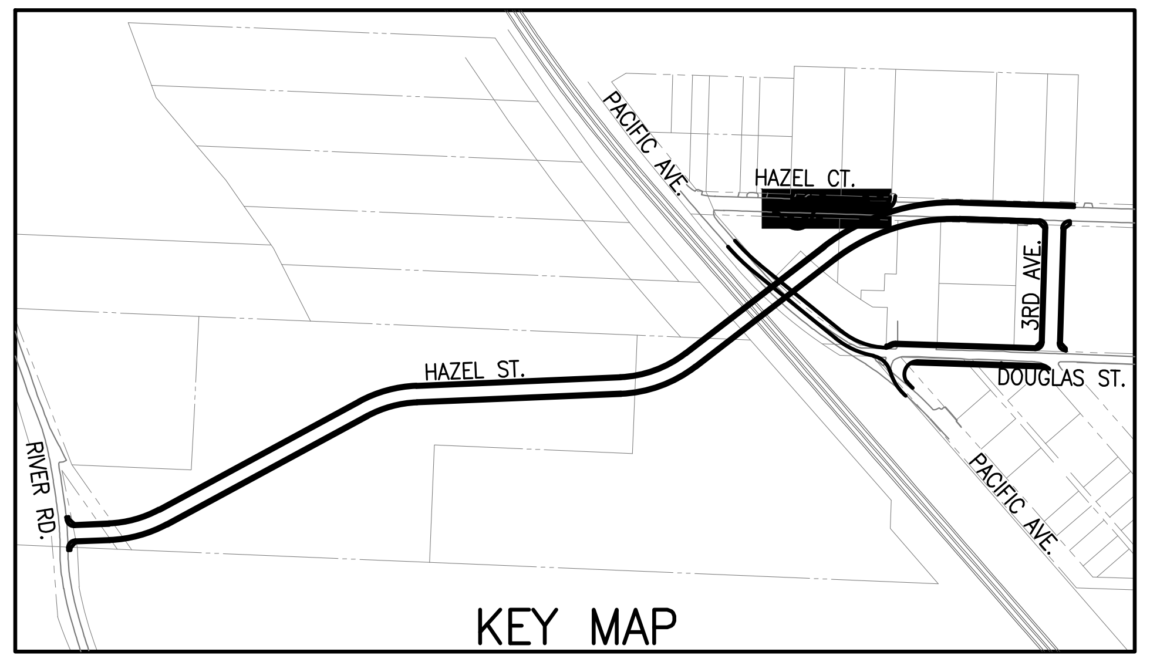

KEY MAP

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ALLEN THOMAS HENDY
42636
PRELIMINARY

REVISION	DATE	DESCRIPTION	BY

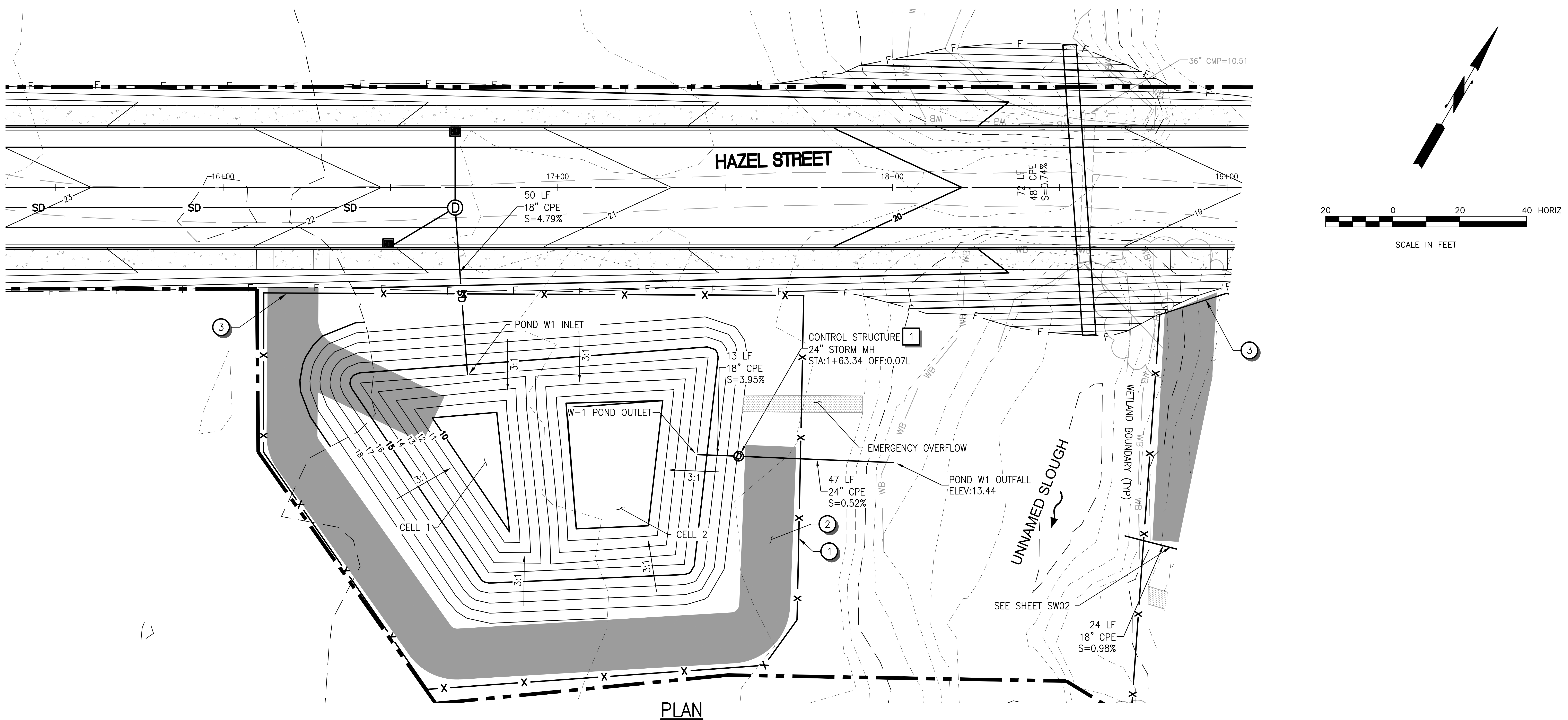
JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: C12

SOUTH KELSO RAILROAD CROSSING
HAZEL COURT PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
14

Plotted: Sep 10, 2018 - 9:49am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913C01.dwg Layout Name: C12



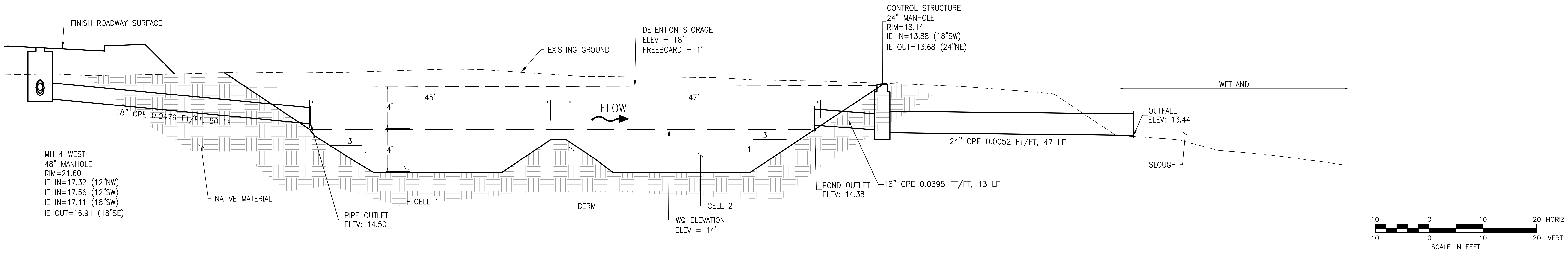
PLAN

CONSTRUCTION NOTES

1. INSTALL 6' CHAIN LINK FENCE
2. INSTALL 15' ACCESS ROAD
3. INSTALL GATE

STORMWATER NOTES

1. FLOW CONTROL STRUCTURE



COMBINATION WATER QUALITY AND FLOW CONTROL FACILITY
TYPICAL SECTION

Plotted: Sep 10, 2018 - 9:49am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913_SW01.dwg Layout Name: SW01

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PRELIMINARY
ALLEN THOMAS HENDY 42636

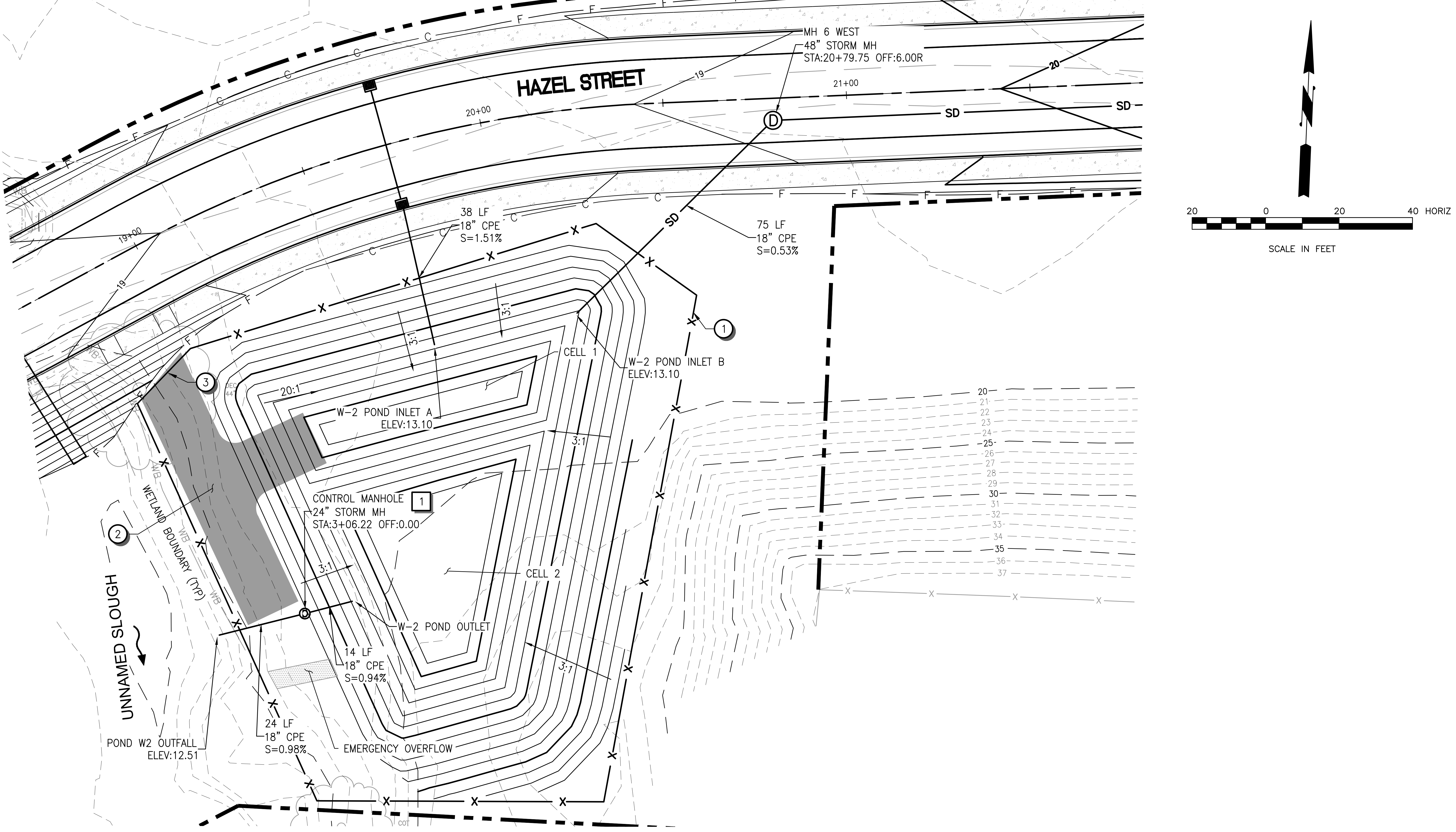
REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: SW01

SOUTH KELSO RAILROAD CROSSING
WEST POND #1 PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
15



PLAN

CONSTRUCTION NOTES

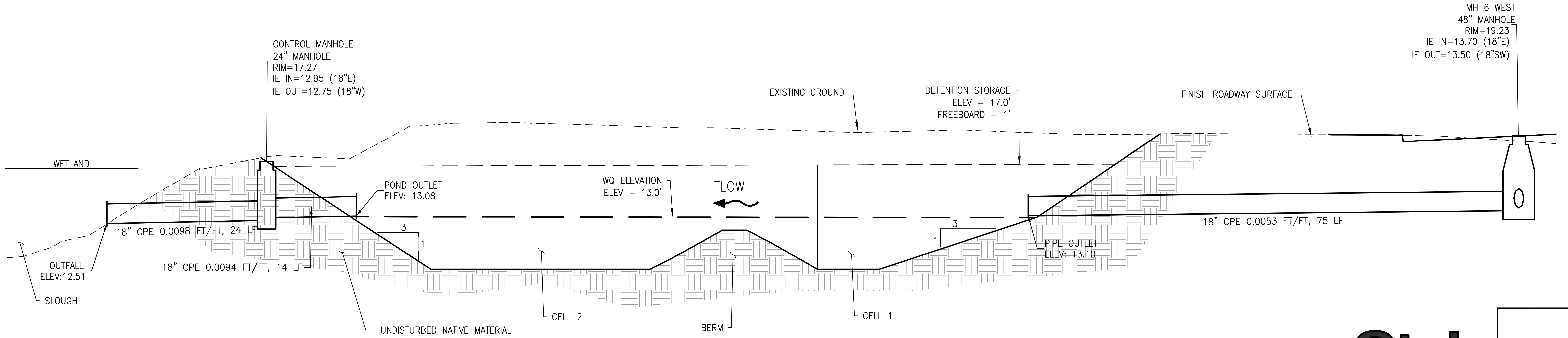
① INSTALL 6' CHAIN LINK FENCE

② INSTALL 15' ACCESS ROAD

② INSTALL GATE

STORMWATER NOTES

1 FLOW CONTROL STRUCTURE



10 0 10 20 HORIZ
10 0 10 20 VERT
SCALE IN FEET

COMBINATION WATER QUALITY AND FLOW CONTROL FACILITY
TYPICAL SECTION

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ALLEN THOMAS HENDY
42636
PRELIMINARY

REVISION	DATE	DESCRIPTION	BY

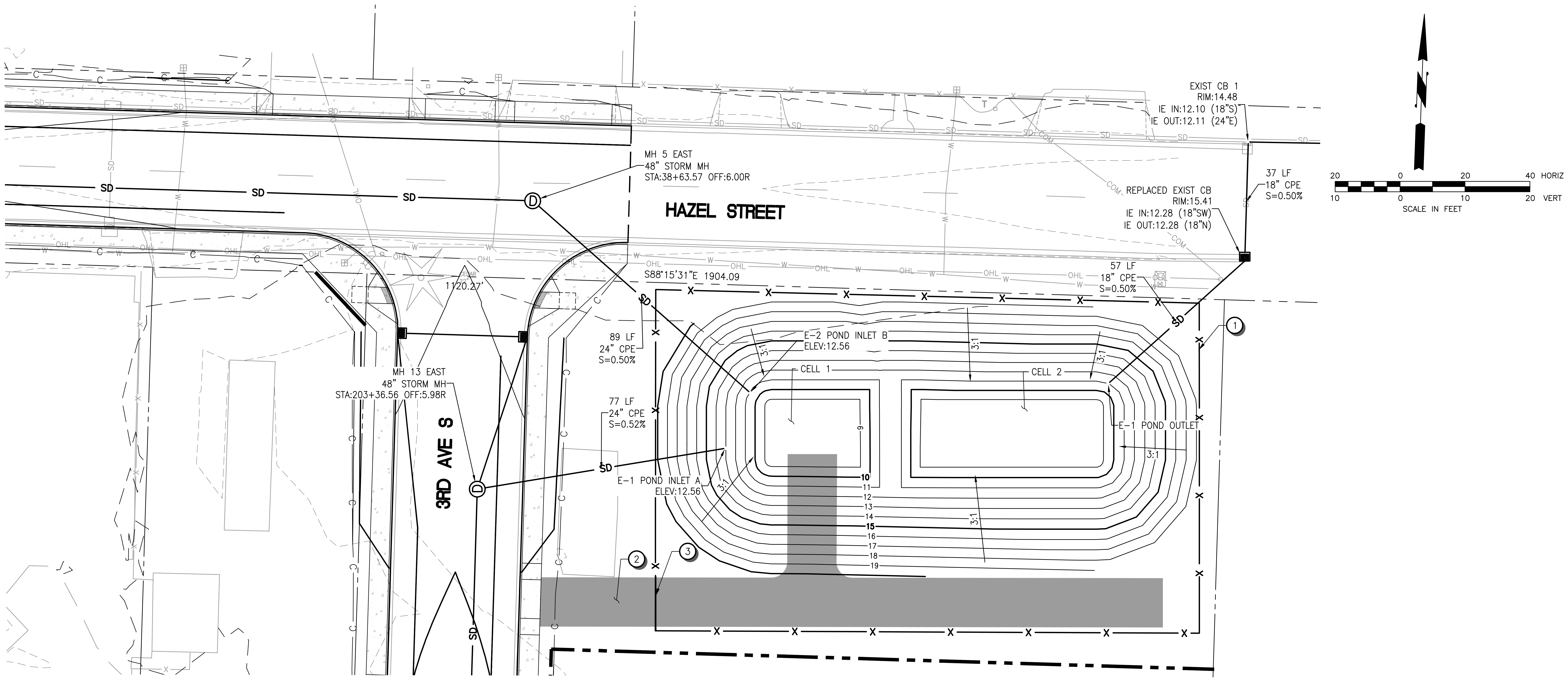
JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: SW02

