DAVID EVANS AND ASSOCIATES INC.

SW 53rd Street Railroad Crossing Project
Benton County, Oregon

Type, Size and Location

Prepared for
Benton County

March 11, 2016

[illegible]
MAY 20, 1991
MICHAEL W. REYNOLDS
Exp: 12/31/16

Submitted by
David Evans and Associates, Inc.
530 Center Street NE, Suite 605
Salem, Oregon 97301

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Appendices

INTRODUCTION

Benton County (County) is continuing project development to increase capacity and reliability of SW 53rd Street. The focus of this report is the railroad crossing of SW 53rd Street just south of the recently re-aligned Reservoir Avenue. The road currently passes beneath the Portland & Western Railroad (PNWR) tracks at this location. The proposed project will realign SW 53rd Street to the east, where a new overcrossing will be constructed over the existing tracks. As part of this road improvement project, Dunawi Creek will also be rerouted to pass under both the existing railroad bridge, as well as the new SW 53rd Street overcrossing structure (See Figure 1 below). This will remove all three culverts along this stretch of Dunawi Creek.

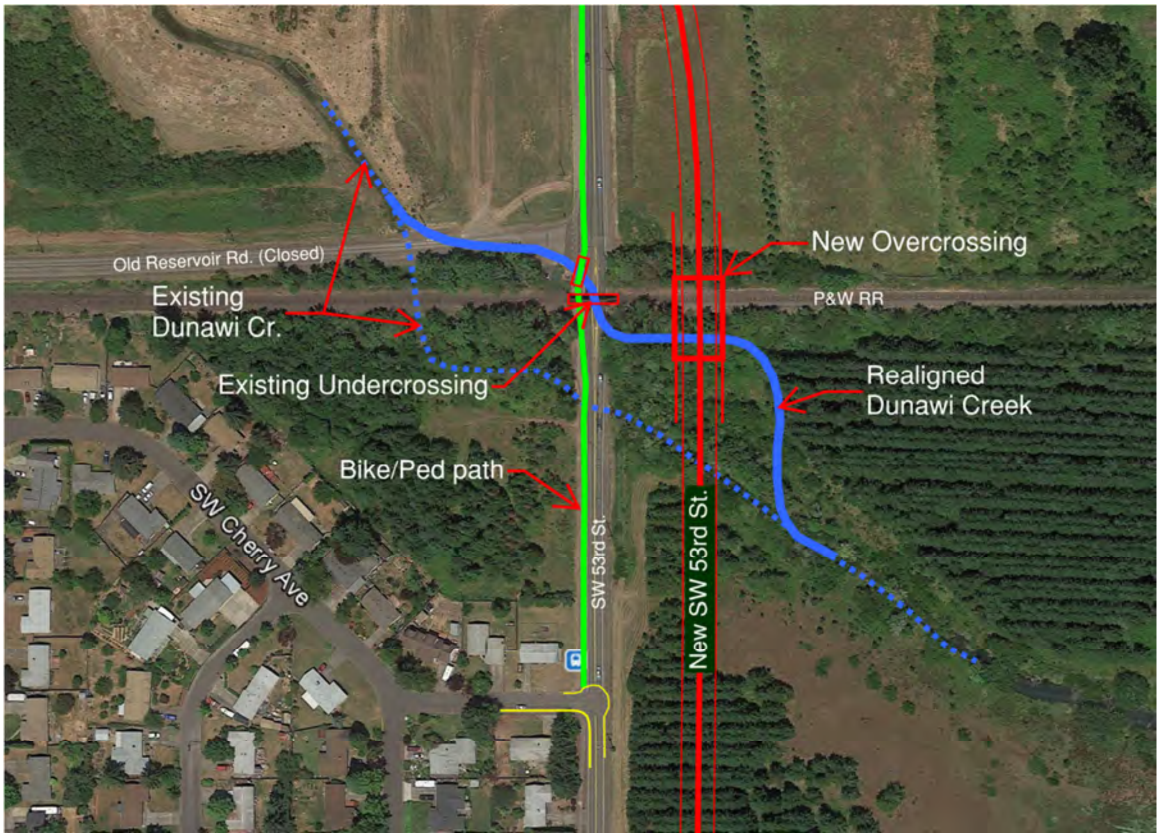


Figure 1. Project Overview

The current railroad structure crossing over SW 53rd Street is a four-span, open deck timber trestle bridge owned by Union Pacific Railroad (UPRR) and leased by PNWR. During high water events, SW 53rd Street beneath the undercrossing regularly floods with 1-2 feet of water and becomes a hazard to the traveling public (Figure 2). This is caused by backwater of Dunawi Creek due to beaver dams downstream and the severe sag vertical curve in SW 53rd Street that creates a low point as it passes under the railroad.

This Type, Size and Location (TS&L) Report describes and summarizes the project and the preferred alternative for the proposed bridge along SW 53rd Street in Benton County, Oregon.

Figure 2. Typical flooding

Project Location and Existing Conditions

Existing SW 53rd Street in the vicinity of the undercrossing is a two-lane rural roadway section with narrow unpaved shoulders. A multi-use path parallels SW 53rd on the west side. At the undercrossing, the multi-use path diverges to pass beneath the west exterior span of the railroad trestle. The two traffic lanes are divided at the bridge to pass below Spans 2 and 3. The vertical profile of SW 53rd Street at the undercrossing has a sharp sag vertical curve with a 13'-6" vertical clearance which limits freight passage under the railroad structure.

Figure 3. Existing Railroad Bridge, looking north

Dunawi Creek currently passes through three culverts in the immediate vicinity of this project. These create fish passage barriers along the creek. Flowing from the northwest to the southeast, the creek first passes below the old Reservoir Avenue, which has since been closed, and is no longer used. The stream then passes below the railroad before turning east where it passes below SW 53rd Street, about 140 feet south of the railroad trestle.

Project History and Previous Phases

Planning for the project started as early as 1988, driven by the safety concerns, low clearance and annual flooding beneath the railroad bridge. Though the project has started and stopped many times, a bridge type selection contract was awarded to David Evans and Associates, Inc. (DEA) in 2009 to determine the most cost effective bridge type. This study looked at realignment of SW 53rd Street as an overcrossing above the tracks, as well as raising the existing road, and railroad tracks above, to improve safety, flooding and vertical clearance as a new undercrossing. The cost estimates for both the overcrossing and undercrossing alternatives were very close. Toward the end of the study, the County was able to secure 14 used prestressed box beams, from the Oregon Department of Transportation (ODOT), to be repurposed for this project. The cost savings of these beams was enough to make the overcrossing option the most cost effective and the preferred alternative.

Although using the repurposed beams for this project is still a possibility, for the purpose of this report it is assumed that the beams could be used by the County for other projects prior to this project obtaining construction funding. Therefore, the design and construction cost estimate assumes that new beams of similar size and length would be produced for this project.

One significant benefit of the overcrossing option is that it allows the existing Dunawi Creek to be realigned beneath the existing railroad bridge, eliminating the need for the three existing culverts, improving fish passage and stream habitat. Because of the possibility of using the existing repurposed beams, the bridge alternative study assumed that the bridge length and configuration was set and did not take into account the stream realignment in trying to optimize a bridge layout.

SUMMARY OF PROPOSED PROJECT

The preferred alternative for the SW 53rd Street overcrossing bridge is a 56’-2” wide by 113’ long, single-span bridge. The bridge will span over the 60’ railroad right-of-way (ROW), as well as the 40’ County easement which will contain the new realigned Dunawi Creek. It will provide the minimum required vertical clearance of 23’-6” over the existing PNWR tracks. The superstructure will consist of side-by-side 48” precast prestressed box girders. The bridge will have two 12-foot lanes, one in each direction, separated by a 6-foot median, two 6-foot shoulders/bike lanes, and a 6’-2” wide sidewalk on each side. Standard ODOT sidewalk mounted combination bridge rails will be used on the bridge, with 8-foot pedestrian fencing, as required by railroad standards. In order to accommodate the tall approach fills on both sides of the new bridge, large MSE walls will be needed, approximately 44 feet high. The walls will wrap around the front of each abutment and extend back along each side, parallel to the new roadway.

The new alignment of SW 53rd Street will diverge from the existing alignment approximately 800 feet north of the railroad tracks, veering east and then south, to parallel the existing roadway about 140 feet east of its current centerline. It crosses over the railroad, and the newly realigned Dunawi Creek before veering southwest to connect back into the existing SW 53rd Street. The alignment of the new road south of the overcrossing is still being finalized by the County and must ensure access will be maintained to the neighborhood located southwest of the project via SW Willow and SW Cherry Avenues.

The multiuse path which parallels SW 53rd Street will remain largely in its current location after completion of the project. The path will continue to cross below the existing railroad bridge. A new pedestrian bridge will be needed over the realigned Dunawi Creek just north of the undercrossing.

Construction of the bridge and approach roadways will be done on new right-or-way and easements, allowing traffic along SW 53rd Street to be maintained in its current location during construction. A temporary culvert will likely be used to carry Dunawi Creek below the new south approach fills until construction of the new channel is complete. After transferring the creek to the new channel, the temporary culvert and the culverts beneath the railroad will need to be filled/decommissioned.

See Appendix A for the Concept Bridge Plans.

This project is being developed by a combined team with David Evans and Associates, Inc. (DEA) and Benton County. The DEA team, which includes subconsultants Foundation Engineering Inc. (FEI), and WEST Consultants (WEST), is providing the bridge design, foundation and pavement design and hydraulics design. The County is providing all other design and permitting activities including but not limited to:

- Draft Wetland Delineation Report
- Wetland Mitigation Plan
- Rare Plant Survey Memorandum
- Stormwater Design and Plans
- Temporary Erosion Control Plans
- Temporary Traffic Control (TP&DT) Plans and Details
- Utility Coordination
- Roadway Design Criteria
- Roadway Design and Plans
- Permanent Roadway Striping and Sign Design
- Right-of-Way

Documentation of the above items will be completed by the County and is not included in this TS&L Report.

DESIGN STANDARDS

Bridge Design Criteria

The bridge design will conform to the standards set forth in the AASHTO LRFD Bridge Design Specifications with the 2016 Interim Revisions (LRFD), and the 2015 ODOT Bridge Design and Drafting Manual (BDDM). The design live load for the proposed bridge is HL-93. The LRFD live load includes a design lane loading of 640 pounds per lineal foot in combination with either a design truck or tandem axles, whichever produces the greatest load effects. The design loading will include an additional allowance of 25 pounds per square foot (psf) for a future wearing surface. Standard ODOT sidewalk mounted combination bridge rails will be used on the bridge, with 8-foot pedestrian fencing, in order to meet railroad standards.

If the repurposed prestressed box beams are used, it is recommended that a load rating be completed to determine their actual capacity. Given that they are only about 12 years old and carried interstate traffic for many years, we would expect that they would be adequate for this project.

Seismic design loads will be evaluated in accordance with the ODOT BDDM implementation of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2011. The 1000-year Peak Ground Acceleration Coefficient (PGA) is 0.24 and the site is classified as Site Class D.

GEOTECHNICAL INVESTIGATIONS AND FOUNDATIONS

Regional and Site Geology

The city of Corvallis (City) is located between the western edge of the central Willamette Valley and the eastern foothills of the Coast Range. The City is set on gently sloping foothills and a broad, flat terrace adjacent to the Willamette River. This setting has created a variety of geologic terrains beneath the City. Fluvial and lacustrine sediments (Quaternary alluvial terrace deposits) underlie the lower-lying areas, including downtown Corvallis, the Oregon State University (OSU) campus, and the SW 53rd Street crossing site (Bela, 1979; Yeats et al., 1996; O'Connor et al., 2001; Wiley, 2008). The alluvial sediments thin toward exposures of older, well-indurated sedimentary rock (Eocene Spencer and Flournoy Formations) in the low hills to the south and west.

Explorations performed by Foundation Engineering, Inc. (FEI) indicate the project site is underlain by alluvium including a thin mantle of Willamette Silt, followed by sandy silt, silty sand, silty gravel and silt. The soil profiles encountered in our explorations are consistent with the mapped local geology. Based on review of local well logs, it is anticipated the depth to bedrock exceeds ±100 feet in this area.