SOUTH KELSO RAILROAD CROSSING
WEST POND #2 PLAN AND PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
16

Plotted: Sep 10, 2018 - 9:50am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913_SW01.dwg Layout Name: SW02



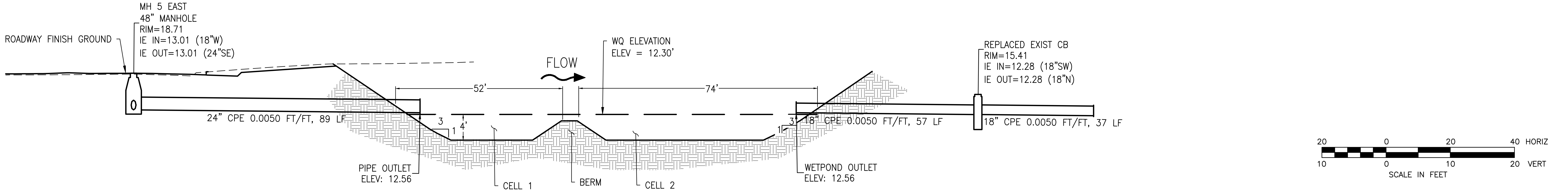
PLAN

CONSTRUCTION NOTES

① INSTALL 6' CHAIN LINK FENCE

② INSTALL 15' ACCESS ROAD

③ INSTALL GATE



PROFILE

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PRELIMINARY
ALLEN THOMAS HENDY
42636

REVISION	DATE	DESCRIPTION	BY

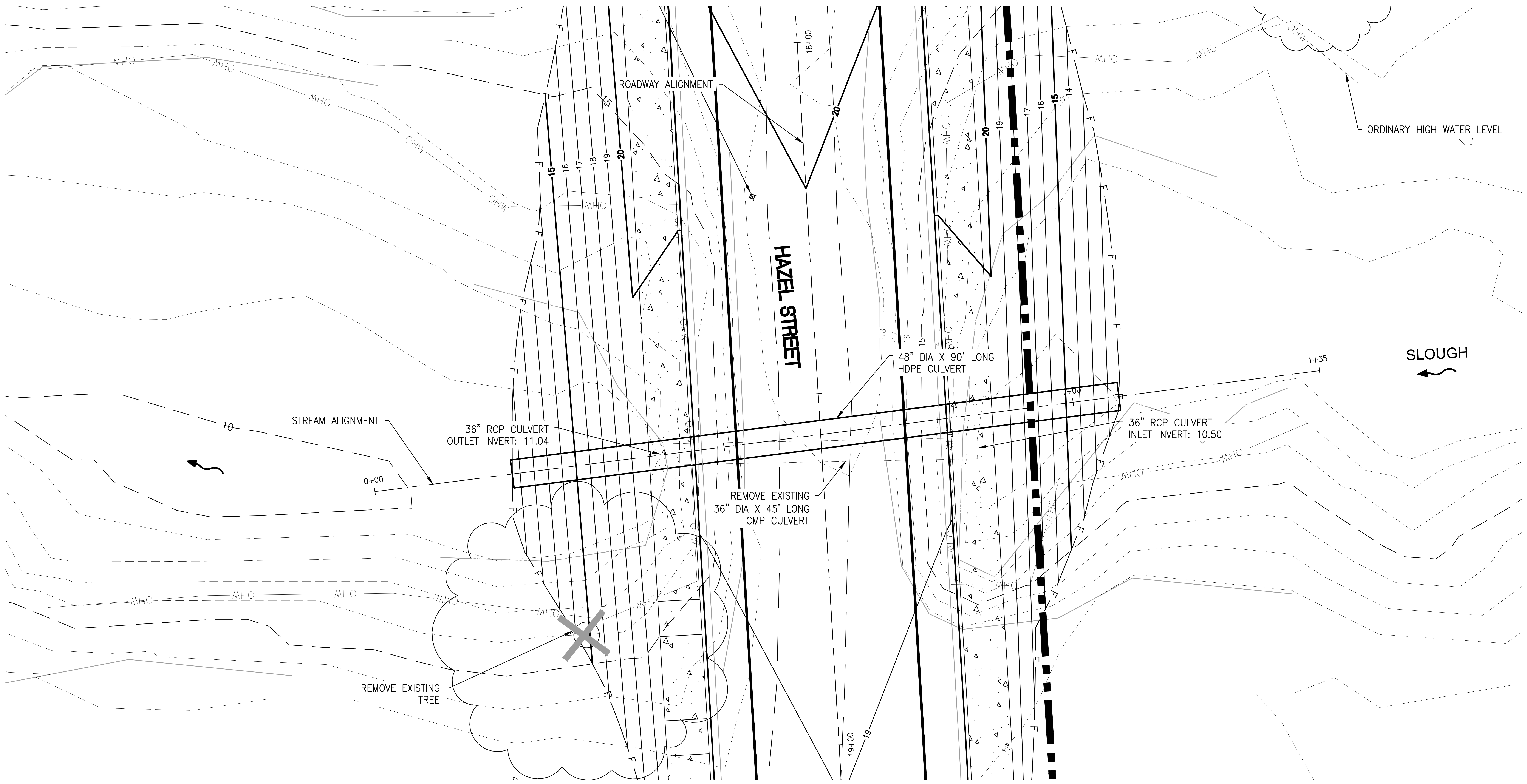
JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: SW03

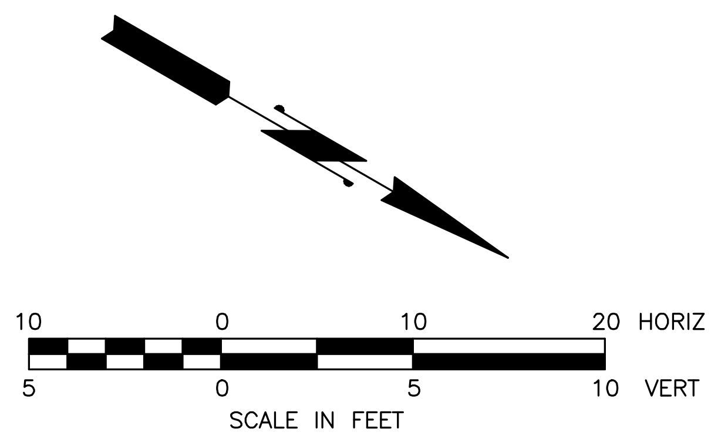
SOUTH KELSO RAILROAD CROSSING
EAST POND PLAN PROFILE

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

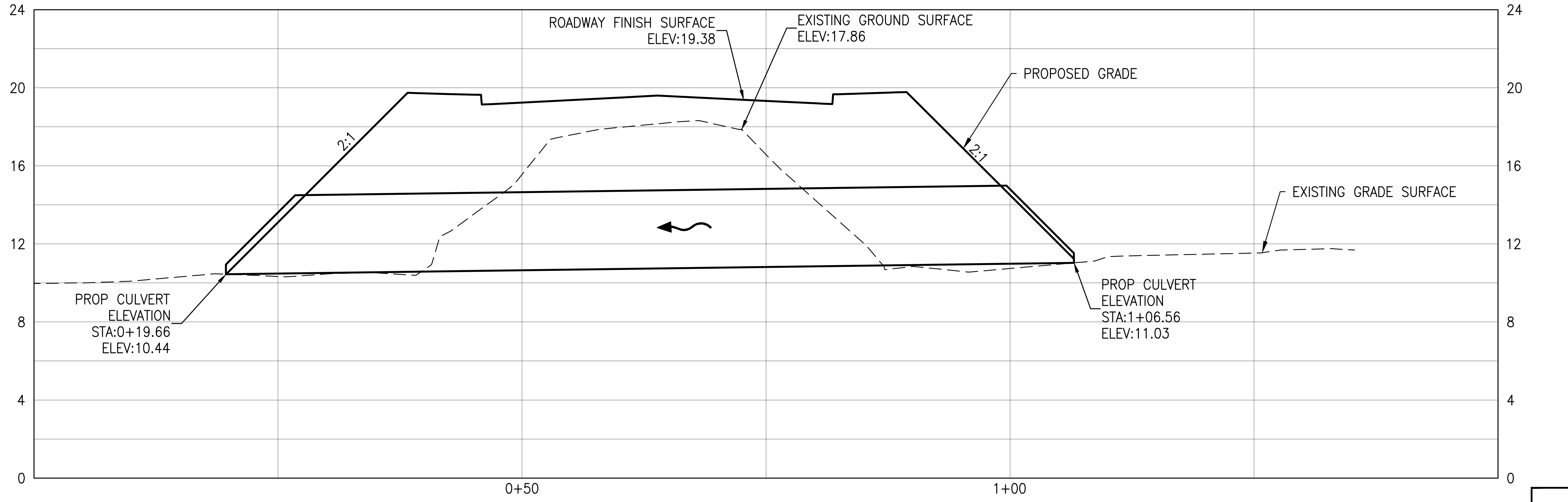
SHEET NUMBER
17

Plotted: Sep 10, 2018 - 9:50am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913_SW01.dwg Layout Name: SW03





PLAN



STREAM ALIGNMENT
HORIZ SCALE= 1:10
VERT SCALE= 1:5

PROFILE

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PRELIMINARY

ALLEN THOMAS HENDY
42636

REVISION	DATE	DESCRIPTION	BY

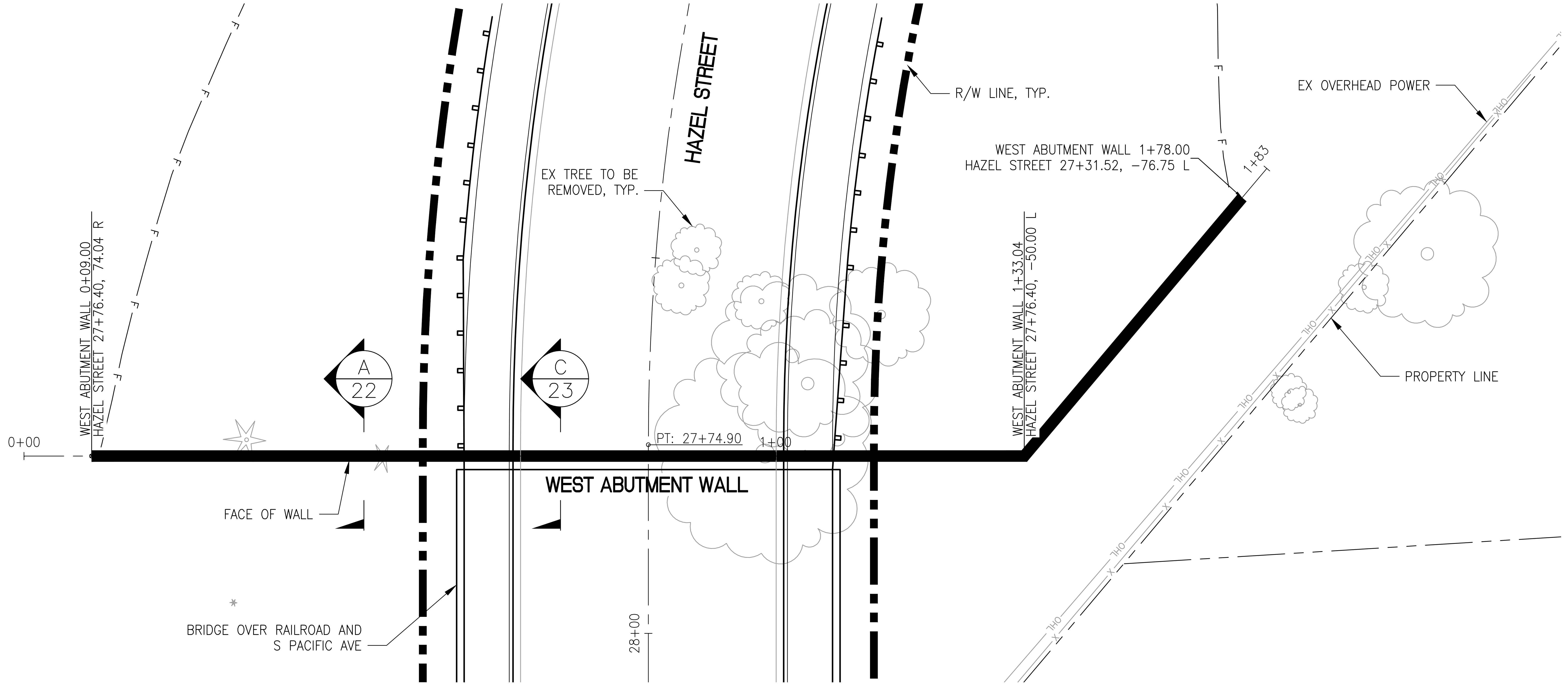
JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 09/2018
SCALE: AS NOTED
DWG NO.: SW04

SOUTH KELSO RAILROAD CROSSING
SLOUGH CROSSING PLAN AND PROFILE

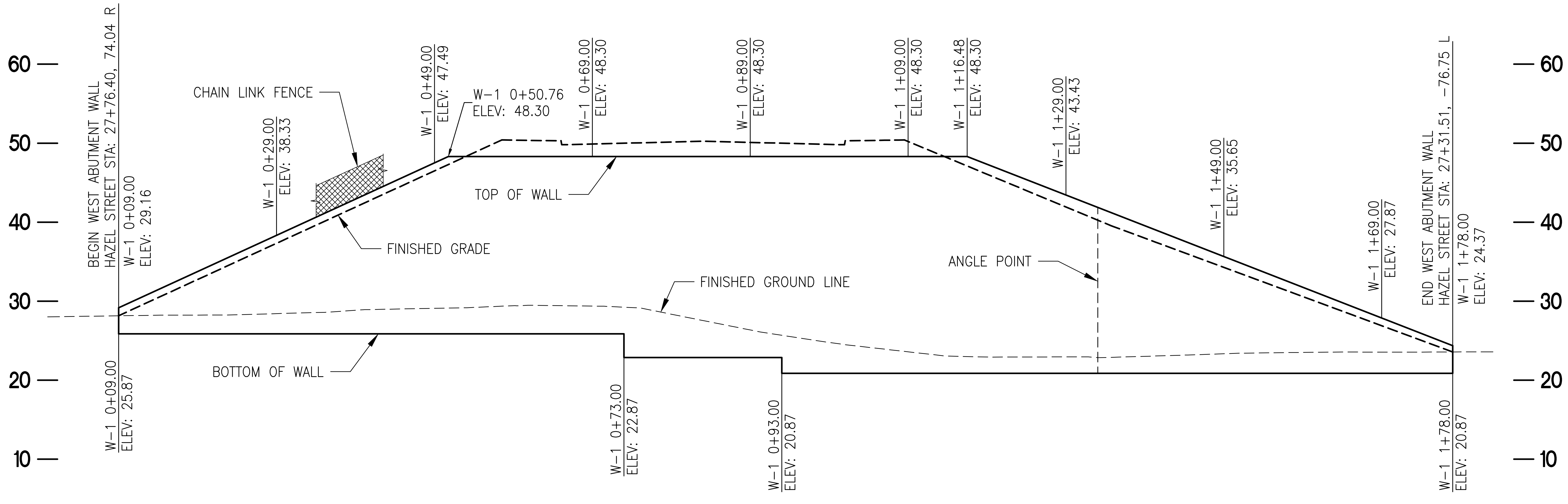
CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
18