Field Exploration

Five exploratory boreholes were drilled at the site between October 27 and 29, 2014. BH-1 and BH-2 were drilled along the north approach and BH-5 was drilled on the south approach. These borings provide subsurface information for the design of the new approach embankments. BH-

3 was drilled near the north abutment and BH-4 was drilled near the north abutment. These borings provide subsurface information for the design of the bridge foundations and MSE walls.

Four exploratory test pits were dug at the site on November 7, 2014, to supplement the borings and provide additional subsurface information for the design of the approach embankments.

Laboratory Testing

Natural water contents, Atterberg limits and percent fines tests were completed on selected soil samples to classify the soils and estimate their overall engineering properties. Two, one-dimensional consolidation tests were also run on relatively undisturbed samples obtained in the upper ±10 feet of BH-2 and BH-4. These tests were run to evaluate the compressibility of the fine-grained soil beneath the new approach embankments.

The Foundation Report for the project is located in Appendix C.

Foundations

Shallow foundations are not practical to support the new bridge due to the required tall approach embankments and the risk of abutment settlement. Therefore, deep foundations (drilled shafts or driven piles) will be required. Drilled shafts are typically more expensive than driven piles and would be more difficult to install. Therefore, driven piles are preferred. Driven piles should be able to attain relatively high axial resistances with modest embedment into the very dense silty gravel encountered at depths of ±7.5 to 12.5 feet below the base of the planned MSE walls.

We recommend constructing the MSE walls with corrugated metal pipe (CMP) sleeves installed in the wall backfill at the pile locations. This approach will allow the piles to be driven after the MSE walls and approach embankments are constructed, thereby reducing or eliminating the downdrag forces on the piles caused by the settlement of the soil beneath the wall.

Steel pipe piles or H-piles could be used. Pipe piles are preferred because they will attain the required axial resistance with less penetration relative to H-piles. Pipe piles will also provide symmetric lateral resistance. PP16x0.5 and PP24x0.5 piles were considered. PP24x0.5 (ASTM A-252 Grade 3 steel) piles were selected based on the design loads, the soil conditions and the need to support the abutments on a single row of piles.

A CMP sleeve with a 30-inch inside diameter can be used with the 24-inch diameter piles. The annulus between the piles and CMP sleeves should be backfilled with pea gravel to allow post-construction settlement of the walls (if any) to occur without mobilizing downdrag loads on the piles within the wall backfill zone. The pea gravel will also provide flexibility to help accommodate relative lateral movement between the piles and MSE walls.

MSE Retaining Walls

MSE walls are planned to retain the approach fills at both abutments. The walls will wrap around the bridge abutments and extend ±85 feet back along the sides of the approaches parallel to the street. An MSE wall height of ±33 feet is anticipated in front of the abutments beneath the abutment walls and pile caps. The MSE walls parallel to the street will have a maximum height of ±44 feet along the sides and behind the abutments. These walls will step up the approach embankments and become shorter as they extend back from the bridge abutments.

The MSE walls will be designed using a proprietary system with internal stability analysis and design provided by the manufacturer. FEI will provide soil parameters for the MSE wall design, as well as external stability checks including; bearing capacity, sliding resistance and overturning resistance and global stability of the retained fill and slope.

HYDRAULICS

WEST Consultants, Inc. has prepared a Bridge Hydraulics and Scour Assessment Report for the proposed SW 53rd Street Overcrossing Project. As part of the project, portions of Old Reservoir Avenue and 53rd Street will be removed and Dunawi Creek will be realigned to flow eastward along a portion of the Old Reservoir Avenue, then southward underneath an existing railroad bridge, where 53rd Street is currently located, eastward beneath the new overpass, and southward to its connection with the existing channel. Additionally, a pedestrian bridge will be added across Dunawi Creek to the northwest of the 53rd Street railroad overpass. As part of this work, hydraulic and scour evaluations were performed to determine the hydraulic impacts of the proposed project and assist in the project designs.

Scour Calculations

A hydraulic and scour evaluation for the construction of a new 53rd Street overpass bridge, a new pedestrian bridge, and the existing railroad bridge over Dunawi Creek was conducted. Scour calculations estimated a total scour depth of 5.2 feet for the existing railroad bridge and 0.6 feet for the proposed pedestrian bridge. The proposed 53rd Street bridge is not expected to induce any scour. However, some long-term adjustment to the longitudinal profile of the channel should be expected.

Abutment Riprap

A Using the ODOT and HEC-11 criteria for riprap revetment, a D50 of 0.08 feet, 0.05 feet, and 0.04 feet was calculated for the proposed pedestrian bridge, existing railroad bridge, and proposed 53rd Street bridge abutments. This corresponds to ODOT Class 50 English riprap. The longitudinal extents of the riprap should extend sufficiently upstream and downstream to prevent flanking of the riprap. All riprap revetments should include the standard ODOT toe trench to help prevent potential future undermining that may occur as a result of long-term adjustment to the longitudinal profile.

The Draft Hydraulics and Scour Assessment Report is included in Appendix E.

PAVEMENT DESIGN

The flexible pavement design was performed by FEI. The design traffic was based on a detailed breakdown of traffic counts from previous work performed by FEI in 2003/2004 and an ADT from 2012 with 6.46% trucks and no breakdown of traffic. The County has indicated the truck traffic may increase to 8-10% after the bridge is replaced and the height restrictions are gone. Therefore, adjustments were made for an influx of additional trucks assuming both 8% and 10% trucks.

Based on the ODOT (2011) guidelines, the following pavement sections and mix designs are recommended for the new approach pavements, unless local County practice or experience warrants modifications.

- 2-inch thick (minimum) Wearing Course of Level 2, ½-inch Dense-Graded HMAC with PG 64-22 binder
- 2 to 3-inch thick lifts of Level 2, ½-inch or ¾-inch Dense-Graded HMAC Base Course, with PG 64-22 binder
- 1 inch – 0 Dense-Graded Base Aggregate

Section 10.4 (Table 5) of the ODOT (2011) guidelines indicates the project location does not mandate the use of anti-stripping additives in the HMAC. The 1 inch – 0 Base Aggregate should conform to the material requirements of Section 02630 and grading requirements of Table 02630-1. The Subgrade Geotextile should be a woven geotextile meeting the material requirements in Table 02320-4.

FEI recommends moisture-conditioning and compacting the subgrade prior to paving in accordance with Section 00330.43. The finished subgrade should be proof-rolled with a loaded dump truck or other approved heavy construction vehicle prior to placing the Base Aggregate to identify any soft areas. Any soft or pumping subgrade should be reworked or over excavated and replaced with Base Aggregate.

The Draft Pavement Design Memorandum is included in Appendix D.

BRIDGE DESIGN

Layout and Geometry

The bridge span length was determined by the length of the repurposed box beams the County was able to procure from ODOT. The north abutment was located as close as practical to the north edge of the railroad ROW boundary without the need to impact it during construction. This leaves approximately 40 feet for the realigned Dunawi Creek to flow between the south railroad ROW and the south abutment.

Substructure

Based on the explored subsurface conditions and the preferred bridge design alternative, the abutments will be supported on deep foundations. Driven steel piles will be installed through CMP sleeves in the MSE fills beneath the bridge abutments. By constructing the tall approach fills before driving the piles, much of the settlement will occur prior to pile driving. This reduces the axial forces on the pile due to down drag, allowing slightly shorter piles. Total pile lengths are estimated to be approximately 45 feet.

Bridge End Panels

Bridge end panels are necessary for this structure, even though it has a relatively low Average Daily Traffic count. Given the tall approach fills on either end of the bridge, a small amount of settlement is unavoidable. The 30-foot end panels will assist with transitioning from the slightly settled fill to the rigid bridge abutment. They also help to reduce the traffic impact force and extend the life of the structure.

RAILROAD COORDINATION

Coordination with PNWR has been ongoing throughout this design phase. DEA and County project leaders attended a meeting with PNWR personnel at their office in Salem, OR on August 10th, 2015. At this meeting, the possibility of re-routing Dunawi Creek beneath the existing timber trestle was presented. PNWR did not give approval, but it was discussed that the stream could likely be rerouted beneath the trestle, assuming the center, in-water pier, would be strengthened to mitigate any scour potential. In addition, all work related to the existing railroad bridge would have to be reviewed and approved by the Union Pacific Railroad headquarters in Omaha, NE. Repair of the trestle is outside the scope of this project, but will need to be coordinated in the future, before proceeding with construction of the stream realignment.

CONSTRUCTION SCHEDULE

As mentioned above, the design utilizes a new offset alignment that will allow the new bridge and approach roadways to be constructed without traffic or railroad impacts. The bridge foundations will be constructed in a manner that minimizes any settlement after the piles are installed.

As the project will involve impacts to a regulated waterway, care will need to be taken to perform in water work during the In Water Work Window, which is July 1 through October 15. Most of the work for the project can be done without impacting the waterway. Only the installation of the temporary culvert and the transfer of water to the new streambed will involve in-water work. Since funds for construction have not been procured to date, a construction time period has not been determined. Thus, a detailed construction schedule has not been developed for this phase of the project.

RIGHT OF WAY

This project will require new right-of-way to be purchased by the County. The area north of the railroad has already been established as a permanent easement by the County during an earlier phase of the project. South of the new overcrossing, right-of-way will be purchased to accommodate the new bridge and approach roadway fill slopes. This should only impact one property owner, located southeast of the existing undercrossing. Discussions between the property owner and the County are ongoing.

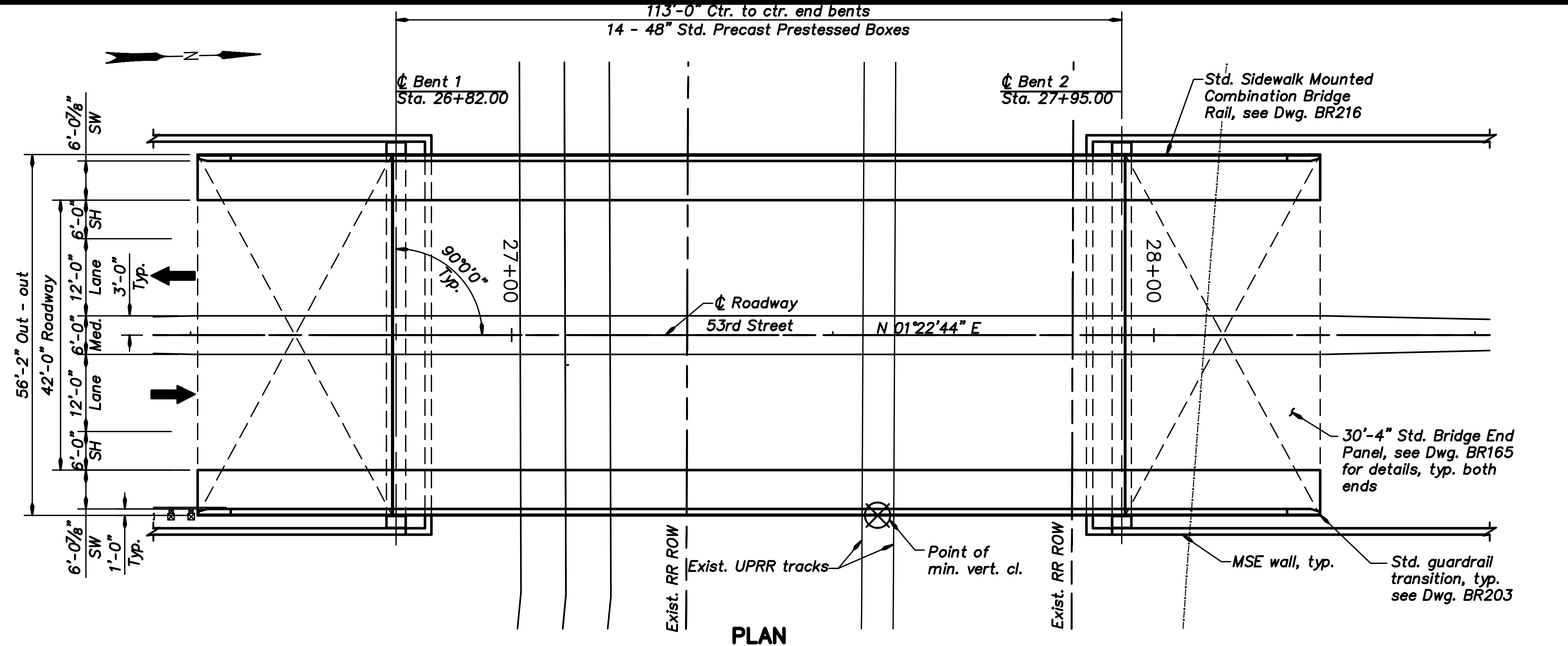
BRIDGE COST ESTIMATE

The estimated cost of construction for the proposed overcrossing bridge and MSE walls is $2,493,026 which includes 30% for contingencies. These costs are for the bridge and retaining wall items only, and do not include erosion control, traffic control, roadway construction, paving, stream restoration, right-of-way, utility relocation or other related items.