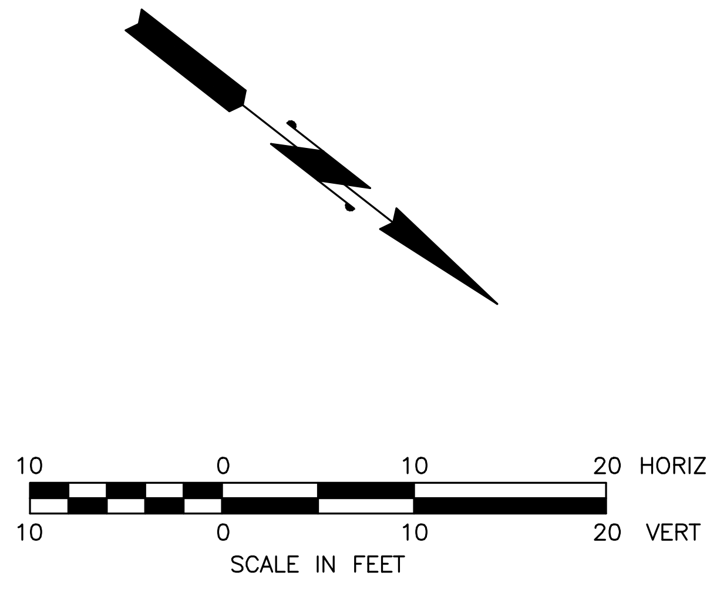
Plotted: Sep 10, 2018 - 9:50am V:\PROJECT\17900\17913\CADD\ACAD\Xref\C3D\S17913_CULVERT.dwg Layout Name: SW04



WEST ABUTMENT WALL PLAN (WALL 1)
SCALE: 1"=10'-0"



WEST ABUTMENT WALL DEVELOPED ELEVATION (WALL 1)
SCALE: 1"=10'-0"



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Phone: (360) 737-9613
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DOUGLAS L. SARKKINEN
44136
PRELIMINARY

REVISION	DATE	DESCRIPTION	BY

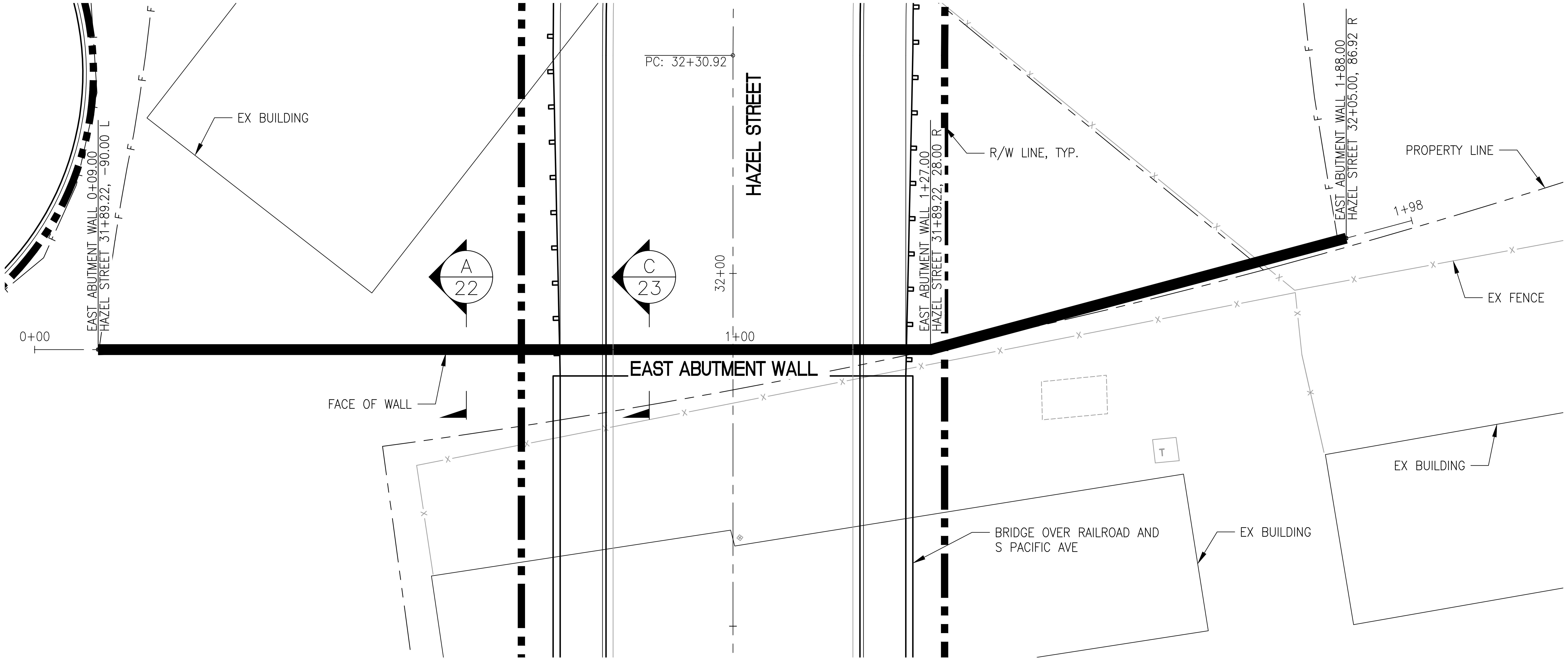
JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 08/2018
SCALE: AS NOTED
DWG NO.: W01

SOUTH KELSO RAILROAD CROSSING
WEST ABUTMENT WALL PLAN AND ELEVATION

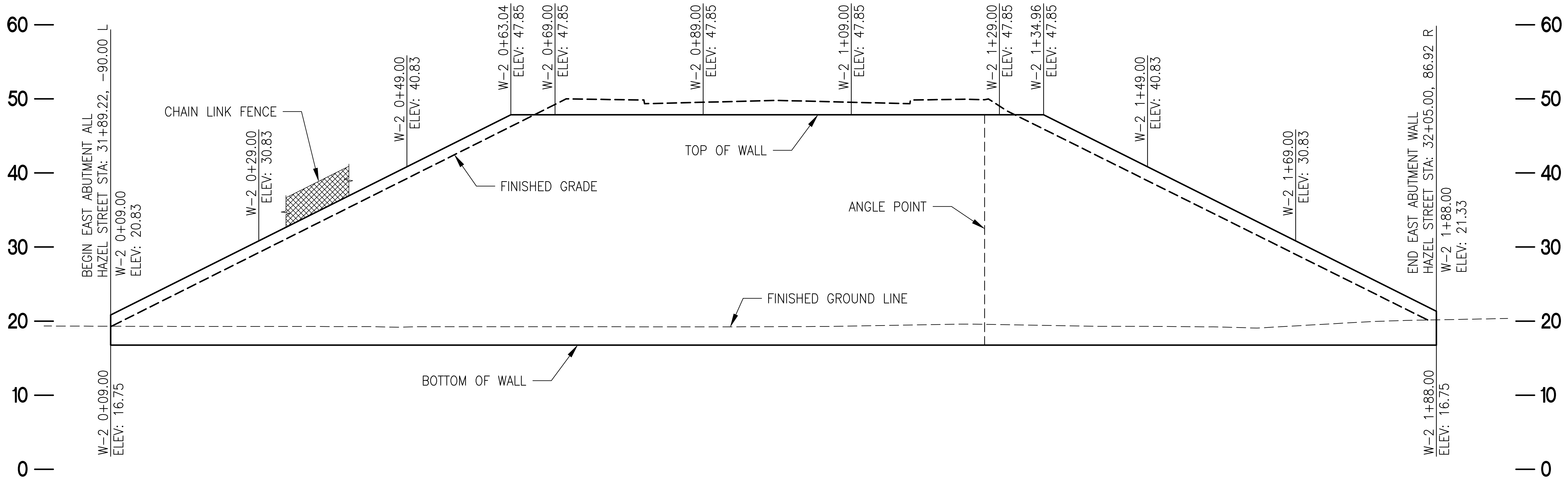
CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
19

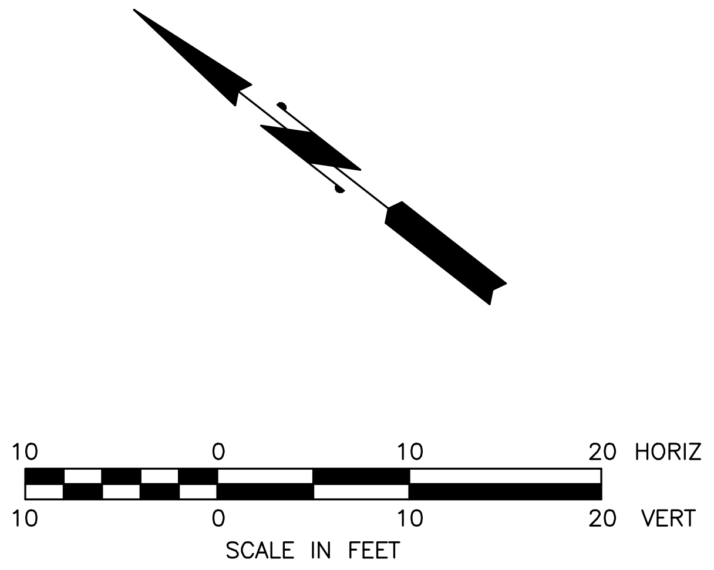
Plotted: Sep 07, 2018 - 11:27am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913W01.dwg Layout Name: W01



EAST ABUTMENT PLAN (WALL 2)
SCALE: 1"=10'-0"



EAST ABUTMENT DEVELOPED ELEVATION (WALL 2)
SCALE: 1"=10'-0"



Plotted: Sep 07, 2018 - 11:27am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913W01.dwg Layout Name: W02

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DOUGLAS L. SARKKINEN
24136
PRELIMINARY

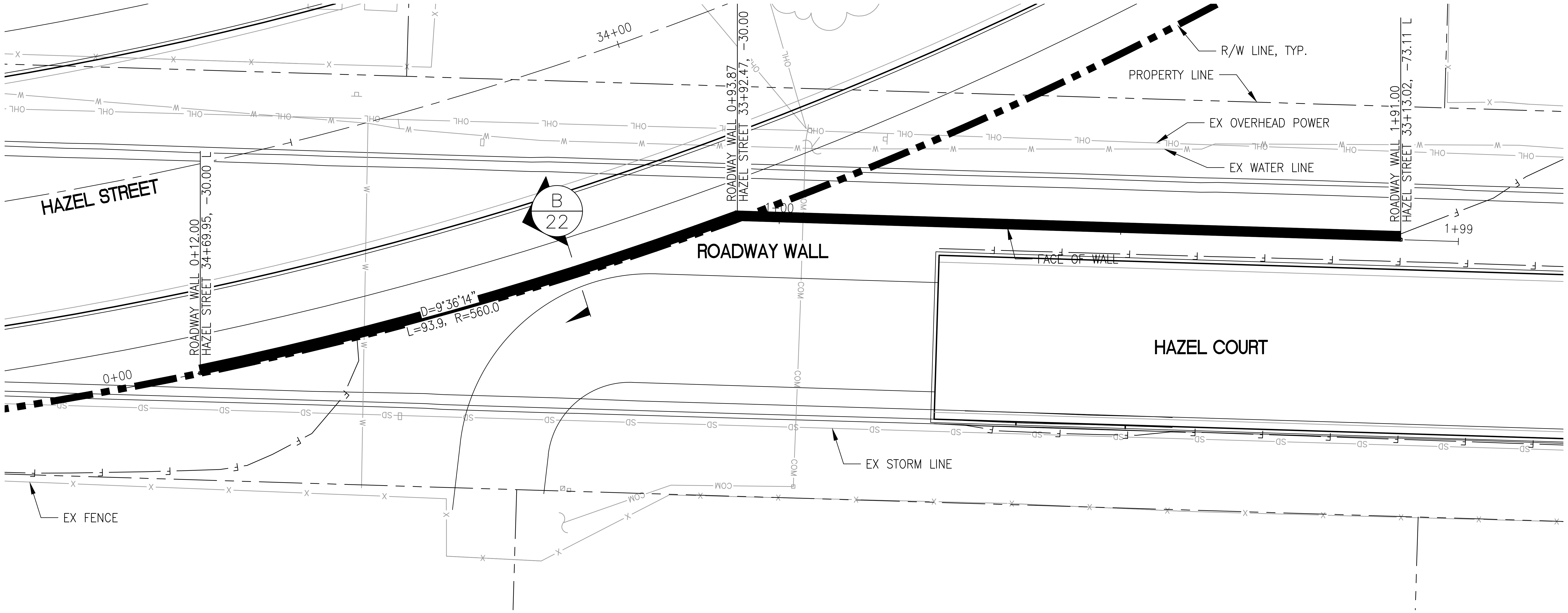
REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 08/2018
SCALE: AS NOTED
DWG NO.: W02

SOUTH KELSO RAILROAD CROSSING
EAST ABUTMENT WALL PLAN AND ELEVATION

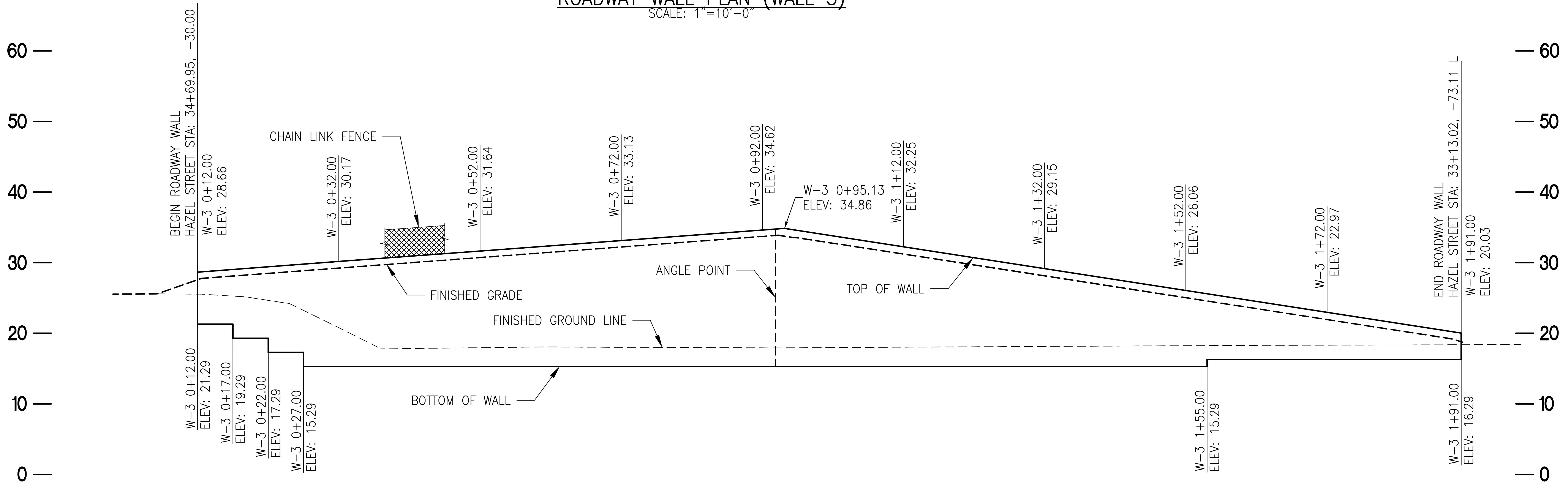
CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
20