Of the total bridge cost above, about half of the costs are associated with the MSE walls. These types of walls are very cost effective, but can vary in price, depending on the type of wall system used, and the availability of quality fill rock close to the project site. The DAP phase construction cost used to estimate the MSE wall costs are from a recent project bid opening that included large MSE walls that averaged $80/sq. ft.

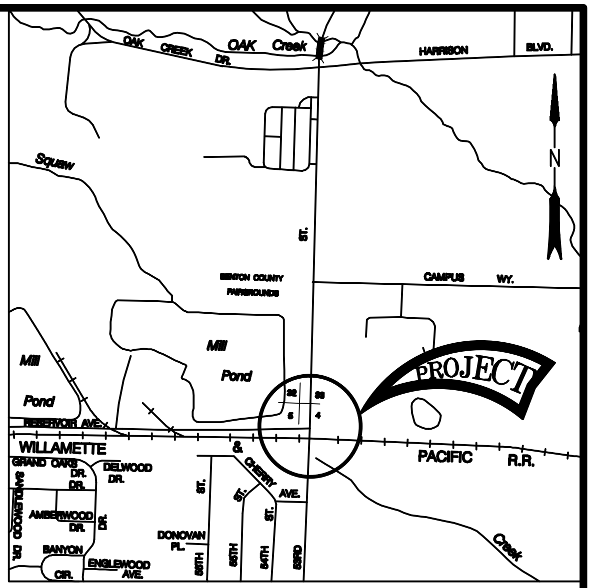
See Appendix B for a breakdown of costs for the new bridge.

Appendix A: Concept Bridge Plans



PLAN

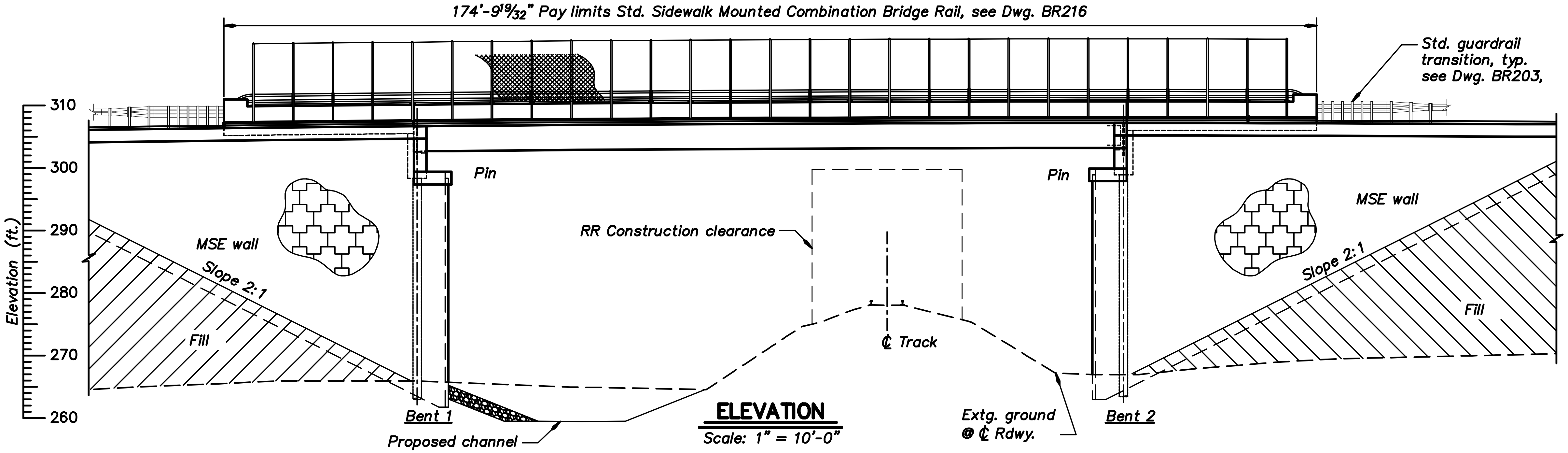
Scale : 1" = 10'-0"



NW 1/4 SEC. 4, T. 12 S., R. 5 W., W.M.

LOCATION MAP

No Scale



ELEVATION

Scale: 1" = 10'-0"

Xrefs: County Border.dwg EB-BCOX0012-LM.dwg 53RD STREET-DEA TRANSMITTAL-10-12-2015.dwg EB-BCOX0012-MASTER.dwg 53RD STREET-DEA TRANSMITTAL-9-30-2015-CHANNEL ALIGNMENT-2.dwg Topo 2-2-15 53rd St. Overpass.dwg 53RD STREET-DEA TRANSMITTAL-9-22-2015.dwg

△	DATE	REVISION	BY

ACCOMPANIED BY DWGS. TSL01-TSL02

DRAFTER: Jim Culpepper

DESIGNER: Anthony Calcagno

CHECKER:

REVIEWER:

EXPIRES:

Benton County
Public Works Department
360 SW Avery Avenue
Corvallis, Oregon 97333
Phone: 541-766-6821
FAX: 541-766-6891

DAVID EVANS AND ASSOCIATES INC.
530 Center Street N.E., Suite 605
Salem Oregon 97301
Phone: 503.361.8635

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

STRUCTURE NO.

DATE
October 2015

CALC. BOOK

53RD STREET OVER WILLAMETTE PACIFIC RAILROAD
53RD STREET
BENTON COUNTY

PLAN AND ELEVATION

SHEET 1 **OF** X

DRAWING NO. TSL01

GENERAL NOTES

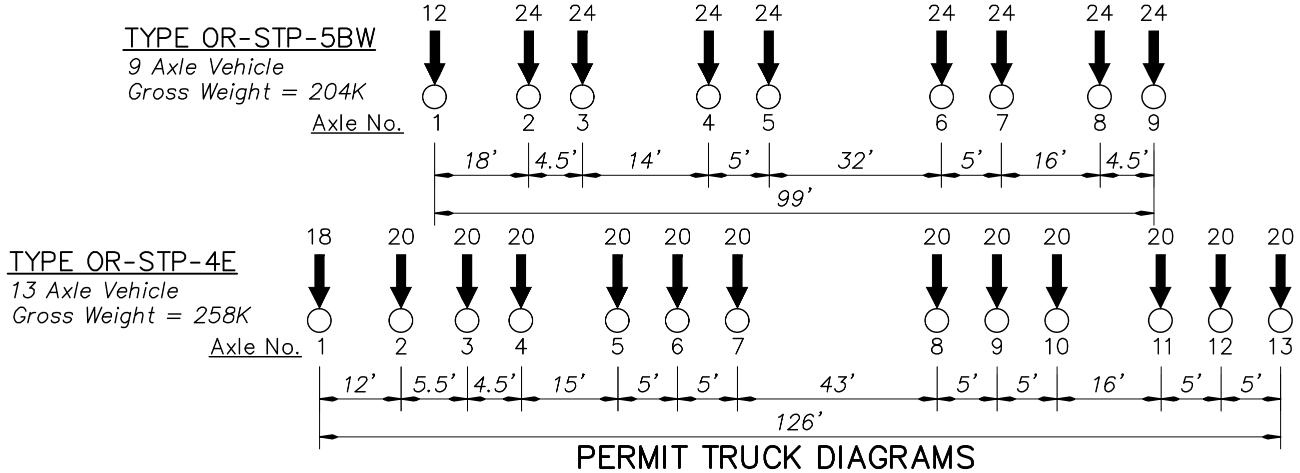
Provide all materials and perform all work according to the 2015 Oregon Standard Specifications for Construction.

Bridge is designed in accordance with the 2012 AASHTO LRFD Bridge Design Specifications (including interm revisions) and the 2014 ODOT Bridge Design & Drafting Manual (BDDM), updated October 2014.

Design includes an allowance of 50 psf for present wearing surface and 25 psf for future wearing surface and the following Live Loads according to the 2012 AASHTO LRFD Bridge Design Specifications (including 2013 interim revisions).

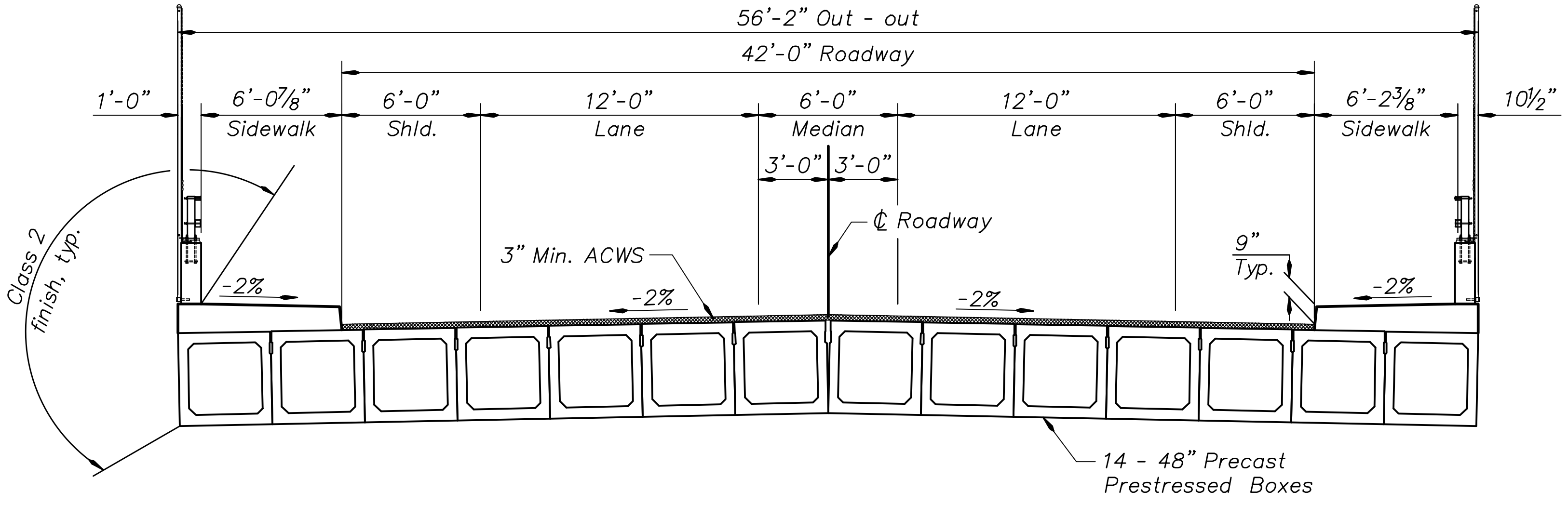
Service and Strength-I Limit States:
HL-93: Design truck (or trucks per LRFD 3.6.1.3) or the design tandems and the design lane load.

Strength-II Limit States:
ODOT Type STP-5BW Permit truck
ODOT Type STP-4E Permit truck



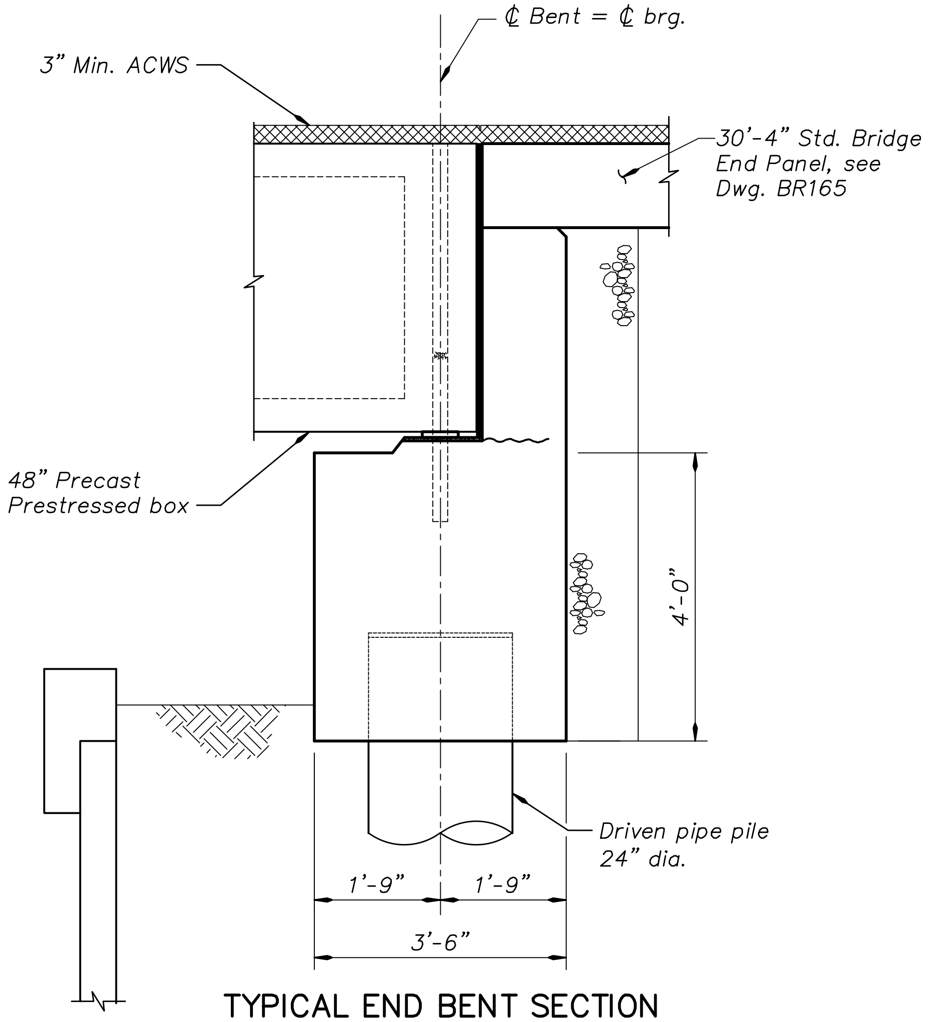
PERMIT TRUCK DIAGRAMS

No Scale



TYPICAL DECK SECTION

Scale: 1/4" = 1'-0"



TYPICAL END BENT SECTION

Scale: 3/4" = 1'-0"

△	DATE	REVISION	BY

ACCOMPANIED BY DWGS. TSL01-TSL02

DRAFTER: Jim Culpepper
DESIGNER: Anthony Calcagno
CHECKER:
REVIEWER:
EXPIRES:

Benton County
Public Works Department
360 SW Avery Avenue
Corvallis, Oregon 97333
Phone: 541-766-6821
FAX: 541-766-6891

DAVID EVANS AND ASSOCIATES INC.
530 Center Street N.E., Suite 605
Salem Oregon 97301
Phone: 503.361.8635

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STRUCTURE NO.

DATE
October 2015

CALC. BOOK

53RD STREET OVER WILLAMETTE PACIFIC RAILROAD
53RD STREET
BENTON COUNTY

GENERAL NOTES AND TYPICAL SECTIONS

SHEET 2 **OF** X

DRAWING NO. TSL02

Xrefs: County Border.dwg EB-BCOX0012-MASTER.dwg

Appendix B: Bridge Cost Estimate

BRIDGE DESIGN SECTION

Type, Size, and Location Estimate Sheet

Bridge Name: SW 53rd St. Railroad Crossing
Bridge No. N/A
Station: N/A
Alternative: Preffered
Loading N/A
Highway: SW 53rd Street
M.P. N/A
County: Benton
Description: New Railroad Overcrossing of SW 53rd Street
Estimate Created By: A. Walker
Design Date: 1/25/2016
Calc. Book No. N/A
Estimate Checker: A. Calcagno,
Check Date: 1/27/2016
Calc. Book No. N/A
Estimate From: Sketch ☐ Plans ■
Plans, Dwgs. Nos. N/A

Cost Summary

ITEMS	UNIT	QUANTITY		COST	
		Substructure	Superstructure	$/Unit	Amount
48" BOX, Prestressed Girders	FOOT		1596	$310	$494,760
Furnish Pile Driving Equipment	LS	1		$40,000	$40,000
Drive PP24x1.0	EACH	16		$1,500	$24,000
Furnish PP24x1.0	FOOT	672		$138	$92,736
Pile Isolation Material (Pea Gravel)	CUYD	44		$95	$4,178
General Structural Concrete - 3300 psi	CUYD		56	$500	$27,923
General Structural Concrete - 4000 psi	CUYD	79		$700	$55,404
Structure Reinforcement	LB	15830		$1.15	$18,204
Granular Wall Backfill for Pile Caps	CUYD	30		$95	$2,857
Class 50 Riprap	CUYD	175		$80	$14,000
Bridge Combination Rail with Protective Fencing	FOOT		311	$200	$62,240
Bridge End Panel	SQYD		253	$250	$63,258
ACWS	TON	198		$100	$19,752
SUBTOTAL - BRIDGE ITEMS					**$919,312**
Retaining Wall (MSE)	SQFT	12480		$80	$998,400
SUBTOTAL - BRIDGE AND WALLS					**$1,917,712**
CONTINGENCY - 30%					**$575,314**
GRAND TOTAL					**$2,493,026**

Appendix C: Geotechnical Report

Foundation Report

SW 53rd Street Railroad Crossing
Benton County, Oregon

Prepared for:

David Evans and Associates, Inc.
Salem, Oregon

October 19, 2015

Professional
Geotechnical
Services

Foundation Engineering, Inc.

Foundation Engineering, Inc.
Professional Geotechnical Services

Anthony Calcagno, P.E.
Bridge Engineer
David Evans and Associates, Inc.
530 Center Street, Suite 605
Salem, Oregon 97301

October 19, 2015

SW 53rd Street Railroad Crossing
Foundation Report
Benton County, Oregon

Project 2141009

Dear Mr. Calcagno:

We have completed the requested geotechnical investigation for the above-referenced project. Our report includes a description of our work, a discussion of the site conditions, a summary of laboratory testing, and a discussion of engineering analyses. Recommendations are included for site preparation and the design of bridge foundations, MSE walls, and approach embankments.

This report was prepared to conform to the ODOT Geotechnical Design Manual (ODOT GDM (November 2014)). Our recommendations refer to sections in the Oregon Standard Specifications for Construction (2015).

It has been a pleasure assisting you with this phase of your project. Please do not hesitate to contact us if you have any questions or require further assistance.

Sincerely,

FOUNDATION ENGINEERING, INC.

Jonathan C Huffman
Jonathan C. Huffman, P.E., G.E.
Senior Project Engineer

David L. Running, P.E., G.E.
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER
62647PE
OREGON
NOV. 9, 1999
DAVID L. RUNNING
Expires: 12/31/16

JCH/DLR/wg
enclosure

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FOUNDATION REPORT
SW 53RD STREET RAILROAD CROSSING
BENTON COUNTY, OREGON

1.0. INTRODUCTION

1.1. *Project Description*

A new bridge is planned crossing over the Portland & Western Railroad (PNWR) mainline tracks at SW 53rd Street in Corvallis, Oregon. The site location is shown on Figure 1A (Appendix A).

SW 53rd Street currently crosses under the railroad tracks, which are supported on a timber trestle bridge. For the new crossing, the street will be shifted to the east of its current alignment and will cross over the tracks. At the crossing, the railroad tracks are laid on an embankment elevated ±10 feet above the surrounding terrain. New approach embankments up to ±44 feet tall and ±1,100 to 1,800 feet long will be required to raise the street above the existing track. Mechanically Stabilized Earth (MSE) retaining walls up to ±44 feet tall are planned to retain the approach fill at the abutments. The MSE walls will extend ±85 feet back from the abutments along the sides of the approaches parallel to the street. The new bridge will be a 60.9-foot wide by 113-foot long, single-span concrete structure.

Benton County is the project owner and David Evans and Associates, Inc. (DEA) is the prime designer. Foundation Engineering, Inc. was retained by DEA as the geotechnical consultant.

1.2. *Purpose and Scope*

The purpose of the geotechnical investigation was to develop recommendations for the design and construction of the bridge foundations, approach embankments, MSE retaining walls, and approach pavements. The scope of the geotechnical work included exploratory drilling, laboratory testing, engineering analyses and preparation of this report. Design and construction recommendations for the approach pavements will be provided in a separate memorandum.

1.3. *Literature Search*

Prior to the field investigation, we reviewed available literature to provide a general overview of the site geology and select drilling depths for the exploration program. Reviewed information included geologic maps, reports, and local water well logs available from the Oregon Water Resources Department website. Information from our previous investigations in the area was also reviewed.

2.0. GEOLOGY AND FAULTING

2.1. *Local Geology*

Corvallis is located between the western edge of the central Willamette Valley and the eastern foothills of the Coast Range. The City is set on gently sloping foothills and a broad, flat terrace adjacent to the Willamette River. This setting has created a variety of geologic terrains beneath the City. Fluvial and lacustrine sediments (Quaternary alluvial terrace deposits) underlie the lower-lying areas, including downtown Corvallis, the Oregon State University (OSU) campus, and the SW 53r Street crossing site (Bela, 1979; Yeats et al., 1996; O'Connor et al., 2001; Wiley, 2008). The alluvial sediments thin toward exposures of older, well-indurated sedimentary rock (Eocene Spencer and Flournoy Formations) in the low hills to the south and west.

Our explorations indicate the project site is underlain by alluvium including a thin mantle of Willamette Silt, followed by sandy silt, silty sand, silty gravel and silt. The soil profiles encountered in our explorations are consistent with the mapped local geology. Based on review of local well logs, it is anticipated the depth to bedrock exceeds ±100 feet in this area.

2.2. *Local and Regional Faults*

A review of nearby faults was completed to evaluate the seismic setting and the seismic sources. Numerous concealed and inferred crustal faults are located within ±10 miles of Corvallis (Bela, 1979; Yeats et al., 1996; Wiley, 2008; McClaughry et al., 2010). However, none of these faults show any evidence of movement in the last ±1.6 million years except for the Owl Creek fault (Geomatrix Consultants, 1995; USGS, 2006). Six potentially active Quaternary (<1.6 million years or less) crustal fault zones have been mapped within ±40 miles of the site (Geomatrix Consultants, 1995; Personius et al., 2003; USGS, 2006). These fault zones are listed in Table 1. Additional fault information can be found in the literature (Personius et al., 2003; USGS, 2006).

Table 1. Potentially Active Quaternary Crustal Faults within ±40 miles of Corvallis

Fault Name	Length (miles)	Last Known Activity	Distance from Site (miles)	Slip Rate (mm/yr)
Corvallis (#869)	±25	<1.6 million years	±0.5 NW	<0.20
Owl (#870)	±9	<750,000 years	±5 E	<0.20
Mill Creek (#871)	±11	<1.6 million years	±18 NE	<0.20
Waldo Hills (#872)	±8	<1.6 million years	±24 NE	<0.20
Yaquina (#885)	±8	<130,000 years	±35 W-NW	0.60*
Cape Foulweather (#884)	±6	<130,000 years	±36 NW	<0.20
Waldport (#886)	±9	<130,000 years	±37 SW	0.14*

Note: Fault data based on USGS (2006) and Personius et al. (2003). *From Table H-1 (Petersen et al., 2008).

All but the Corvallis fault are considered Class A faults by the United States Geologic Survey (USGS). A Class A fault is a fault with geologic evidence supporting tectonic movement in the Quaternary, known or presumed to be associated with large-magnitude earthquakes.