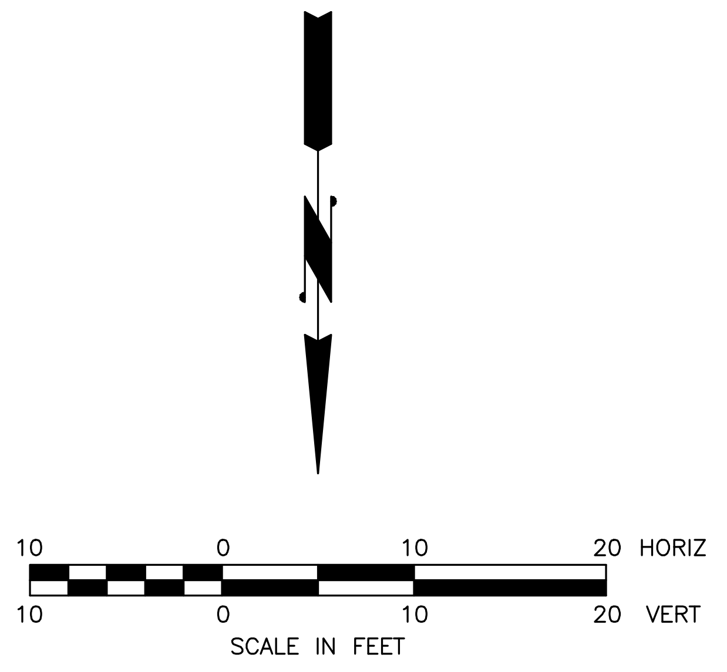
ROADWAY WALL PLAN (WALL 3)

SCALE: 1"=10'-0"



ROADWAY WALL DEVELOPED ELEVATION (WALL 3)

SCALE: 1"=10'-0"



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DOUGLAS L. SARKKINEN
24136
PRELIMINARY

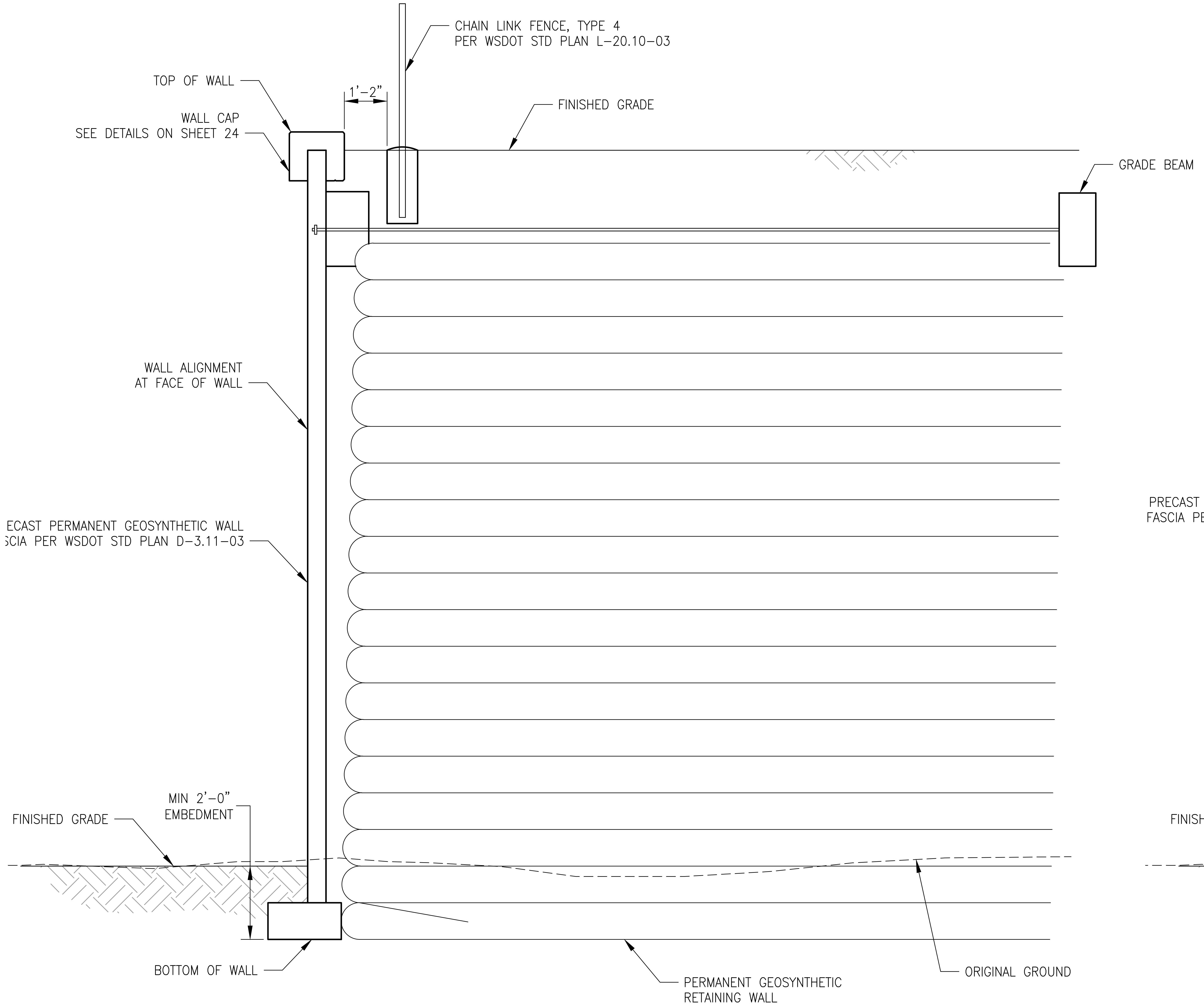
REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 08/2018
SCALE: AS NOTED
DWG NO.: W03

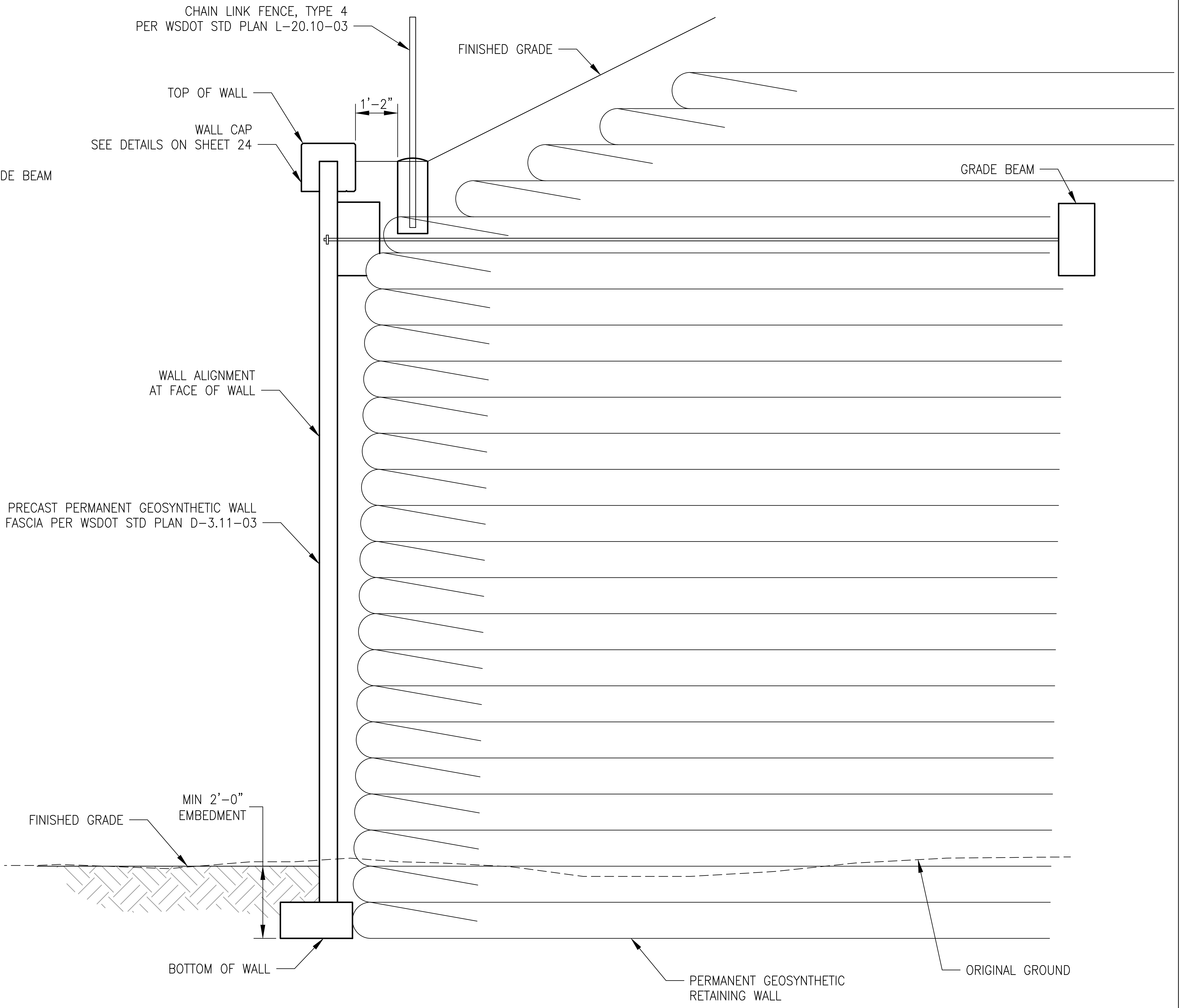
SOUTH KELSO RAILROAD CROSSING
ROADWAY WALL PLAN AND ELEVATION

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
21



SECTION A
SCALE: 1/2"=1'-0"



SECTION B
SCALE: 1/2"=1'-0"

Plotted: Sep 07, 2018 - 11:27am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913W01.dwg Layout Name: W04

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24136
PRELIMINARY

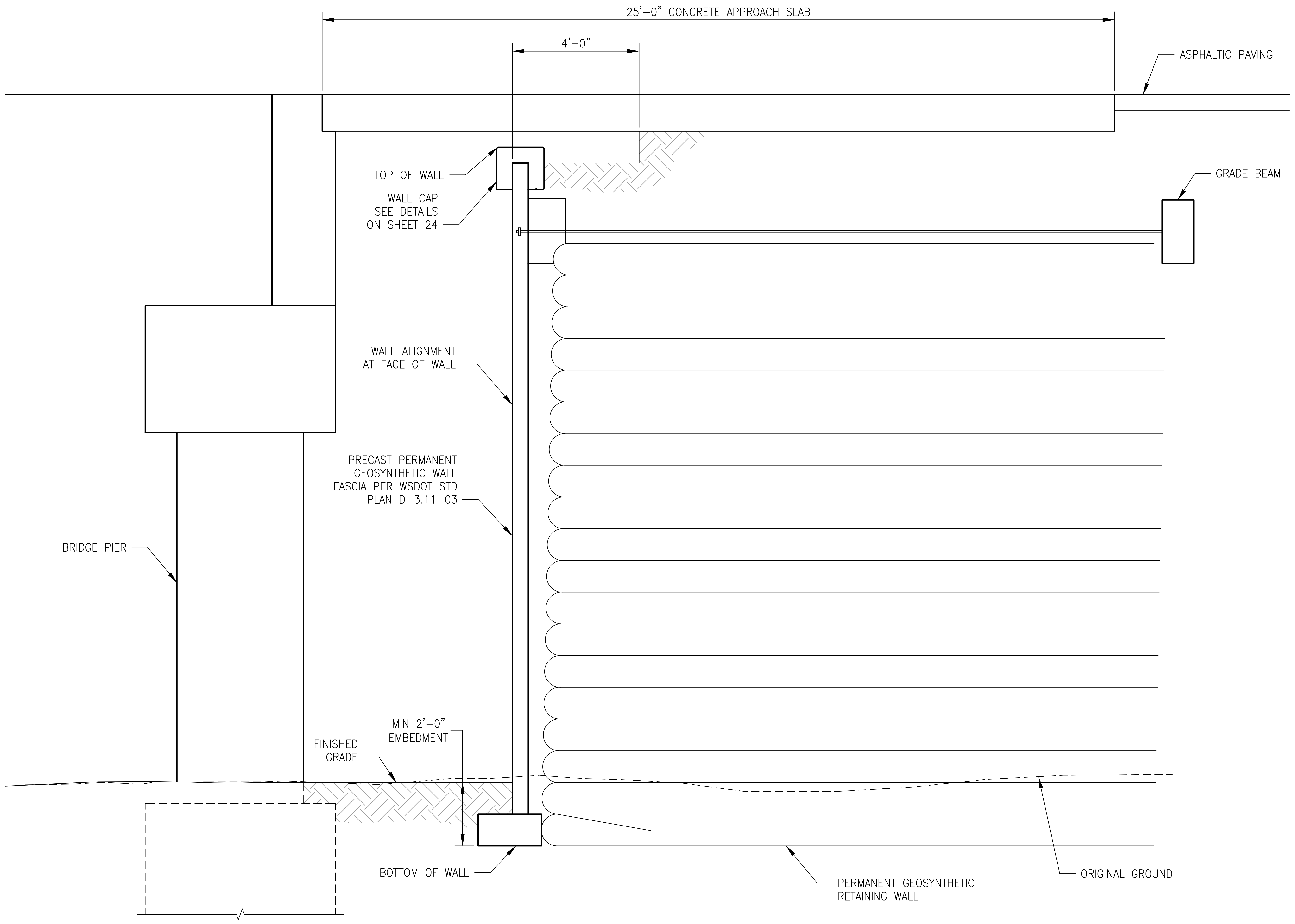
REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 08/2018
SCALE: AS NOTED
DWG NO.: W04

SOUTH KELSO RAILROAD CROSSING
WALL TYPICAL SECTIONS

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
22



SECTION C
SCALE: 1/2"=1'-0"

Plotted: Sep 07, 2018 - 11:28am   V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913W01.dwg   Layout Name: W05

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PRELIMINARY

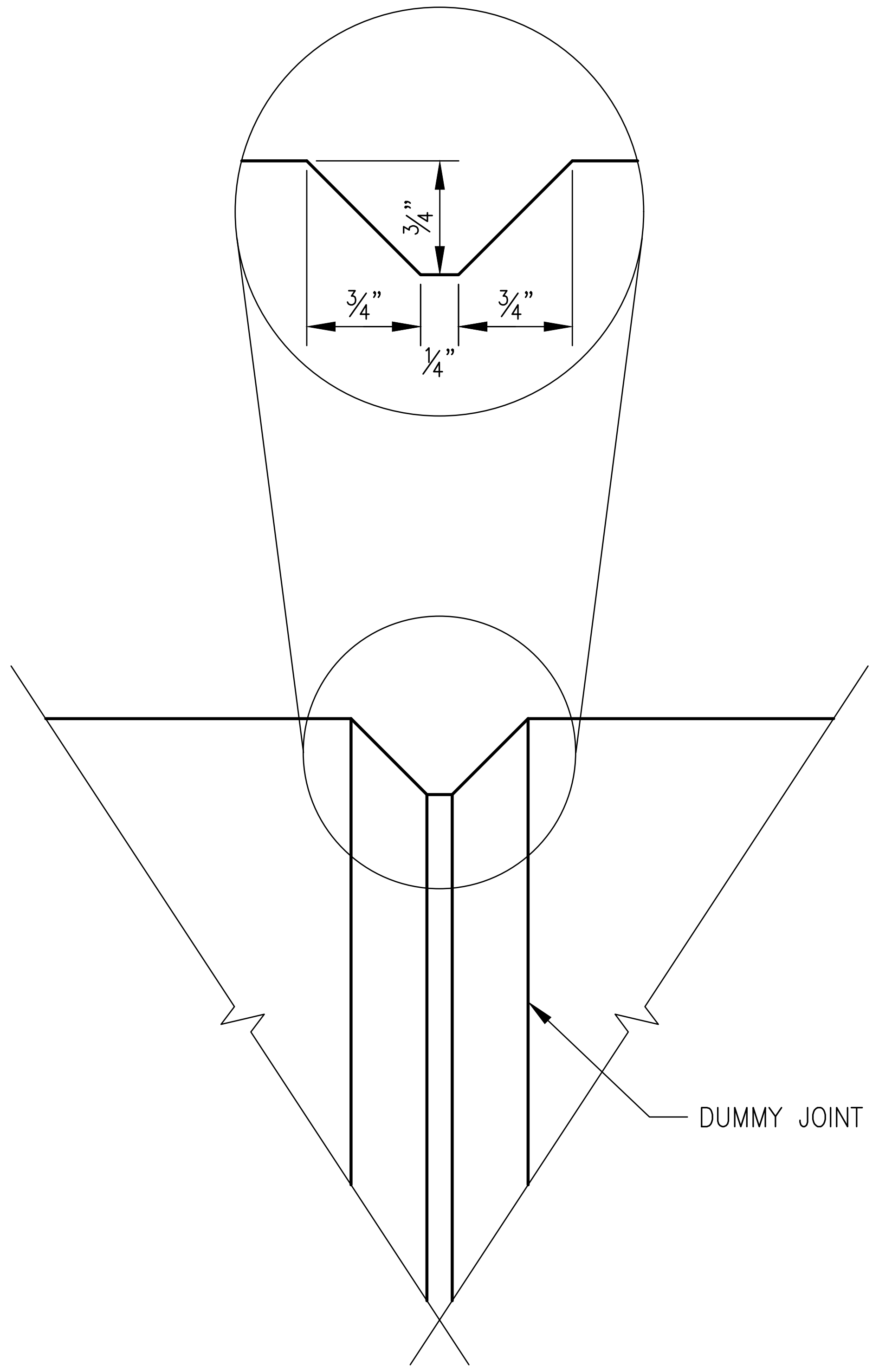
REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 08/2018
SCALE: AS NOTED
DWG NO.: W05