Although there are several crustal faults in the area, the USGS 2002 interactive deaggregation indicates the primary seismic source affecting the site is the Cascadia Subduction Zone (CSZ). The CSZ is a converging, oblique plate boundary where the Juan de Fuca plate is being subducted beneath the western edge of the North American continent (Geomatrix Consultants, 1995). The CSZ extends from central Vancouver Island in British Columbia, Canada, through Washington and Oregon to Northern California (Atwater, 1970).

Available information indicates the CSZ is capable of generating earthquakes within the descending Juan de Fuca plate (intraplate), along the inclined interface between the two plates (interface or subduction zone), or within the overriding North American Plate (crustal) (Weaver and Shedlock, 1996). The Oregon Department of Geology and Mineral Industries (DOGAMI) estimates the maximum magnitude of an interface subduction zone earthquake ranges from moment magnitude (M_w) 8.5 to M_w 9.0 (Wang and Leonard, 1996; Wang et al., 1998; Wang et al., 2001), and the rupture may potentially occur along the entire length of the CSZ (Weaver and Shedlock, 1996).

3.0. SUBSURFACE EXPLORATION AND CONDITIONS

3.1. ***Exploration***

3.1.1. *Borings*. Five exploratory boreholes were drilled at the site between October 27 and 29, 2014. BH-1 and BH-2 were drilled along the north approach and BH-5 was drilled on the south approach. These borings provide subsurface information for the design of the new approach embankments. BH-3 was drilled near the north abutment and BH-4 was drilled near the south abutment. These borings provide subsurface information for the design of the bridge foundations and MSE walls. The borehole locations are shown on Figure 2A (Appendix A). The locations were surveyed by Benton County.

The borings were drilled using a CME 55, track-mounted drill rig with mud-rotary drilling techniques. BH-1, BH-2, and BH-5 extended to depths of ±16 to 21.5 feet. BH-3 and BH-4 both extended to ±80.9 feet. Disturbed samples were obtained in conjunction with the Standard Penetration Test (SPT), typically at ±2.5-foot intervals to a depth of ±15 feet and at ±5-foot intervals thereafter. Relatively undisturbed soil samples were obtained at select intervals using a thin-walled Shelby tube sampler.

The boreholes were continually logged during drilling. The final logs (Appendix B) were prepared based on a review of the field logs, an examination of the soil samples in our office, and the laboratory test results. The subsurface conditions are discussed below.

3.1.2. *Test Pits*. Four exploratory test pits were dug at the site on November 7, 2014, to supplement the borings and provide additional subsurface information for the design of the approach embankments. TP-1 and TP-2 were dug between BH-1 and BH-2, and TP-3 and TP-4 were dug south of BH-5. The approximate locations are shown on Figure 2A (Appendix A). The test pit locations are shown on Figure 2A (Appendix A). The locations were surveyed by Benton County.

The test pits were dug using a rubber-tired backhoe and extended to depths of ±9 to 11.5 feet. Disturbed soil samples were obtained for laboratory testing. Undrained shear strength measurements were completed on the test pit sidewalls using a field vane shear device. The test pits were logged and the soil profiles, sampling depths, and strength measurements are summarized in the appended test pit logs (Appendix B). The observed subsurface conditions are discussed below.

3.2. ***North Approach***

BH-1, TP-1, TP-2 and BH-2 were completed to investigate the subsurface conditions for the north approach. BH-1 was completed on top of an existing fill stockpile to characterize the quality of the stockpiled material and evaluate the suitability of this material for reuse in the new embankment. A shallow trench was also dug down the south slope of the stockpile to help evaluate the material. The remaining explorations were completed outside the stockpile area, operating from the original ground surface.

The following provides a narrative of the subsurface conditions observed in these explorations. More detailed subsurface information is provided on the boring and test pit logs included in Appendix B.

3.2.1. *BH-1*. The fill stockpile extends ±10 feet above the adjacent ground surface at BH-1. Drilling at BH-1 began at ±El. 283.3 and encountered fill consisting of medium dense sandy gravel with some silt to ±7.5 feet, followed by grey, soft to medium stiff, low plasticity silt to ±10 feet.

The fill in BH-1 is underlain by light brown, iron-stained, medium stiff to stiff, medium plasticity clayey silt (Willamette Silt) to ±12.5 feet, followed by brown, stiff, medium plasticity clayey silt with trace sand (alluvium) to ±15 feet. Very dense silty gravel with some sand (alluvium) extends below the clayey silt from ±15 feet (±El. 268.3) to ±21.5 feet (the bottom of the boring).

3.2.2. *Stockpile Trench*. The trench dug on the south slope of the fill stockpile extended ±2.5 to 3 feet below the surface from the top of the slope to the bottom. This exploration encountered dark brown, stiff, low to medium plasticity silt with trace to some sand and gravel in the upper ±3 to 4 feet of stockpile. Light brown, iron-stained, stiff, medium plasticity clayey silt with trace to some gravel followed to the bottom of the stockpile.

3.2.3. *TP-1*. Digging at TP-1 began at ±El. 275.6 and encountered brown, medium stiff, low to medium plasticity silt with scattered organics (topsoil) to ±12 inches. The topsoil is underlain by light brown, iron-stained, stiff, medium plasticity clayey silt (Willamette Silt) to ±9 feet. Dense silty gravel with trace sand and scattered cobbles follows from ±9 feet (±El. 266.6) to ±10.5 feet (the bottom of the test pit).

3.2.4. *TP-2*. Digging at TP-2 began at ±El. 273.8 and encountered brown, medium stiff, low to medium plasticity silt with scattered organics (topsoil) to ±12 inches. The topsoil is underlain by light brown, iron-stained, stiff to very stiff, medium plasticity clayey silt (Willamette Silt) to ±9 feet, followed by brown, stiff, medium plasticity clayey silt with trace to some sand and gravel (alluvium) to ±10.5 feet. Very dense silty gravel with some sand (alluvium) extends below the clayey silt from ±10.5 feet (±El. 263.3) to ±11.5 feet (the bottom of the test pit).

3.2.5. *BH-2*. Drilling at BH-2 began at ±El. 273.1 and encountered light brown, iron-stained, stiff to very stiff, medium plasticity clayey silt (Willamette Silt) to ±9.5 feet. The Willamette Silt is underlain by brown, stiff, medium plasticity clayey silt with trace sand and gravel (alluvium) to ±11.5 feet. Very dense silty gravel with some sand (alluvium) extends below the clayey silt from ±11.5 feet (±El. 261.6) to ±16 feet (the bottom of the boring).

3.3. ***Bridge Abutments***

BH-3 and BH-4 were drilled to investigate the subsurface conditions at the abutments. The following provides a narrative of the subsurface conditions observed in these borings. More detailed subsurface information is provided on the boring logs included in Appendix B.

3.3.1. *BH-3 – North Abutment.* Drilling at BH-3 began at ±El. 269.5 and encountered light brown, iron-stained, stiff, medium plasticity clayey silt (Willamette Silt) to ±5 feet, followed by brown, iron and manganese-stained, very stiff, low to medium plasticity, clayey silt with trace sand and gravel (alluvium) to ±10 feet. The clayey silt is underlain by brown, iron-stained, very stiff, low plasticity sandy silt to ±15 feet.

Drilling from ±15 feet (±El. 254.5) to 80.9 feet (the bottom of the boring) encountered predominantly very dense silty gravel with some sand. Two, ±4.5 to 5-foot thick layers of blue-grey, hard, low plasticity silt with trace sand were encountered within the gravel stratum from ±39 to 43.5 feet and ±72 to 77 feet.

3.3.2. *BH-4 – South Abutment.* Drilling at BH-4 began at ±El. 266.0 and encountered light brown, iron-stained, stiff, medium plasticity clayey silt (Willamette Silt) to ±9 feet, followed by grey-brown, medium plasticity clayey silt with trace sand and gravel (alluvium) to ±11 feet. The clayey silt is underlain by medium dense silty sand to ±16.5 feet.

Drilling from ±16.5 feet (±El. 249.5) to 80.9 feet (the bottom of the boring) encountered predominantly very dense silty gravel with some sand. Two, ±3 to 4-foot thick layers of blue-grey to grey, very stiff to hard, low plasticity silt with trace to some sand were encountered within the gravel stratum form ±43 to 46 feet and ±72.5 to 76.5 feet.

3.4. ***South Approach***

BH-5, and TP-3 and TP-4 were completed to investigate the subsurface conditions for the south approach. The following provides a narrative of the subsurface conditions observed in these explorations. More detailed subsurface information is provided on the boring and test pit logs included in Appendix B.

3.4.1. *BH-5*. Drilling at BH-5 began at ±El. 263.4 and encountered grey-brown, iron-stained, stiff to very stiff, medium plasticity clayey silt (Willamette Silt) to ±8 feet. Very dense silty gravel with some sand follows from ±8 feet (±El. 255.4) to ±20.3 feet (the bottom of the boring).

3.4.2. *TP-3*. Digging at TP-3 began at ±El. 262.7 and encountered brown, medium stiff, low to medium plasticity clayey silt with trace gravel and scattered organics (topsoil) to ±8 inches. The topsoil is underlain by grey-brown, iron-stained, stiff to very stiff, medium plasticity clayey silt (Willamette Silt) to ±9 feet (the bottom of the test pit). A 15-inch diameter PVC sewer line was encountered at the south edge of the test pit at a depth of ±9 feet.

3.4.3. *TP-4*. Digging at TP-4 began at ±El. 268.1 and encountered brown, medium stiff to stiff, low to medium plasticity silt with scattered organics (topsoil) to ±12 inches. The topsoil is underlain by light brown, iron-stained, stiff to very stiff, medium plasticity clayey silt (Willamette Silt) to ±10 feet. Dense to very dense silty gravel with some sand (alluvium) follows from ±10 feet (±El. 258.1) to ±11.5 feet (the bottom of the test pit).

3.5. ***Ground Water***

Mud-rotary drilling techniques precluded an accurate determination of the ground water levels in the borings at the time of drilling. No ground water seepage was observed in the exploratory test pits, which extended to depths of ±9 to 11.5 feet. Well log information from the project vicinity indicates static ground water depths ranging from ±12 to 20 feet below the ground surface. However, the observed iron-staining in the surficial soils suggests water perches on the site and the local ground water may rise within a few feet of the ground surface in the lower-lying areas during the wet winter and spring months.

4.0. LABORATORY TESTING

Natural water contents, Atterberg limits and percent fines tests were completed on selected soil samples to classify the soils and estimate their overall engineering properties. The results of these tests are summarized on Table 1C (Appendix C).

Two, one-dimensional consolidation tests were also run on relatively undisturbed samples obtained in the upper ±10 feet of BH-2 and BH-4. These tests were run to evaluate the compressibility of the fine-grained soil beneath the new approach embankments. The consolidation curves are shown on Figures 1C and 2C (Appendix C).

5.0. SEISMIC ANALYSIS AND EVALUATION

5.1. *Bedrock Acceleration and Site Response*

The ODOT GDM (2014) recommends all bridge structures be designed using "serviceable" and "no collapse" seismic performance criteria for earthquake ground motions having a 500 and 1,000-year average return period, respectively. Response spectra for the site were established using the General Procedure in the AASHTO LRFD Bridge Design Specifications (2014) and seismic design maps based on the USGS National Seismic Hazard Maps (2002).

The average subsurface conditions across the site correspond to an AASHTO 2012 Site Class D. The AASHTO General Procedure Response Spectra established for a Site Class D are shown on Figure 3A (Appendix A).

5.2. *Liquefaction, Settlement and Lateral Spread*

Liquefiable soils typically consist of saturated, loose sand and non-plastic silt. The site is underlain by predominantly very dense silty gravel with layers of stiff to very stiff clayey silt, hard silt, very stiff sandy silt and medium dense to dense silty sand. The soils encountered in our explorations are not considered susceptible to liquefaction due to the stiffness and plasticity of the clayey and silty soils and the density of the sand and gravel. Therefore, the risk of liquefaction-induced settlement and lateral spread is considered negligible.

6.0. FOUNDATION ANALYSIS AND DESIGN RECOMMENDATIONS

6.1. *Bridge Foundation Options and Discussion*

Shallow foundations are not practical to support the new bridge due to the required tall approach embankments and the risk of abutment settlement. Therefore, deep foundations (drilled shafts or driven piles) will be required. Drilled shafts are typically more expensive than driven piles and would be more difficult to install. Therefore, driven piles are preferred. Driven piles should be able to attain relatively high axial resistances with modest embedment into the very dense silty gravel encountered at depths of ±7.5 to 12.5 feet below the base of the planned MSE walls.

We recommend constructing the MSE walls with corrugated metal pipe (CMP) sleeves installed in the wall backfill at the pile locations. This approach will allow the piles to be driven after the MSE walls and approach embankments are constructed, thereby reducing or eliminating the downdrag forces on the piles caused by the settlement of the soil beneath the wall.

Steel pipe piles or H-piles could be used. Pipe piles are preferred because they will attain the required axial resistance with less penetration relative to H-piles. Pipe piles will also provide symmetric lateral resistance. PP16x0.5 and PP24x0.5 piles were considered. PP24x0.5 (ASTM A-252 Grade 3 steel) piles were selected based on the design loads, the soil conditions and the need to support the abutments on a single row of piles.

A CMP sleeve with a 30-inch inside diameter can be used with the 24-inch diameter piles. The annulus between the piles and CMP sleeves should be backfilled with pea gravel to allow post-construction settlement of the walls (if any) to occur without mobilizing downdrag loads on the piles within the wall backfill zone. The pea gravel will also provide flexibility to help accommodate relative lateral movement between the piles and MSE walls.

6.2. *Estimated Foundation Loads*

Table 2 summarizes the Service I and Strength I (factored) loads DEA provided for each abutment.

Table 2. Design Foundation Loads per Abutment

Load Case	Dead Load (kips)	Live Load (kips)	Total (kips)
Service I	1,490	840	2,330
Strength I	1,910	1,470	3,380

We calculated a required factored axial load of 422.5 kips per pile, assuming eight (8) piles will support each abutment and the loads are evenly distributed between the piles.

6.3. ***Driven Pile Analysis and Design***

Axial pile analysis was completed using the AASHTO (2014) Load Resistance Factor Design (LRFD) approach. The design criteria are presented in the following subsections. The calculations will be included in Appendix D of the final Foundation Report.

6.3.1. *Pile Type and Material Specifications*. PP24x0.5 piles were selected to support the abutments. We recommend driving the piles closed-ended to limit the required embedment depths. Inside-fitting conical tips are recommended to facilitate driving through the upper, stiff fine-grained soils and maintaining pile alignment. The recommended pile properties are summarized in Table 3.

Table 3. Recommended Pile Properties

Pile Properties	PP24x0.5
Steel Grade	ASTM A252 (Grade 3)
Yield Stress (F_y)	45 ksi
Area Steel (A_s)	36.9 in^2
Nom. Structural Resistance (F_y x A_s)	1,660 kips
End Condition	Closed-ended with inside-fitting conical tip

6.3.2. *Downdrag*. At least ± ½ inch of ground settlement around the pile is typically required to induce downdrag loads on deep foundations following their installation. We estimated settlements on the order of ± 2 inches at the abutments due to the weight of the new embankment fill. However, most of this settlement is expected to occur during construction of the embankments and MSE walls. Furthermore, our analysis indicates less than ½ inch of abutment settlement will remain within ± 2 weeks of completing the embankments and MSE wall construction. Therefore, we have assumed pile installation can accommodate this schedule and downdrag will not be an issue. Additional discussion of the embankment settlement is provided in a subsequent section of this report.

6.3.3. *Nominal and Factored Axial Resistances*. The nominal and factored axial resistances were estimated from soil profiles interpolated based on BH-3 and BH-4. Strength parameters for the foundation soils were estimated based on correlations to SPT N-values and field vane measurements. The nominal axial resistance is based on skin friction along the length of the driven pile and end-bearing at the pile tip.

The factored resistances are based on an AASHTO LRFD resistance factor (ϕ) of 0.5, assuming Wave Equation analysis will be used to establish the final driving criteria per Section 00520.20(d) of the ODOT Standard

Specifications for Construction (2015). Nominal and factored axial resistances are plotted versus embedment on Figures 4A and 5A (Appendix A).

6.3.4. *Minimum/Estimated Tip Elevations and Pile Lengths*. A nominal driving resistance of ±845 kips per pile is required based on the factored (Strength I) load and a resistance factor of 0.5. The analysis indicates closed-ended PP24x0.5 piles should attain the required driving resistance with shallow penetration below the top of the very dense silty gravel stratum. We anticipate some variation in the depth and the density of the gravel across the width of the new bridge abutments. Therefore, estimated tip elevations correspond to ±10 feet of embedment into the gravel stratum. Minimum tip elevations correspond to the assumed surface of the gravel stratum.

We estimated the ground surface elevations and bottom of pile cap elevations at each abutment from preliminary drawings provided by DEA. Pile cut-off elevations were estimated assuming ±1.5 feet of embedment into the pile caps. Table 4 provides a summary of the minimum and estimated tip elevations and corresponding pile lengths.

Table 4. Minimum/Estimated Tip Elevations and Pile Lengths

Bent	Nominal Axial Resistance/Pile (kips)	[1]Est. Cut-Off Elevation (ft)	Min. Tip Elevation (ft)	Est. Tip Elevation (ft)	Finished Pile Length (ft)
1	845	299.5	250.5	240.5	60
2	845	301.0	254.0	243.0	60

Notes:

1. Pile cut-off elevations assume 1.5 feet of embedment into the pile cap.
2. Minimum tip elevations correspond to the estimated surface of the very dense silty gravel.
3. Estimated tip elevations correspond to ±10 feet of embedment below the silty gravel surface.
4. Finished pile lengths based on estimated tip and cut-off elevations rounded up to the nearest 5-foot interval. These lengths do not include stickup for pile driving.

6.3.5. *Nominal and Factored Uplift Resistance*. The nominal uplift resistance for the PP24x0.5 piles was calculated based on the estimated skin resistance mobilized in the soil above the minimum tip elevations. We estimate the nominal uplift resistances to be ±87 kips per pile at Bent 1 and ±54 kips per pile Bent 2. Factored uplift resistances for extreme event loading were calculated using an AASHTO ϕ factor of 0.8. The factored uplift resistances are ±70 kips for Bent 1 and ±43 kips for Bent 2.

6.3.6. *Pile Settlement*. The pile tips will be seated in very dense silty gravel, which has low compressibility characteristics. Therefore, pile settlement is expected to be limited to the elastic compression of the section caused by the working load. We anticipate the pile settlement will be less than ± ¼ inch.

6.3.7. *Lateral Analysis*. Because the bridge will be a single-span structure, lateral analysis for the piles was not completed. It is assumed lateral loads will be resisted predominantly by the abutments.

6.3.8. *Driving Criteria and Driveability Analysis*. The Wave Equation Analysis Program (WEAP 2005) was used to establish a range of hammer field energies required to drive the PP24x0.5 piles to a nominal axial resistance of ±845 kips with a final driving resistance in the range of 2 to 10 blows per inch. Analysis completed using a range of pile hammers indicates a rated hammer field energy range of ±90 to 120 foot-kips is required. Input parameters used in the analysis are summarized in Table 1A (Appendix A).

6.3.9. *Potential Obstructions*. We observed no potential obstructions for pile driving in the bridge borings. However, hard driving conditions should be expected once the pile tips reach the very dense silty gravel stratum. Preboring should not be required and jetting is not recommended.

6.3.10. *Set Period and Redriving*. The piles will be driven into very dense silty gravel. Excess pore pressures are expected to dissipate relatively quickly. In the event the required resistance is not attained at the estimated tip elevations, the contractor should stop driving and allow the piles to set for a period of 24 hours before redriving.

6.3.11. *Tip Protection*. The PP24x0.5 piles should be equipped with inside-fitting conical tips to provide a closed-ended condition while facilitating driving through the upper, stiff fine-grained soils and maintaining pile alignment.

7.0. PAVEMENTS

Design and construction recommendations for the approach pavements will be provided in a separate memorandum.

8.0. APPROACHES AND EMBANKMENTS

New approach embankments will be required to realign and raise SW 53rd Street to cross over the railroad tracks. The north approach will extend ±1,100 feet north of the tracks to the intersection of SW Reservoir Road. The south approach will extend ±1,800 feet south of the tracks to ±500 feet south of the intersection with SW Willow Avenue.

The tallest portion of the embankment (at the bridge abutments) will include MSE retaining walls up to ±44 feet tall. The MSE walls will extend ±85 feet back from the abutments along the sides of the approaches, parallel to SW 53rd Street. The portion of the embankment that is not retained by MSE walls will have fill slopes. The following includes a discussion of the analysis and design recommendations for the approach embankments. The MSE walls are discussed in Section 9.0.

8.1. *Embankment Stability*

Outside of the MSE walls, the new embankments may be constructed with a range of soils. The soils should meet the requirements of Borrow Material (Section 00330.12). We assumed an internal friction angle (ϕ) of 32 degrees, cohesion of 100 psf, and total unit weight of 125 pcf for our evaluation of the embankment fill. These material parameters assume the fill will be placed and compacted per the specifications. Maximum fill slopes of 2(H):1(V) are recommended for the new embankments.

8.2. *Embankment Settlement*

Most of the settlement beneath the new approaches will occur due to consolidation of the Willamette Silt that was encountered within the upper ±8 to 12 feet of the borings and test pits. The compressibility this material was estimated based on the consolidation test results included in Appendix C. A modified compression index ($C_{C\varepsilon}$) of 0.15 and a recompression index ($C_{R\varepsilon}$) of 0.008 were assumed for the analysis. A preconsolidation pressure of ±5 ksf was also assumed. Elastic compression parameters for the deeper sand and gravel were selected from available literature based on the recorded SPT N-values.

Settlement of the approach embankments was estimated using the computer program Settle^{3D}. The height of the embankment (and resulting change in effective stress) will vary along the length of the new approach, with the tallest portion being near the new bridge abutments. A maximum fill height of ±44 feet was assumed.

The results of our analysis indicate a maximum settlement of ±4 inches, where the new embankment fill is deepest (±44 feet). Correspondingly less settlement should occur for lesser embankment heights. We estimated total settlements of ±2.3 inches for a ±30-foot tall embankment and ±1.9 inches for a ±20-foot tall embankment.

Most of the estimated settlement will likely occur as the embankment is being constructed. Based on the observed thicknesses of the upper fine-grained soils and the consolidation time-rate properties estimated from the laboratory tests, we estimate ±1 to 1.3 inches of settlement will occur post-construction for the tallest portion of the embankment. Most of the post-construction settlement is expected to occur within a few weeks of the completion of the embankment. The settlement calculations will be included in Appendix D of the final Foundation Report.

9.0. MSE WALLS

MSE walls are planned to retain the approach fill at both abutments. The walls will wrap around the bridge abutments and extend ±85 feet back along the sides of the approaches parallel to the street. An MSE wall height of ±33 feet is anticipated in front of the abutments beneath the abutment walls and pile caps. The MSE walls parallel to the street will have a maximum height of ±44 feet along the sides and behind the abutments. These walls will step up the approach embankments and become shorter as they extend back from the bridge abutments.

The MSE walls will be designed using a proprietary system with internal stability analysis and design provided by the manufacturer. Therefore, our work is limited to providing soil parameters for the MSE wall design, and completing external stability checks including; bearing capacity, sliding resistance and overturning resistance, and global stability of the retained fill and slope.

9.1. *Soil Parameters*

MSE Granular Backfill will be used in the reinforced zone. The walls will retain compacted Borrow Fill. It is anticipated the Borrow Fill will include a combination of granular and cohesive fine-grained soils. However, cohesion in the retained soil was ignored when calculating lateral earth pressures for external stability. Table 5 provides recommended strength parameters for these materials.

Table 5. Recommended Soil Parameters for MSE Wall Design

Material	**Moist Unit Weight** γ_m **(pcf)**	**Friction Angle** ϕ **(degrees)**	**Cohesion C (psf)**
Reinforced Soil - MSE Granular Backfill	130	34	0
Retained Soil – Compacted Embankment Fill	125	32	0

The foundation soils are expected to vary along the length of the MSE walls. The tallest portion of the walls (i.e., near the bridge abutments) will be underlain by native alluvium, including ±7.5 to 12.5 feet of stiff clayey silt. Where the walls step up to shorter configurations, they will be supported on embankment fill.