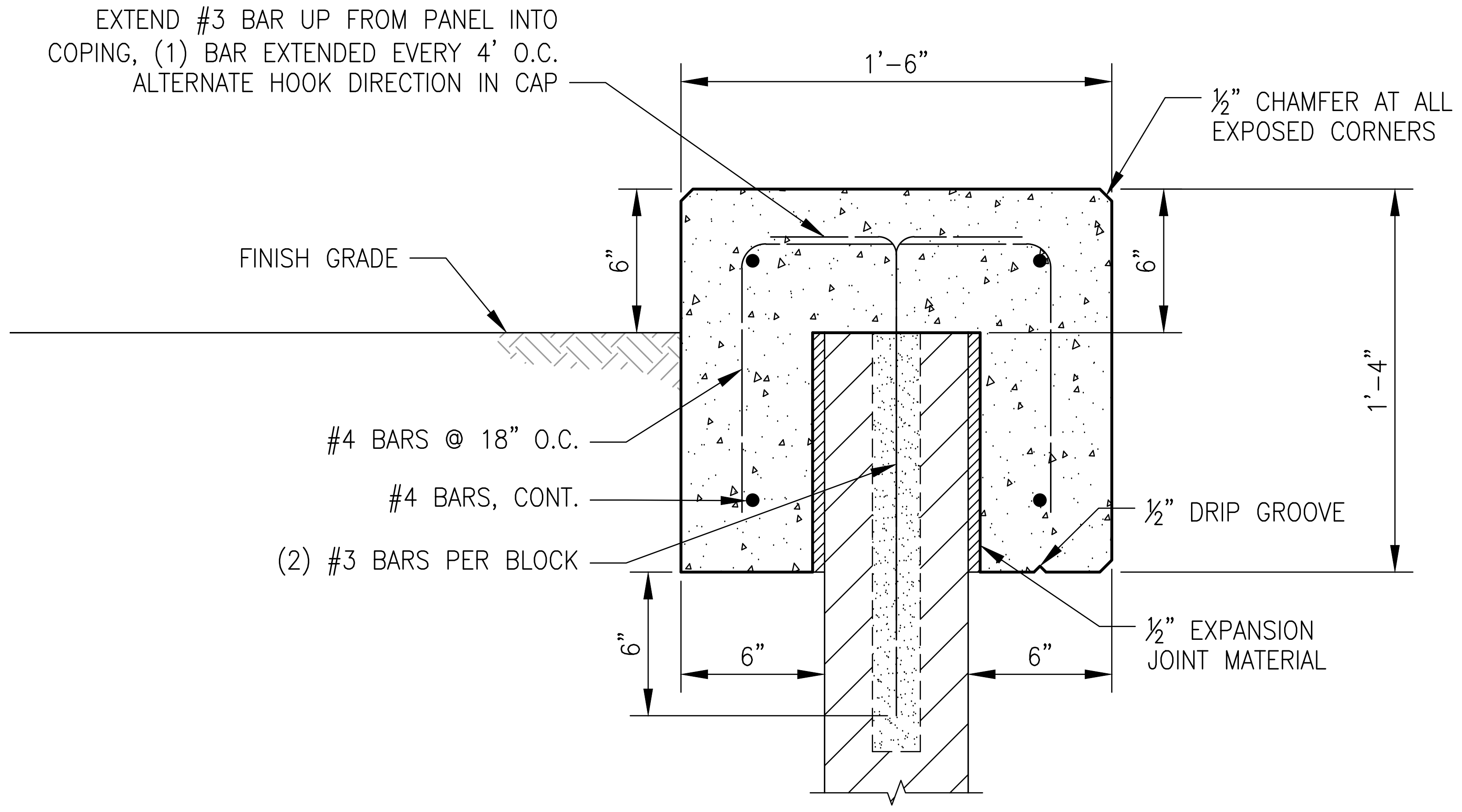
SOUTH KELSO RAILROAD CROSSING
WALL TYPICAL SECTIONS

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

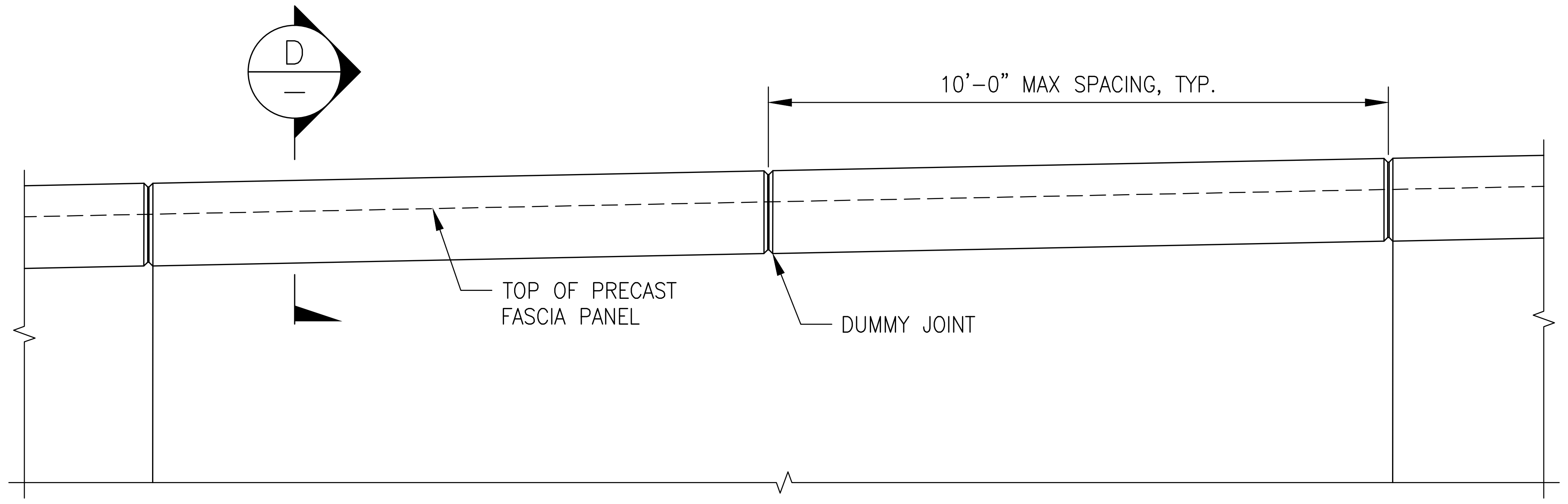
SHEET NUMBER
23



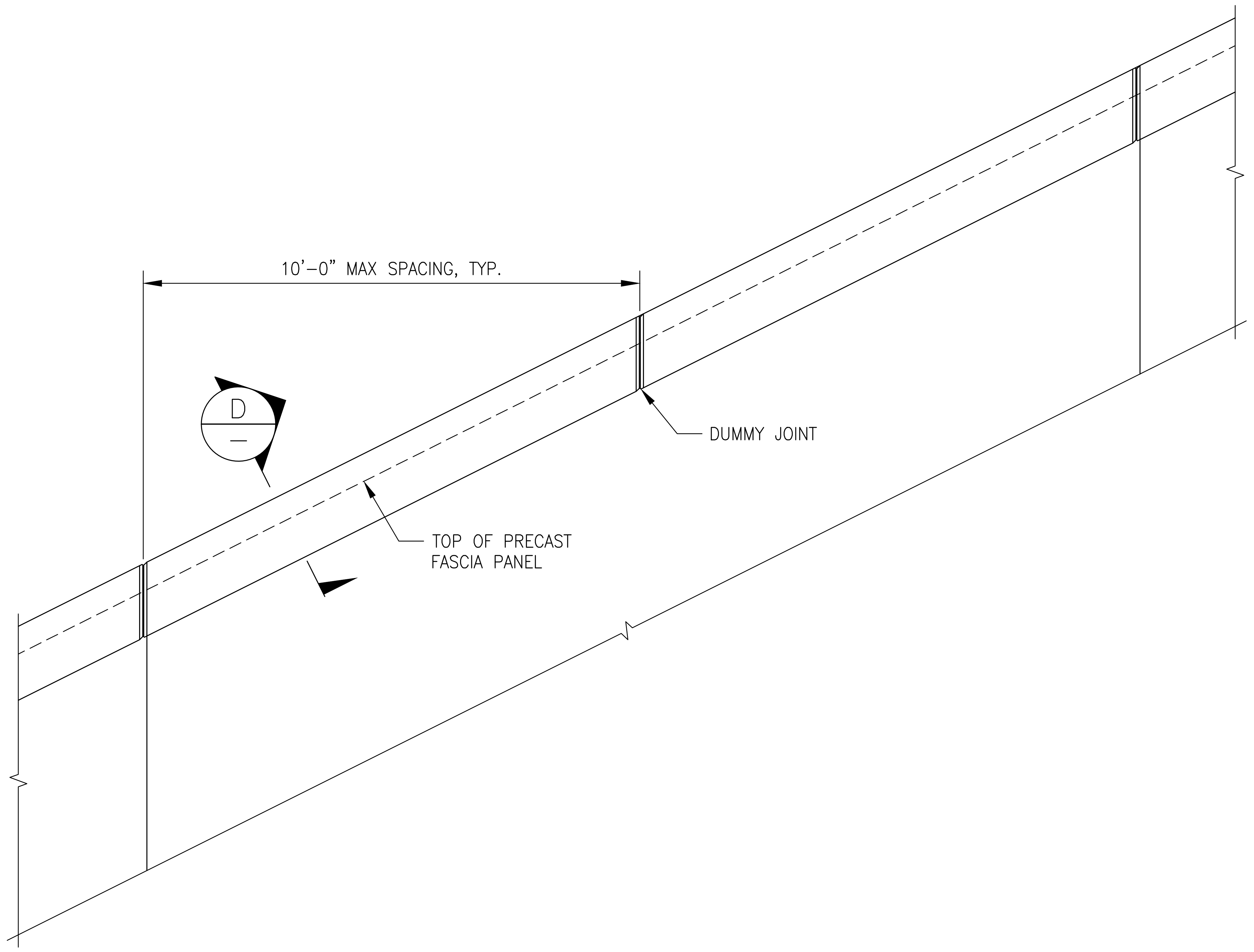
DUMMY JOINT DETAIL
SCALE: 8"=1'-0"



SECTION D/–
SCALE: 1/2"=1'-0"



LEVEL WALL PARTIAL ELEVATION
SCALE: 1"=1'-0"



SLOPED WALL PARTIAL ELEVATION
SCALE: 1"=1'-0"

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PRELIMINARY

REVISION	DATE	DESCRIPTION	BY

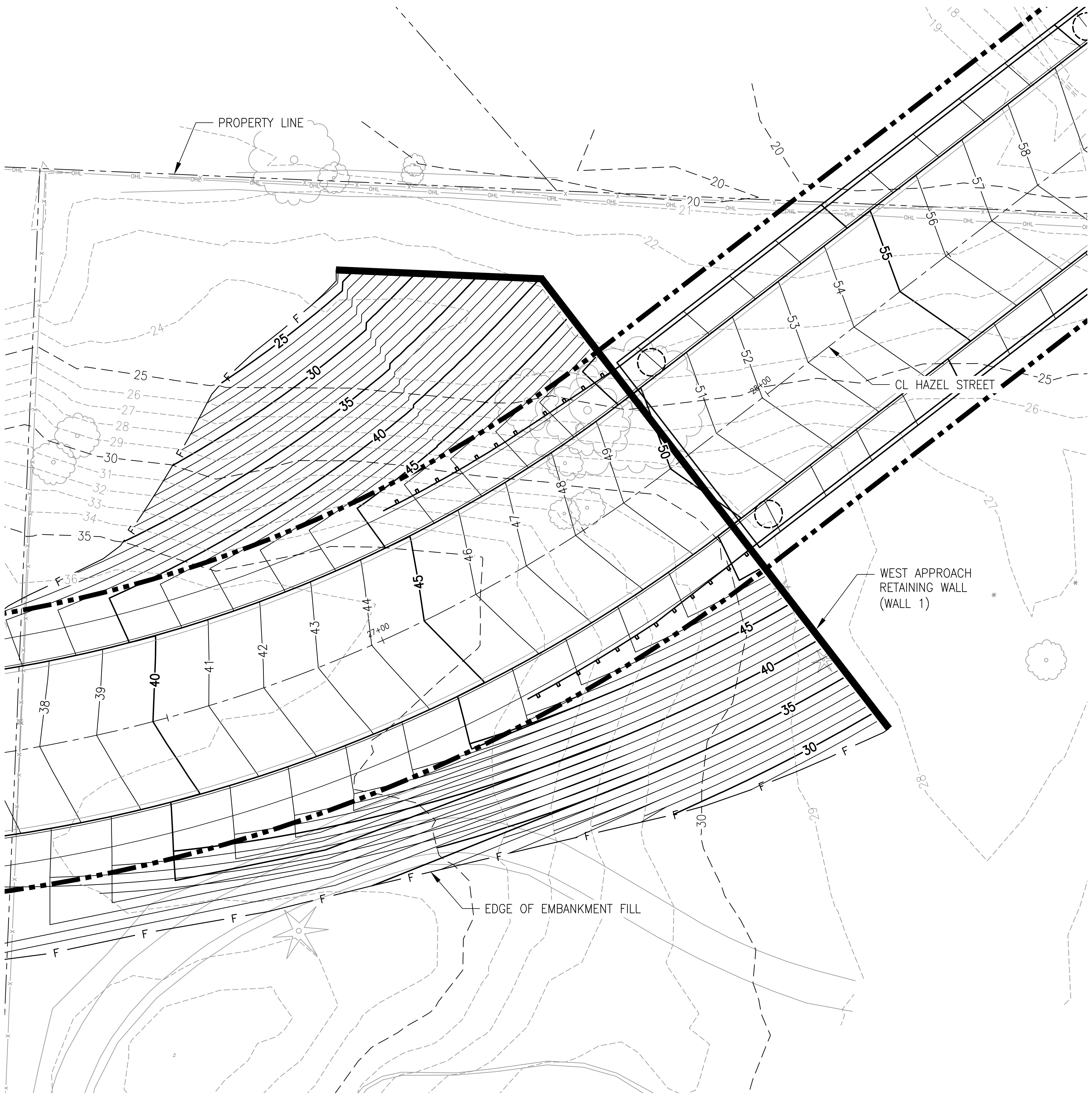
JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 08/2018
SCALE: AS NOTED
DWG NO.: W06