For the foundation evaluation, drained strength values (i.e., c-ϕ' parameters) are recommended for the alluvium and fill since the loading will occur over an extended period of time, allowing pore pressure dissipation as the embankments are constructed. Table 6 provides recommended soil parameters for the foundation soils.

Where the MSE walls are underlain by native alluvium or less than 10 feet of embankment fill, we recommend assuming strength properties consistent with the stiff clayey silt. Where the walls is underlain by at least 10 feet of fill, we recommend assuming strength parameters consistent with the fill.

Table 6. Recommended Foundation Soil Parameters for MSE Wall Design

Wall Location	Material	Moist Unit Weight γ_m (pcf)	Friction Angle ϕ (degrees)	Cohesion C (psf)
Wall constructed on native soil or less than 10 feet of fill	Stiff clayey SILT	114	28	100
Wall constructed on greater than 10 feet of fill	Compacted Embankment Fill	125	32	100

Note: Ground water was assumed at ±El. 262 for bearing capacity analysis.

The MSE walls will be supported on prepared subgrade. The recommended foundation soil parameters assume any soft or disturbed soil encountered beneath the walls will be overexcavated and replaced with Granular Structure Backfill or Stone Embankment Material.

The ground water level was assumed at ±El. 262, corresponding to the approximate base of the walls near the bridge abutments. Therefore, the effective unit weight should be used to calculate bearing resistance, where the wall is constructed on the native soil.

9.2. *LRFD Design Parameters*

External stability analyses were completed using the AASHTO (2014) LRFD approach. Table 7 summarizes the load factors based on AASHTO (2014) Table 3.4.1-1 and 3.4.1-2.

Table 7. Load Factors for External Stability

Condition	Strength I-a (Sliding and Eccentricity)	Strength I-b (Bearing Resistance)	Extreme Event I (Sliding and Eccentricity)	Extreme Event I (Bearing Resistance)
Horizontal Active Earth Pressure, EH	1.5	1.5	1.5	1.5
Vertical Earth Pressure, EV	1.0	1.35	1.0	1.35
Live Load (Traffic) Surcharge, LL	1.75	1.75	γ_{EQ}	γ_{EQ}
Earthquake Loads, EQ	--	--	1.0	1.0

Note: γ_{EQ} is project dependent and is typically 1.0 or less.

Table 8 summarizes the external stability resistance factors (ϕ) based on AASHTO (2014) Table 11.5.6-1.

Table 8. Resistance Factors for External Stability

Condition	Strength	Extreme Event
Sliding Resistance	1.0	1.0
Bearing Resistance	0.65	1.0

9.3. *Nominal and Factored Bearing Resistance*

The nominal bearing resistance (q_n) for the foundation soils was calculated using the strength parameters presented in Table 6 and the bearing capacity equation and tables in FHWA NHI-10-024. The nominal bearing resistance is calculated as:

$$q_n = cN_c + 0.5(L')\gamma N_\gamma$$

where q_n is in units of lb/ft^2, c is the foundation soil cohesion, N_c and N_γ are unitless bearing capacity coefficients, L′ is the effective foundation width accounting for eccentricity ($L' = L-2e$), and γ is the effective unit weight of the foundation soil. The eccentricity varies depending on wall height and loading conditions. Sloping ground conditions were assumed where the walls will step up the embankment and be constructed on new embankment fill. A maximum fill slope of 2(H):1(V) was assumed for walls constructed on fill. It was assumed these walls would have a minimum embedment of 2 feet (i.e., the fill slope will project a minimum of 4 feet horizontally away from the face of the wall).

The factored bearing resistance for static loading is the nominal bearing resistance multiplied by a resistance factor (φ) of 0.65. The calculated factored bearing resistances are summarized in Tables 10 and 11.

9.4. *Wall Settlement*

Settlement of the MSE walls was estimated based on the methods described in Section 8.2. A maximum settlement of ±2 inches is expected at the wall face near the bridge abutments, where the tallest section of the walls will be constructed. The settlement of the fill in the reinforced zone and in the retained fill behind the wall is expected to be up to ±4 inches. Consistent with the overall embankment settlement, most of the wall settlement is expected to occur as the walls are constructed, with less than ½ inch expected post-construction. The settlement calculations will be included in Appendix D of the final Foundation Report.

We recommend the MSE walls be designed to accommodate differential settlement of 100(H):1(V). The wall facing should be constructed to accommodate the differential settlement without cracking or separation of the wall panels.

9.5. ***Lateral Earth Pressures***

Lateral earth pressures for the design of the MSE walls were calculated based on the design practices recommended in the ODOT GDM (2014), FHWA (2009) and AASHTO (2014). Calculations include the effects of lateral earth pressures from the retained fill, traffic surcharge parallel and perpendicular to the walls, and seismic considerations, including inertial seismic forces.

9.5.1. *Static Loading*. We anticipate the MSE walls will deflect sufficiently to mobilize active conditions. Therefore, active earth pressures were assumed. Coulomb analysis was used to calculate the active earth pressures. The wall geometry was used along with an assumed internal friction angle of 32 degrees for the retained soil to calculate an active Earth Pressure Coefficient (k_a) of 0.31. Any cohesion in the retained soil was ignored. The active earth pressure was calculated as an equivalent fluid density of 38 pcf, assuming a unit weight of 125 pcf for the retained soil.

9.5.2. *Seismic Loading*. The ODOT GDM (2014) requires walls be designed for a peak horizontal acceleration corresponding to a 1,000-year return period. The USGS 2002 map indicates a peak bedrock acceleration of 0.24g, for the 1,000-year design earthquake. An AASHTO F_{pga} value of 1.32 for Site Class D was used to calculate a peak seismic ground acceleration coefficient (A_S) of 0.32g at the surface.

The total seismic earth pressure coefficient (k_{ae}) was calculated using the Mononobe-Okabe (M-O) analysis method. For the M-O analysis, the vertical acceleration coefficient (k_v) was assumed to be zero.

For external stability, a reduced horizontal acceleration coefficient (k_{h_d}) was calculated to account for ±2 inches of potential wall displacement (d). The maximum horizontal coefficient (k_h) was then calculated for the MSE walls accounting for inertial wall forces.

For internal stability, the seismic force was calculated using the maximum acceleration developed within the wall ($A_{m(int)}$) without reduction for displacement. The recommended parameters for static and seismic design are summarized in Table 9.

9.5.3. *Traffic Surcharge Loads*. A vertical traffic surcharge pressure of 250 psf was estimated for the walls using a soil surcharge height of 2 feet based on AASHTO (2014) Table 3.11.6.4-1. A factored, uniform surcharge pressure of 438 psf was calculated using a load factor (γ_{LL}) of 1.75. This corresponds to a factored, uniform lateral earth pressure of 135 psf calculated using a k_a of 0.31.

Table 9. Lateral Earth and Seismic Parameters for MSE Wall Design

Parameter	Equation	Value
Active Earth Pressure Coefficient, k_a	$\tan^2(45 - \Phi/2)$	0.31
Active Earth Equivalent Fluid Density	$k_a * \gamma_{backfill}$	38 pcf
Traffic Surcharge (uniform pressure)	$k_a * \gamma_{surcharge} *$surcharge ht.	135 psf
Ground Acceleration, A_S	PGA* F_{pga}	0.32g
Max. Acceleration (Internal Stability), $k_h = A_{m(int)}$	$(1.45 - A_S)A_S$	0.36g
Max. Acceleration, Reduced for Displacement*, k_{h_d}	$0.74A_S(A_S/d)^{0.25}$	0.15g
Max. Horizontal Acceleration (External Stability), k_h	$(1.45 - k_{h\_d})k_{h\_d}$	0.19g
Seismic Earth Pressure Coefficient, k_{ae}	M-O	0.44
Seismic Thrust Coefficient* Δk_{ae}	$k_{ae} - k_a$	0.13

Note: k_{ae} based on 2 inches of displacement

9.6. ***Sliding Resistance***

MSE wall sliding resistance is a function of the weight of the reinforced fill and the friction developed between the materials at the base of the wall. The frictional resistance is estimated using the lessor of the sliding resistance developed within the foundation soil ($c_f + \tan\phi_f$) or within the reinforced fill ($\tan\phi_r$). For the range of assumed reinforced lengths of the proposed walls, the sliding resistance for the foundation soil controls.

Depending on the type of reinforcement, sliding resistance may also depend on the soil-reinforcement interface. It is assumed the sliding resistance at the soil-reinforcement interface will be checked by the wall designer for the final wall configurations.

9.7. ***External Stability***

External stability calculations (bearing resistance, eccentricity/overturning resistance and sliding resistance) were completed using MSEW 3.0 software using the soil parameters recommended herein. Three wall configurations were assumed for the analyses.

- A 37-foot tall wall beneath a 7-foot tall abutment. The base of the wall was assumed at ±El. 262, bearing on clayey silt. Ground water was assumed at ±El. 262. Level ground was assumed in front of the wall

- A 44-foot tall wall adjacent to the abutment. It was assumed the facing would extend to the top of the wall and the wall would have level backfill. The base of the wall was assumed at ±El. 262, bearing on clayey silt. Ground water was assumed at ±El. 262. Level ground was assumed in front of the wall.

- A 30-foot tall wall stepping up the embankment. It was assumed the facing would extend to the top of the wall and the wall would have level backfill. The base of the wall was assumed to bear on embankment fill. It was assumed ground water would not influence the design. A 2(H):1(V) slope was assumed in front of the wall with a minimum embedment of 2 feet.

Table 10 summarizes the results of the analyses and the required reinforced lengths, L, that provide Capacity to Demand Ratios (CDR) of at least 1.0 for bearing resistance and sliding, and e/L values less than 0.25 for overturning.

Table 10. MSE Wall External Stability Calculations (Static)

Wall Configuration	Assumed Foundation Condition	Calculated Minimum Reinforced Length, L (feet)	Factored Bearing Resistance (lb/ft^2)	Bearing CDR	Eccentricity Ratio (e/L)	Sliding CDR
37-foot wall beneath the abutment	Stiff clayey SILT	0.90H	9,390	1.03	0.09	1.83
44-foot wall adjacent to the abutment	Stiff clayey SILT	0.85H	10,354	1.03	0.12	1.87
30-foot wall stepping up the embankment	Embankment Fill (2:1 slope)	0.92H	6,989	1.00	0.11	2.27

Note: H is the total height of the wall.

The external stability calculations were also performed for seismic conditions using the seismic acceleration parameters discussed above and the LRFD extreme event load and resistance factors. Results of the seismic analyses are summarized in Table 11. For each case, the results indicated acceptable CDR values greater than 1.0 for bearing resistance and sliding and e/L values of ±0.25 or less for overturning evaluation.

Table 11. MSE Wall External Stability Calculations (Seismic)

Wall Configuration	Assumed Foundation Condition	Calculated Minimum Reinforced Length, L (feet)	Factored Bearing Resistance (lb/ft^2)	Bearing CDR	Eccentricity Ratio (e/L)	Sliding CDR
37-foot wall beneath the abutment	Stiff clayey SILT	0.90H	12,938	1.24	0.14	1.30
44-foot wall adjacent to the abutment	Stiff clayey SILT	0.85H	13,894	1.17	0.21	1.25
30-foot wall stepping up the embankment	Embankment Fill (2:1 slope)	0.92H	9,417	1.18	0.19	1.55

Note: H is the total height of the wall.

9.8. ***Global Stability***

Global stability analyses were completed for the MSE walls using the computer program Slide 5.0. We analyzed three wall configurations, consistent with those described above for external stability analyses. The contribution of resistance from adjacent walls (e.g., parallel approach walls) and possible overlapping resistance was not accounted for in the global stability models. Therefore, the results are likely to be conservative. Potential failure planes were assumed to extend behind and below (but not through) the reinforced zone.

The subsurface conditions beneath the walls (and/or beneath the embankment fill) were interpolated based on BH-3 and BH-4. Analysis for global stability at the abutment focused on Bent 1 because it will be constructed adjacent to the realigned creek channel and will represent the more critical case.

Ground water was assumed at ±El. 262 for the analyses. We believe this ground water level is conservative for seismic design, since the average annual ground water level (typically used for seismic analysis) is likely lower. A horizontal ground acceleration (k_h) of 0.19g was used for the seismic global stability analysis, consistent with the k_h value in Table 9.