SOUTH KELSO RAILROAD CROSSING
WALL CAP DETAILS

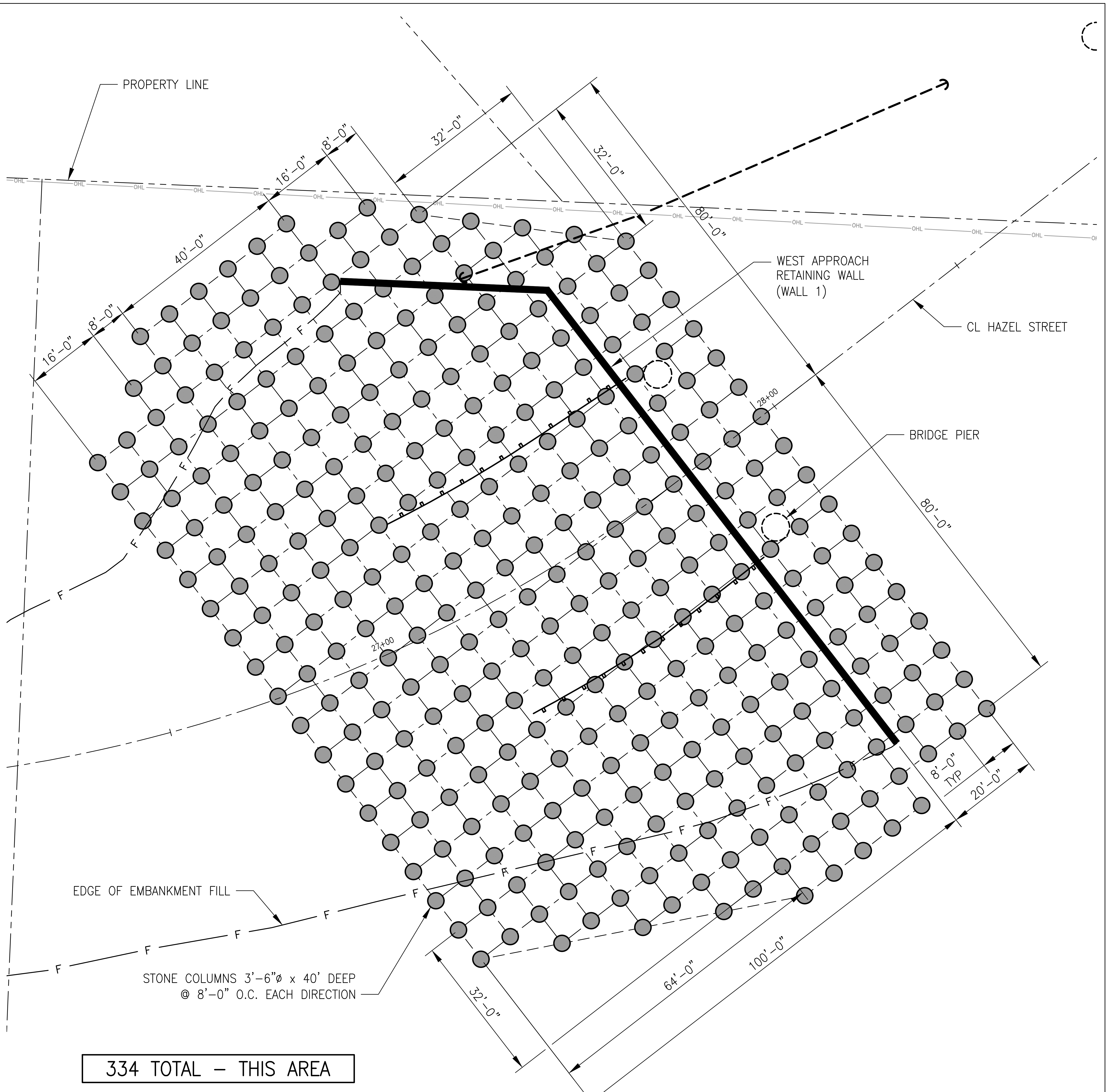
CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER 24

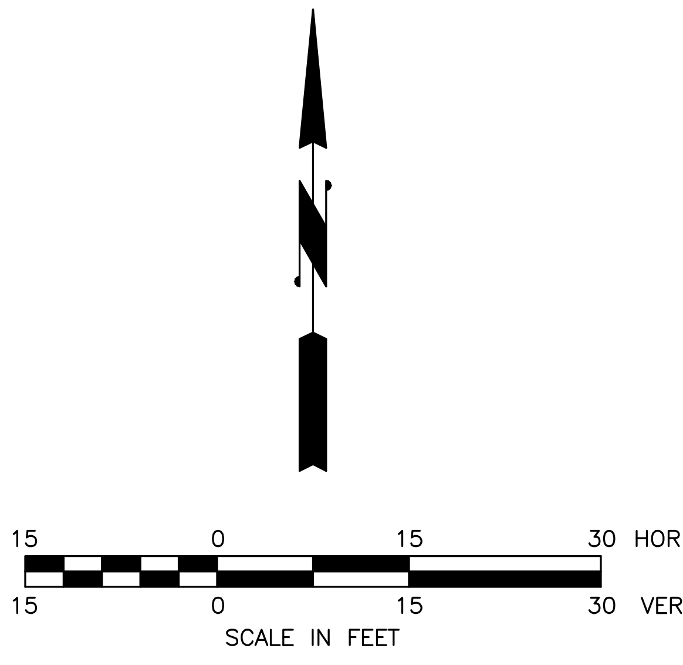
Plotted: Sep 07, 2018 - 11:28am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913W01.dwg Layout Name: W06



WEST APPROACH PLAN
SCALE: 1"=15'-0"



WEST APPROACH GROUND IMPROVEMENT PLAN
SCALE: 1"=15'-0"



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PRELIMINARY

REVISION	DATE	DESCRIPTION	BY

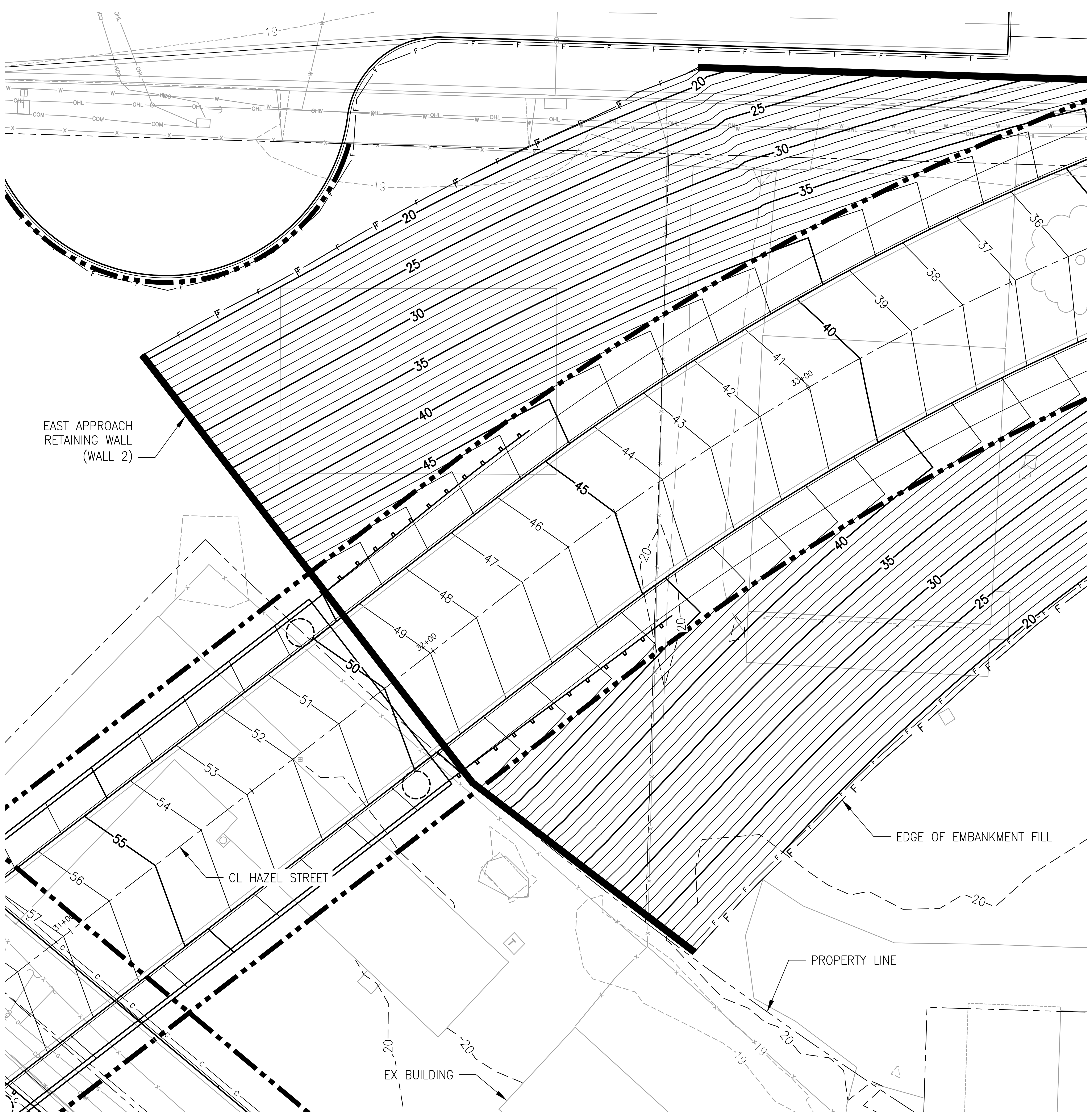
JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 08/2018
SCALE: AS NOTED
DWG NO.: W07

SOUTH KELSO RAILROAD CROSSING
STONE COLUMN GROUND IMPROVEMENTS
WEST BRIDGE APPROACH

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

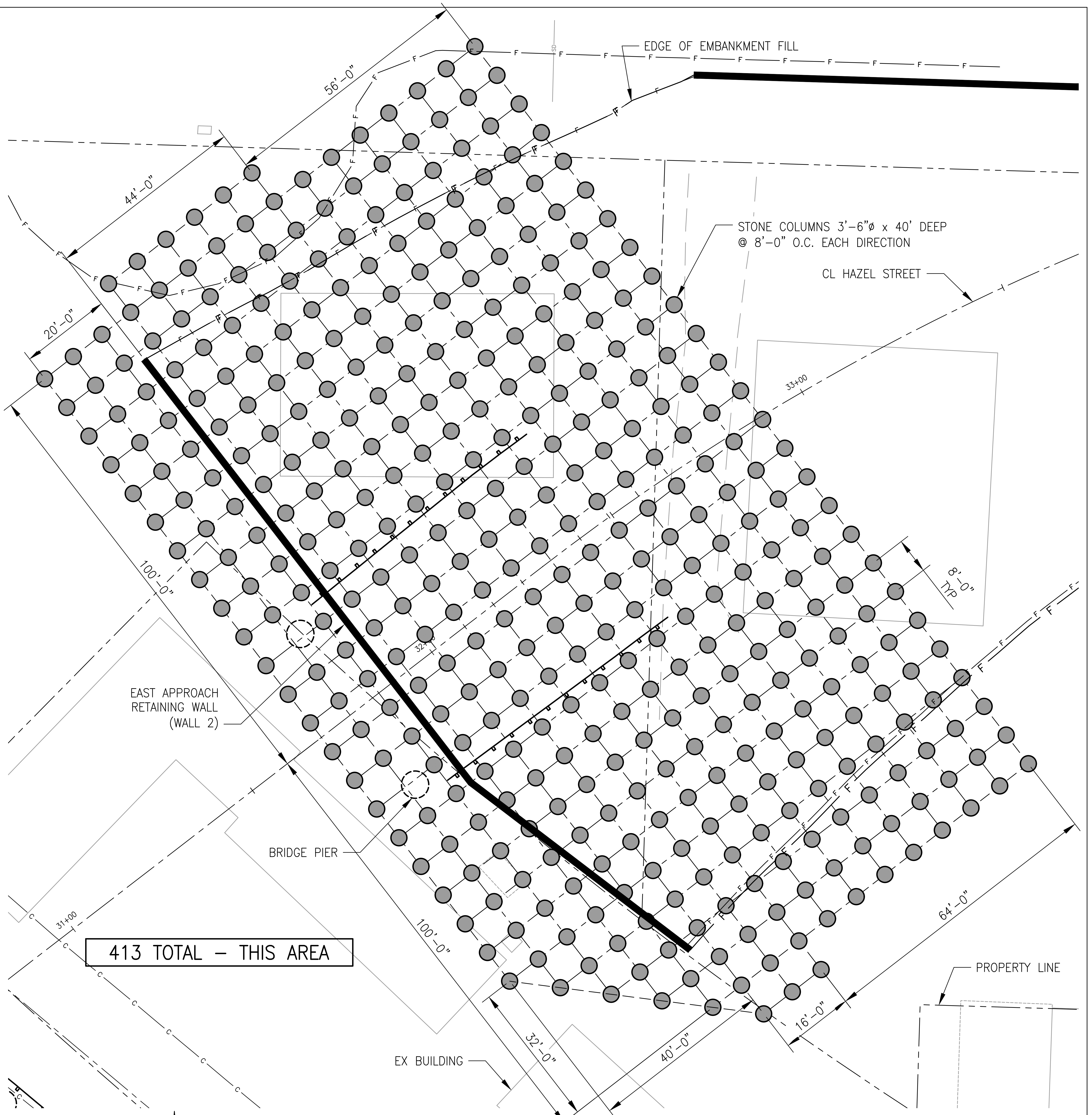
SHEET NUMBER
25

Plotted: Sep 07, 2018 - 11:28am   V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913-W02.dwg   Layout Name: West Bridge Approach



EAST APPROACH PLAN

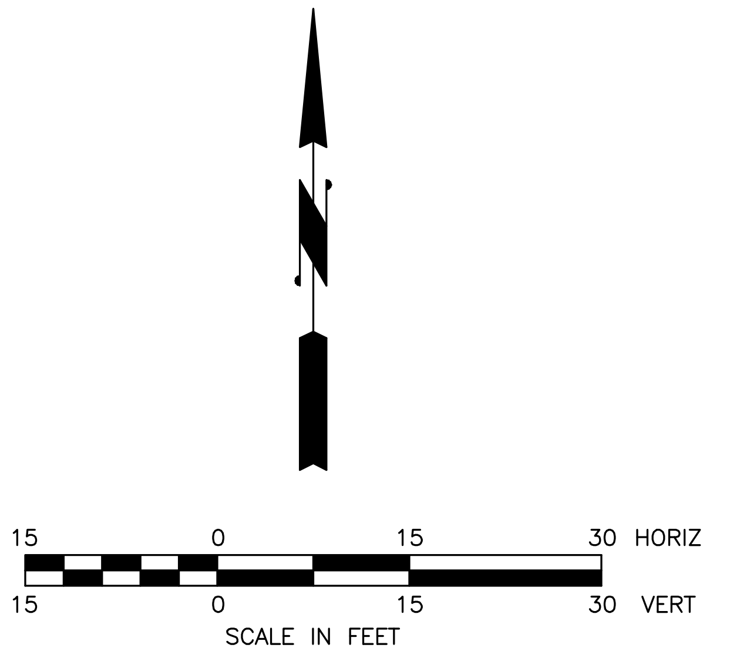
SCALE: 1"=15'-0"



413 TOTAL – THIS AREA

EAST APPROACH GROUND IMPROVEMENT PLAN

SCALE: 1"=15'-0"



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24136
PRELIMINARY

REVISION	DATE	DESCRIPTION	BY

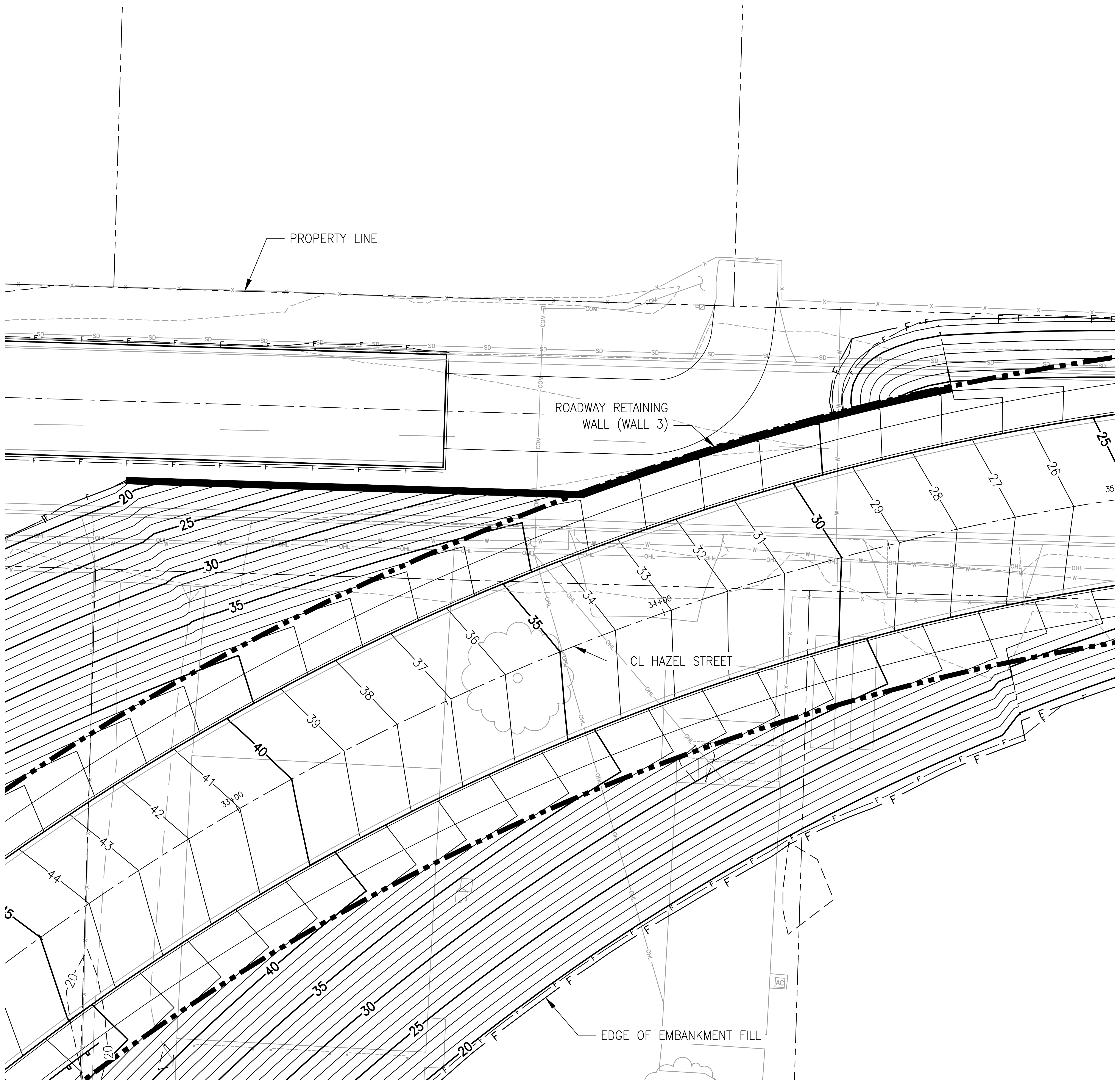
JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 08/2018
SCALE: AS NOTED
DWG NO.: W08

SOUTH KELSO RAILROAD CROSSING
STONE COLUMN GROUND IMPROVEMENTS
EAST BRIDGE APPROACH

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

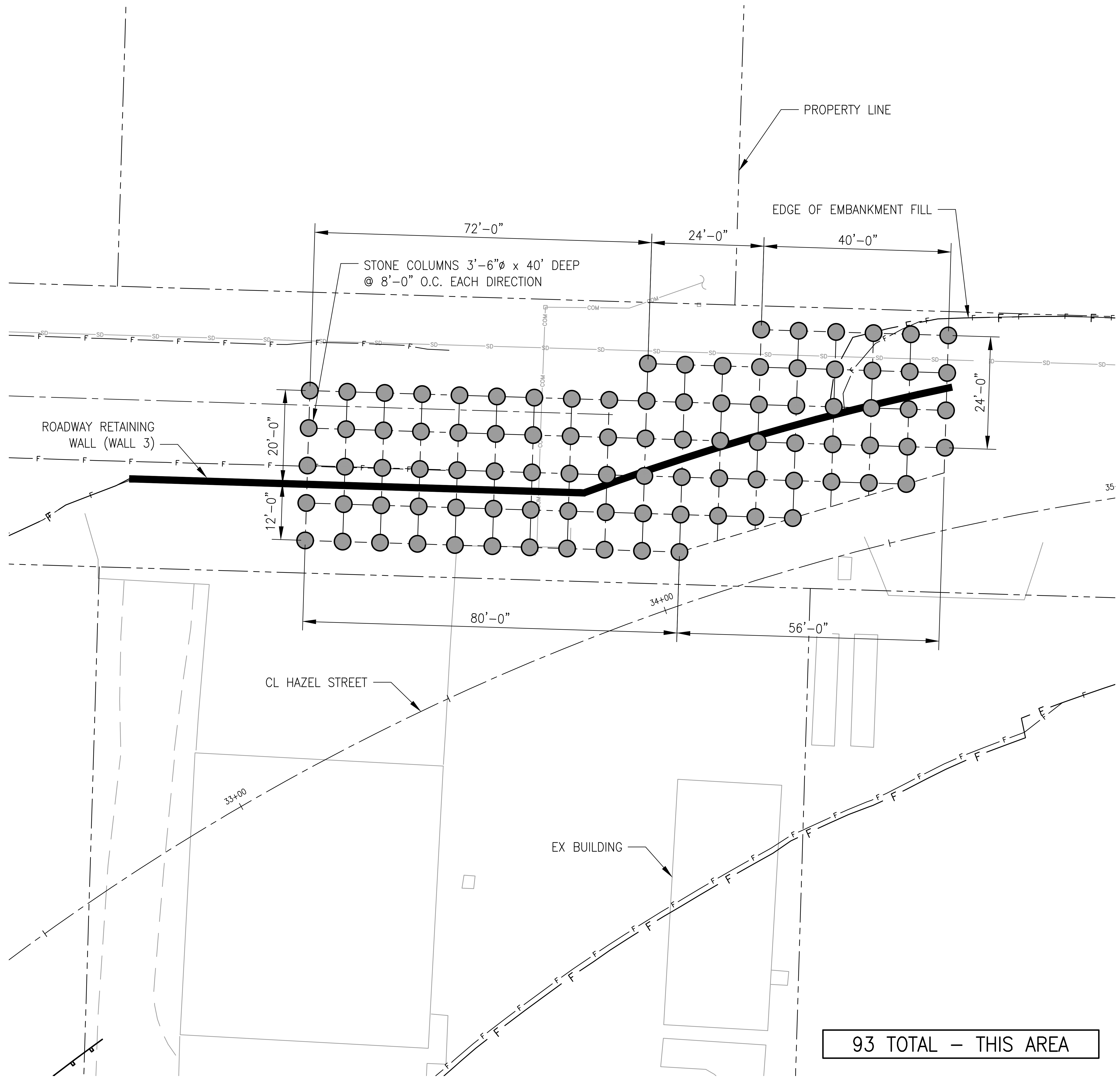
SHEET NUMBER
26

Plotted: Sep 07, 2018 – 11:28am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913-W02.dwg Layout Name: East Bridge Approach



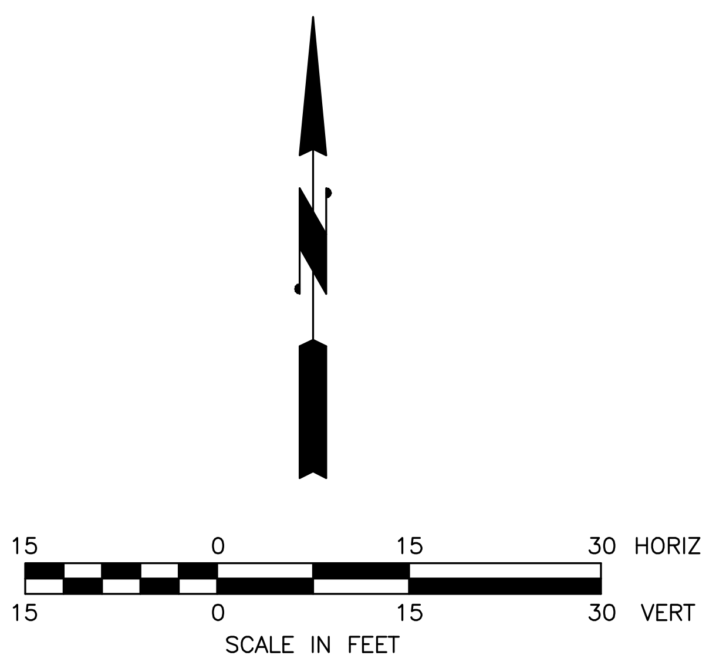
ROADWAY WALL PLAN

SCALE: 1"=15'-0"



ROADWAY WALL GROUND IMPROVEMENT PLAN

SCALE: 1"=15'-0"



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FAX: (360) 737-9651
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24136
PRELIMINARY

REVISION	DATE	DESCRIPTION	BY

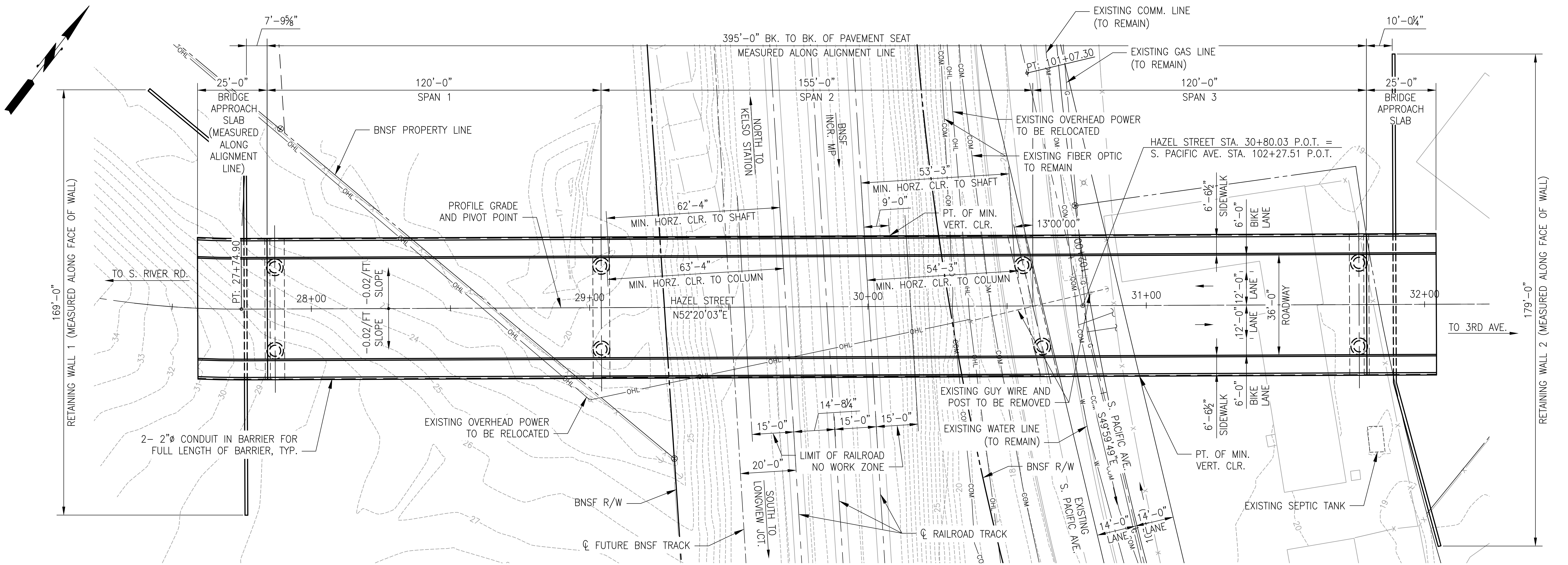
JOB No. 17913
Designed By: PJH
Drawn By: RPW
Checked By: ATH
DATE: 08/2018
SCALE: AS NOTED
DWG NO.: W09

SOUTH KELSO RAILROAD CROSSING
STONE COLUMN GROUND IMPROVEMENTS
ROADWAY WALL

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

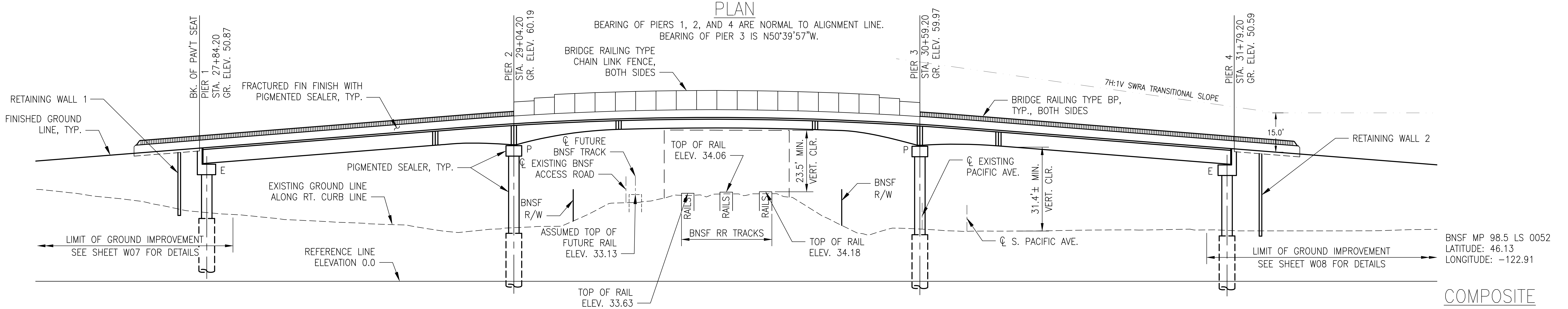
SHEET NUMBER
27

Plotted: Sep 07, 2018 - 11:28am V:\PROJECT\17900\17913\CADD\ACAD\Dwg\17913-W02.dwg Layout Name: Roadway Wall Ground Improv



PLAN

BEARING OF PIERS 1, 2, AND 4 ARE NORMAL TO ALIGNMENT LINE.
BEARING OF PIER 3 IS N50°39'57"W.



BNSF MP 98.5 LS 0052
LATITUDE: 46.13
LONGITUDE: -122.91

COMPOSITE
STEEL PLATE GIRDER
LOADING: HL-93

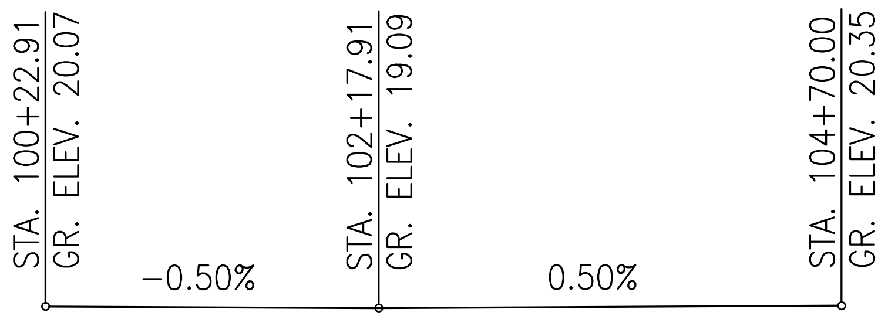
ELEVATION

GRADE ELEVATIONS SHOWN ARE FINISH GRADES AT TOP OF BRIDGE DECK AT CENTERLINE HAZEL STREET AND ARE EQUAL TO PROFILE GRADE

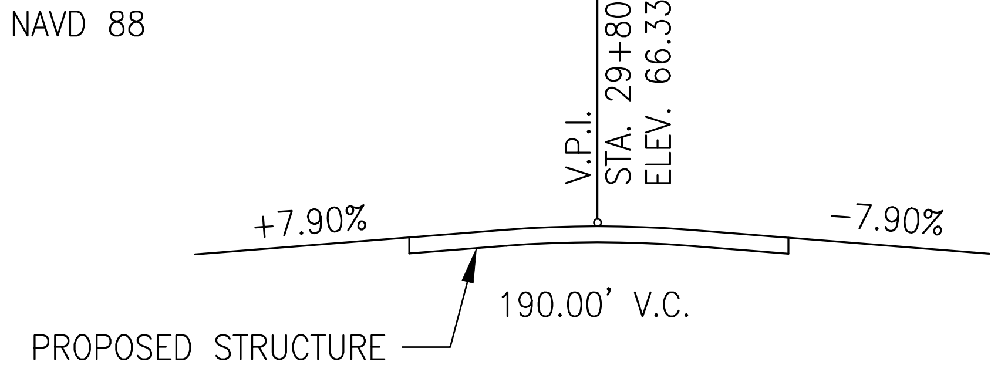
20 0 20 40 HORIZ
20 0 20 40 VERT
SCALE IN FEET

DATUM

NAVD 88



S. PACIFIC AVE. PROFILE



HAZEL STREET PROFILE

HDR
700 WASHINGTON ST.
SUITE 405
VANCOUVER, WA 98660

PRELIMINARY

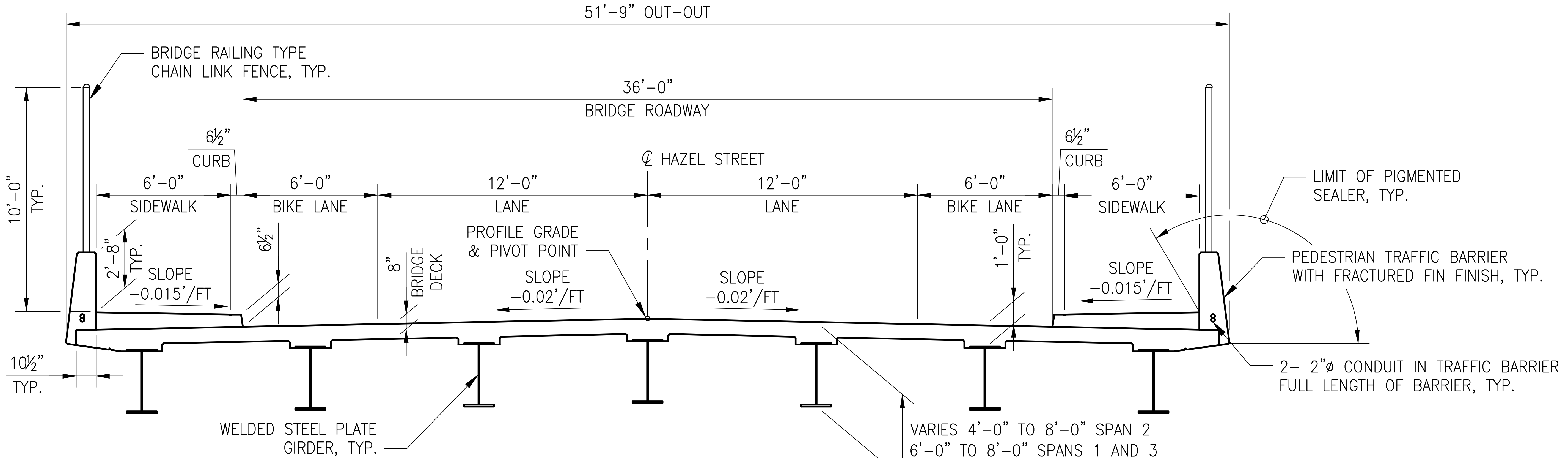
REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: JB
Drawn By: HG
Checked By:
DATE: 09/10/2018
SCALE: AS NOTED
DWG NO.: B01

SOUTH KELSO RAILROAD CROSSING
BRIDGE
PLAN AND ELEVATION

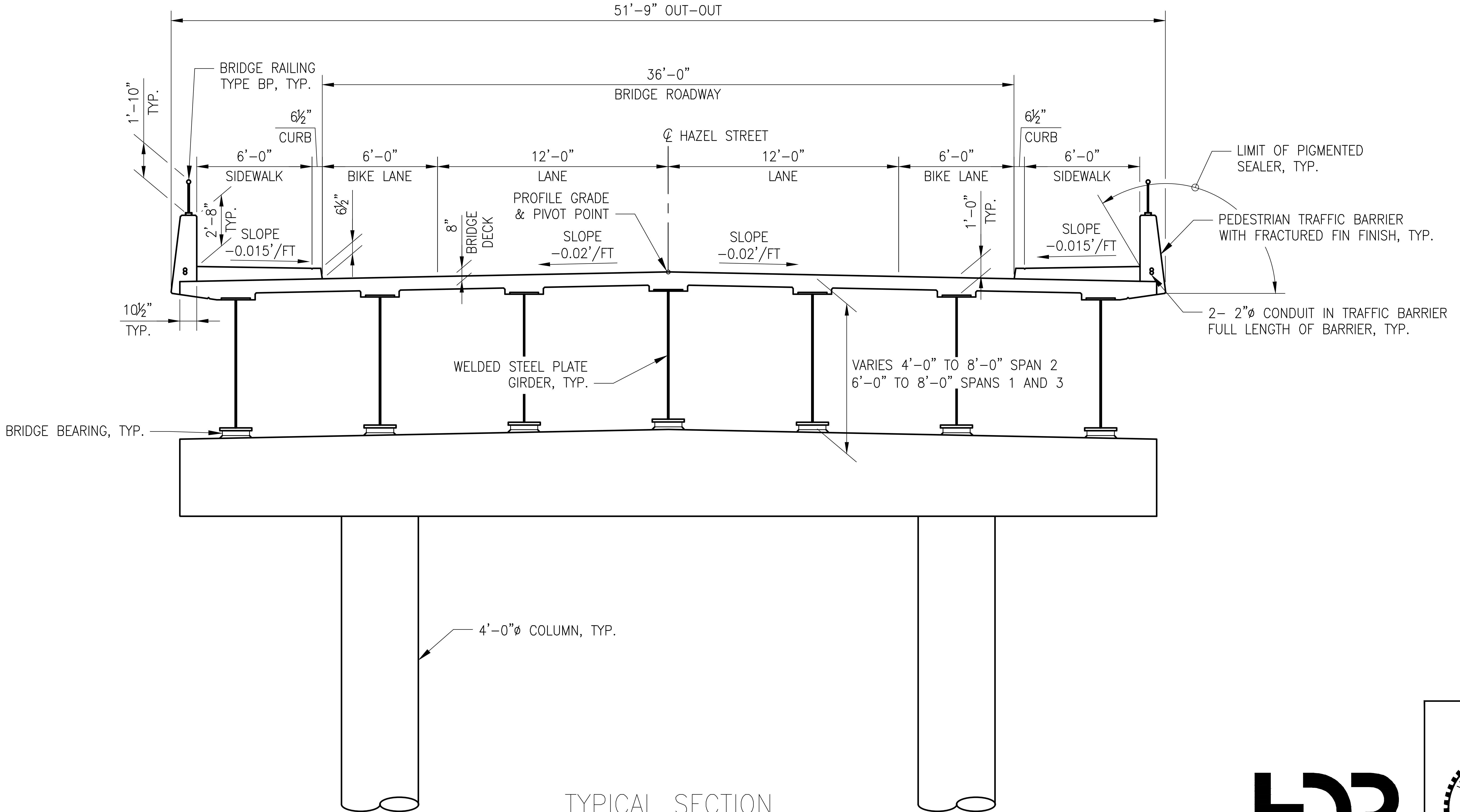
CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
28



TYPICAL SECTION

SHOWN NEAR MIDSPAN SPAN 2.
SCALE: ¼" = 1'-0"



TYPICAL SECTION

SHOWN NEAR PIER 2.
SUBSTRUCTURE DIMENSIONS ARE APPROXIMATE.
SCALE: ¼" = 1'-0"

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PRELIMINARY

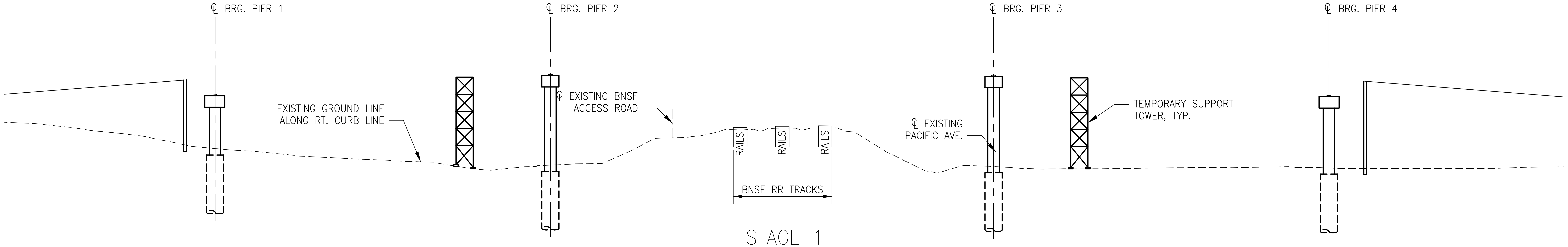
REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: JB
Drawn By: HG
Checked By:
DATE: 09/10/2018
SCALE: AS NOTED
DWG NO.: B02

SOUTH KELSO RAILROAD CROSSING
BRIDGE
TYPICAL SECTIONS

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

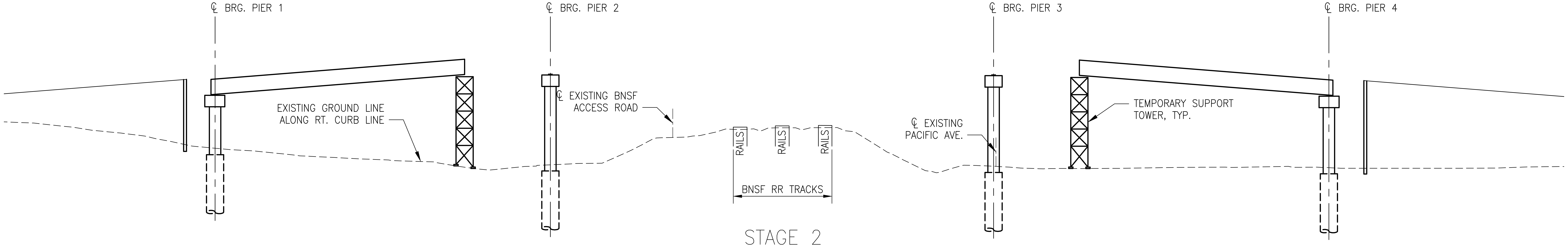
SHEET NUMBER
29



STAGE 1

STAGE 1

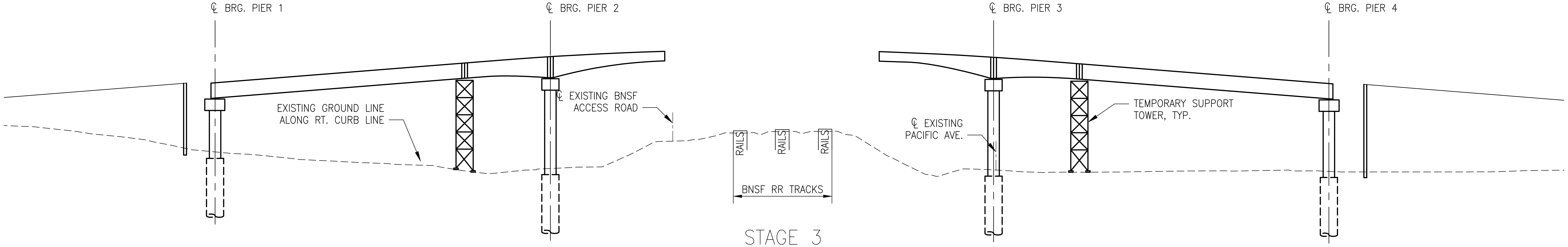
1. CONSTRUCT RETAINING WALL EMBANKMENT.
2. CONSTRUCT DRILLED SHAFTS, COLUMNS, CROSSBEAMS, AND BEARINGS.
3. CONSTRUCT TEMPORARY SUPPORTS FOR STEEL GIRDER SEGMENTS.



STAGE 2

STAGE 2

1. ERECT STEEL GIRDERS.
2. INSTALL CROSSFRAMES.



STAGE 3

STAGE 3

1. ERECT STEEL GIRDERS.
2. INSTALL FIELD SPLICES.
3. INSTALL CROSSFRAMES.

20 0 20 40 HORIZ
20 0 20 40 VERT
SCALE IN FEET

HDR
700 WASHINGTON ST.
SUITE 405
VANCOUVER, WA 98660

PRELIMINARY
45190

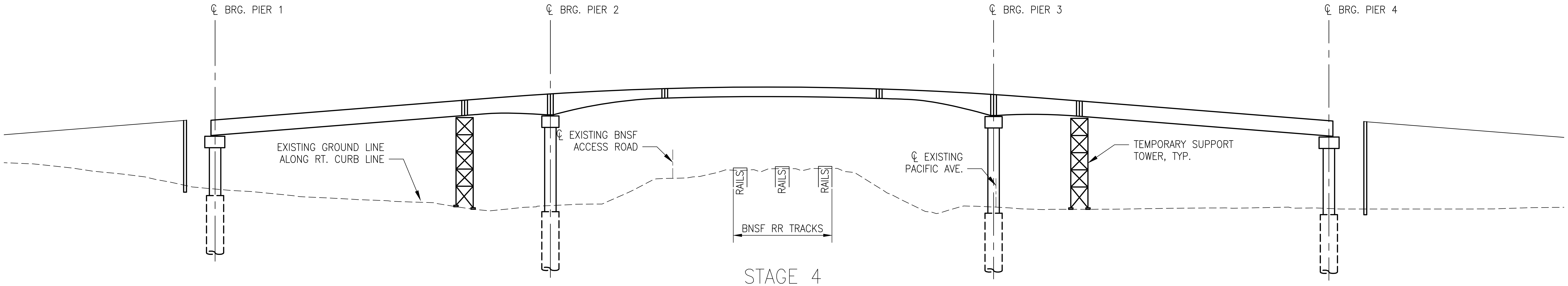
REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: JB
Drawn By: HG
Checked By:
DATE: 09/10/2018
SCALE: AS NOTED
DWG NO.: B03

SOUTH KELSO RAILROAD CROSSING
BRIDGE
CONSTRUCTION SEQUENCE - 1

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

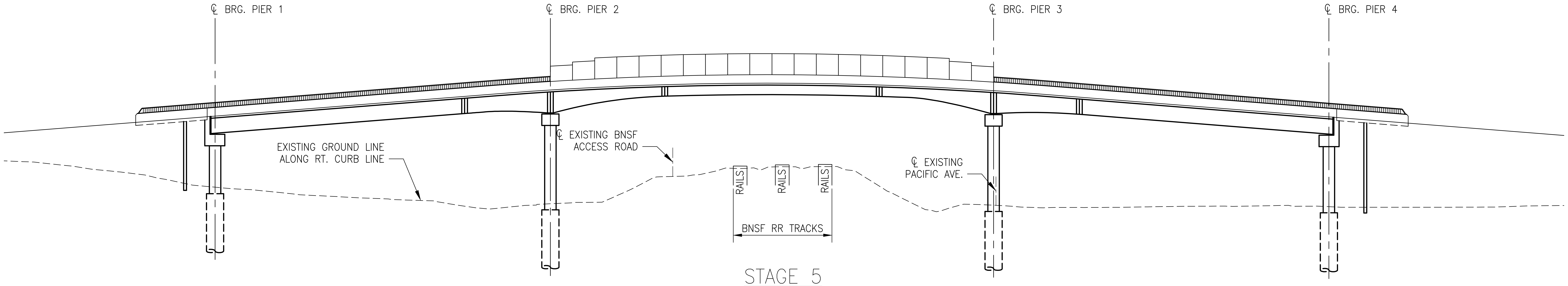
SHEET NUMBER
30



STAGE 4

STAGE 4

1. ERECT STEEL GIRDERS.
2. INSTALL FIELD SPLICES.
3. INSTALL CROSSFRAMES.



STAGE 5

STAGE 5

1. REMOVE TEMPORARY SUPPORT TOWERS.
2. CONSTRUCT BRIDGE DECK, TRAFFIC BARRIER, RAILING, AND BRIDGE APPROACH SLABS.
3. ERECT RETAINING WALL FASCIA PANELS.

20 0 20 40 HORIZ
20 0 20 40 VERT
SCALE IN FEET

700 WASHINGTON ST.
SUITE 405
VANCOUVER, WA 98660

PRELIMINARY

REVISION	DATE	DESCRIPTION	BY

JOB No. 17913
Designed By: JB
Drawn By: HG
Checked By:
DATE: 09/10/2018
SCALE: AS NOTED
DWG NO.: B04

SOUTH KELSO RAILROAD CROSSING
BRIDGE
CONSTRUCTION SEQUENCE - 2

CITY OF KELSO
PUBLIC WORKS DEPARTMENT
203 S. PACIFIC AVE. SUITE 205
KELSO, WA 98626

SHEET NUMBER
31

APPENDIX G

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL / ENVIRONMENTAL REPORT

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

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

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

READ RESPONSIBILITY CLAUSES CLOSELY.

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

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