A minimum factor of safety of 1.5 is required for static design to coincide with a resistance factor of 0.65. A minimum factor of safety of 1.1 is required for seismic design. The results of the analyses, summarized in Table 12, indicate factors of safety satisfying these minimum values. The slope stability calculations will be provided in Appendix D of the final Foundation Report.

Table 12. Global Stability Analysis Results

Wall Height, H (feet)	Assumed Reinforced Length, L (feet)	Factor of Safety (Static)	Factor of Safety (Seismic)
37-foot wall beneath the abutment	33.3	1.5	1.2
44-foot wall adjacent to the abutment	37.4	1.8	1.2
30-foot wall stepping up the embankment	27.5	1.5	1.1

10.0. ABUTMENT WALLS

The bridge abutments will include concrete abutment walls and pile caps. Drawings provided by DEA indicate a wall height of ±7 feet (including the cap). We assume Granular Wall Backfill will be used to backfill the walls. A friction angle of 34 degrees and a unit weight of 130 pcf were assumed for the wall backfill. Drained conditions were also assumed.

Typically, abutment walls deflect to mobilize active earth conditions. A lateral deflection of at least ±0.001*H (where H is the height of the wall) is required for the walls to mobilize active earth pressure conditions within the Granular Wall Backfill. For a 7-foot tall wall, the deflection is less than ±0.1 inch. Therefore, we calculated earth pressures assuming active conditions.

An active earth pressure coefficient (k_a) of 0.28 was calculated based on the soil parameters. The nominal lateral earth pressure on unrestrained walls may be estimated using an equivalent fluid density of 36 pcf.

AASHTO (2014) recommends estimating the traffic loads applied to the top of the abutment walls using an equivalent soil surcharge with a minimum height of 3.6 feet for 7-foot tall backfilled abutments without approach panels. Because approach panels will be used, we assumed a one-half reduction in the surcharge height for design. A unit weight of 125 pcf was assumed for the surcharge. The assumed surcharge height (1.8 feet) corresponds to a uniform surcharge pressure of 225 psf. This results in an additional uniform lateral pressure of ±63 psf for active conditions.

The ODOT GDM (2014) requires abutment walls to be designed for a peak horizontal acceleration corresponding to a 1,000-year return period. The total seismic earth pressure coefficient (k_{ae}) was calculated using the Mononobe-Okabe (M-O) analysis method. Consistent with the MSE wall analysis, a horizontal acceleration coefficient, k_h, of 0.19g was used, assuming up to ±2 inches of wall displacement.

The calculations indicate a resulting horizontal seismic force of 388 lb/ft. The seismic force may be modeled using an additional uniform pressure of ± 55 psf. A summary of the calculated abutment wall lateral earth pressures is provided in Table 13.

Table 13. Lateral Earth Parameters for Abutment Wall Design

Parameter	Source	Value	γ_p
Active Earth Pressure Coefficient, k_a	$\tan^2(45 - \phi/2)$	0.28	
Active Equivalent Fluid Density	$k_a * \gamma_{backfill}$	36 pcf	1.50
Traffic Surcharge (uniform pressure)	$k_a * \gamma_{surcharge} *$ surcharge ht	63 psf	1.35/1.75
Seismic Pressure for Wall backfill for 1,000-year event (assumes ± 2 inch displacement)	Mononobe-Okabe	55 psf	1.00

The appropriate load factors (γ_p) provided in AASHTO (2014) Table 3.4.1-2 should be applied to the nominal pressures to estimate the factored lateral earth loads. Selection of the appropriate load factors are dependent on the load case being analyzed. AASHTO (2014) recommends a load factor 1.5 for active earth loads. For the traffic load surcharge, a load factor of 1.75 is recommended for Strength I and 1.35 for Strength II and V.

11.0. CONSTRUCTION RECOMMENDATIONS

11.1. *Specifications*

All specifications contained herein refer to ODOT's Oregon Standard Specifications for Construction (2015). It is also assumed these specifications will be referred to for general or specific items not addressed in this report.

11.2. *Driven Piles*

The specifications for piles and pile driving should follow the requirements of Section 00520. A monitoring program is recommended during construction to confirm all pile driving criteria are followed. We anticipate a construction inspector will log each pile for driving resistance and hammer efficiency. The driving criteria should be established by Foundation Engineering using WEAP analysis prior to construction once the pile hammer has been selected by the contractor. Driving should be discontinued once the pile meets the required driving resistance (between 2 and 10 blows/inch (bpi) for 3 consecutive inches) at or below the minimum tip elevation.

The piles will be driven through CMP sleeves extending through the MSE wall backfill. Additional details of the CMP installation and backfilling are discussed below.

11.3. *Approach Embankments*

11.3.1. *Subgrade Preparation and Embankment Construction.* Prior to embankment construction, the embankment areas should be cleared and grubbed in accordance with Section 00320.40. An average grubbing depth of ±6 inches should be anticipated in the grassy areas. An average grubbing depth of at least ±12 inches should be expected in the more densely vegetated and tree-lined areas (primarily south of the bridge). Deeper grubbing depths will also be required to remove larger root balls. Organic rich materials from the clearing and grubbing should not be incorporated in the embankment construction.

The subgrade should be evaluated by a Foundation Engineering representative prior to construction. If practical, the subgrade beneath the embankments and MSE walls should be compacted prior to backfilling to provide a firm surface for placing subsequent fill. Compaction of the subgrade will not be practical if the subgrade soils are wet of optimum. Therefore, subgrade preparation should be completed only during the dry summer months.

In the event embankment construction occurs during the winter months or in early spring when the subgrade is still wet, compaction should not be attempted and angular rock will be required for subgrade stabilization.

Finished embankment slopes should be constructed at 2(H):1(V), or flatter. Steeper slopes (up to 1.5(H):1(V)) may be constructed if angular Stone Embankment Material (Section 00330.16) is used.

11.3.2. *Embankment Fill.* Embankment construction outside the MSE walls can be completed using a variety of fill materials. Selection of the most appropriate material will depend on the time of year the embankments are constructed.

If the work is completed in the dry summer months, the embankments can be constructed using Borrow Material (Section 00330.12). Based on our investigation, we anticipate the fill material currently stockpiled adjacent to the proposed north approach should be suitable for use as Borrow Material during dry weather. However, the suitability of this material will need to be confirmed during construction. The consistency of the stockpiled fill encountered in our explorations indicates this material was not placed in compacted lifts. Therefore, the fill will need to be moisture-conditioned and compacted and should not just be left in place, where it lies within the footprint of the new north approach embankment.

If embankment construction is completed during wet weather in the winter or spring, clean, angular, granular fill meeting the requirements of Stone Embankment Material or Granular Structure Backfill should be used. Depending on the site conditions, an Embankment Geotextile (02320.20) may also be required beneath the embankment fill for construction during wet weather.

11.3.3. *Abutment Walls.* Granular Wall Backfill (00510.12) should be used to backfill the abutment walls and pile caps. Placement and compaction of this material should be completed using light, vibratory equipment within a distance equal to one-half of the wall height.

11.4. ***MSE Walls***

Construction of the MSE walls should conform to the requirements for site preparation and wall construction in Special Provision (SP) 00596. We recommend the base of the walls extend a minimum of 2 feet below the finish grade. The subgrade beneath the walls should be compacted prior to constructing the walls, if practical. Any soft or loose soils encountered at the design subgrade elevation should be overexcavated and replaced with additional Stone Embankment Material or Granular Structure Backfill.

A leveling pad should be provided beneath the wall facing units. The leveling pad should consist of at least ±6 inches of compacted Granular Structure Backfill or unreinforced concrete in accordance with the wall manufacturer's specifications.

Pile driving for the abutments will require the installation of CMP sleeves at the pile locations. The sleeves should extend through the MSE backfill and any stabilization fill placed below the MSE walls. Based on the PP24 pile sections, we recommend using a 30-inch diameter CMPs. The CMPs should conform to Section 02420 and have a minimum wall thickness of 0.052 inches.

MSE Granular Backfill should be used as backfill around the sleeves. Backfill within a ±3.5-foot radius of the CMP sleeves should be carefully compacted using light, hand-operated equipment to avoid damaging the sleeves.

Backfill placed in the annulus between the inside of the CMPs and the outside of the piles should consist of durable, ⅜-inch, open-graded, uncrushed, rounded gravel. The gradation in Table 14 is recommended. However, alternative gradations may be submitted for review based on availability.

Table 14. Recommended Gradation for CMP Sleeve Backfill

Sieve Size	Percent Passing (by Weight)
1/2″	100
3/8″	85 – 100
No. 4	10 – 30
No. 8	0 – 10
No. 16	0 – 5
No. 200	0 – 1

The wall backfill within the reinforced zone should consist of granular fill meeting the gradation requirements for MSE Granular Backfill (Section 00596.10(g)). The fill should be compacted to at least 95% relative compaction according to the maximum dry density of AASHTO T99. Compaction adjacent to the wall facing units should be completed using only light, hand-operated or walk-behind equipment (such as vibratory plate compactors) according to the wall manufacturer's specifications. We do not recommend the use of heavy rollers or hydraulic compactors mounted on excavators or backhoes close to the wall facing since they may create excessive lateral earth pressures on the wall. Heavy, vibratory equipment operating close to the CMP sleeves may also cause distortion or damage to the sleeves.

11.5. ***Excavations/Shoring/Dewatering***

Excavations for the MSE walls are expected to extend primarily through stiff clayey silt. This soil corresponds to an OR-OSHA Type B soil. OR-OSHA recommends temporary slopes no steeper than 1(H):1(V) for these soils. Flatter slopes will be required to control erosion and sloughing during wet weather. It is the contractor's responsibility to maintain stable cut slopes and provide the necessary shoring as required by OR-OSHA.

Ground water was not encountered in the test pits to a maximum depth of ±11.5 feet. Therefore, during dry weather, we do not anticipate the need for dewatering with the exception of excavations adjacent to the creek. During wet weather, it should be anticipated ground water will pond at the ground surface and in excavations, and may require dewatering. Shallow perched water may also be encountered during prolonged wet weather.

11.6. ***Falsework Support***

We anticipate any required falsework or temporary structural supports will be designed by the contractor.

11.7. ***Seasonal Issues***

The surficial fine-grained soils will be moisture-sensitive and will become soft, weak and unworkable when exposed to excessive moisture. Therefore, we recommend the construction of the approaches and MSE walls be done only during dry weather to minimize subgrade disturbance and allow the reuse of excavated soils for embankment construction.

12.0. LIMITATIONS

12.1. ***Construction Observation/Testing***

We recommend a member of the consultant team be present to observe the pile driving. Construction observation should also be maintained throughout embankment and MSE wall construction to observe the subgrade conditions, fill placement and compaction procedures. Any geotechnical engineering judgment in the field should be provided by a representative of the consultant team. Frequent field density tests should be run on all compacted subgrade and fill. Compaction of fill material that is too coarse or variable for density testing will need to be evaluated by observation of the compaction procedures and periodic proof-rolls using approved heavy construction equipment.

12.2. ***Variation of Subsurface Conditions, Use of Report and Warranty***

The analysis, conclusions, and recommendations contained herein assume the subsurface profiles encountered in the borings and test pits are representative of the site conditions. The above recommendations assume Foundation Engineering will have the opportunity to review final drawings and be present during construction to confirm the assumed subgrade conditions beneath the proposed MSE walls and embankments and observe pile driving. No changes in the enclosed recommendations should be made without our approval. Foundation Engineering assumes no responsibility or liability for any engineering judgment, inspection, or testing performed by others.

This report was prepared for the exclusive use of David Evans and Associates, Inc., Benton County, and their design consultants for the SW 53rd Street Railroad Crossing project in Benton County, Oregon. Information contained herein should not be used for other sites or for unanticipated construction without our written consent. This report is intended for planning and design purposes. Contractors using this information to estimate construction quantities or costs do so at their own risk. Our services do not include any survey or assessment of potential surface contamination or contamination of the soil or ground water by hazardous or toxic materials. We assume those services, if needed, have been completed by others.

Foundation Engineering's work was done in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

13.0. REFERENCES

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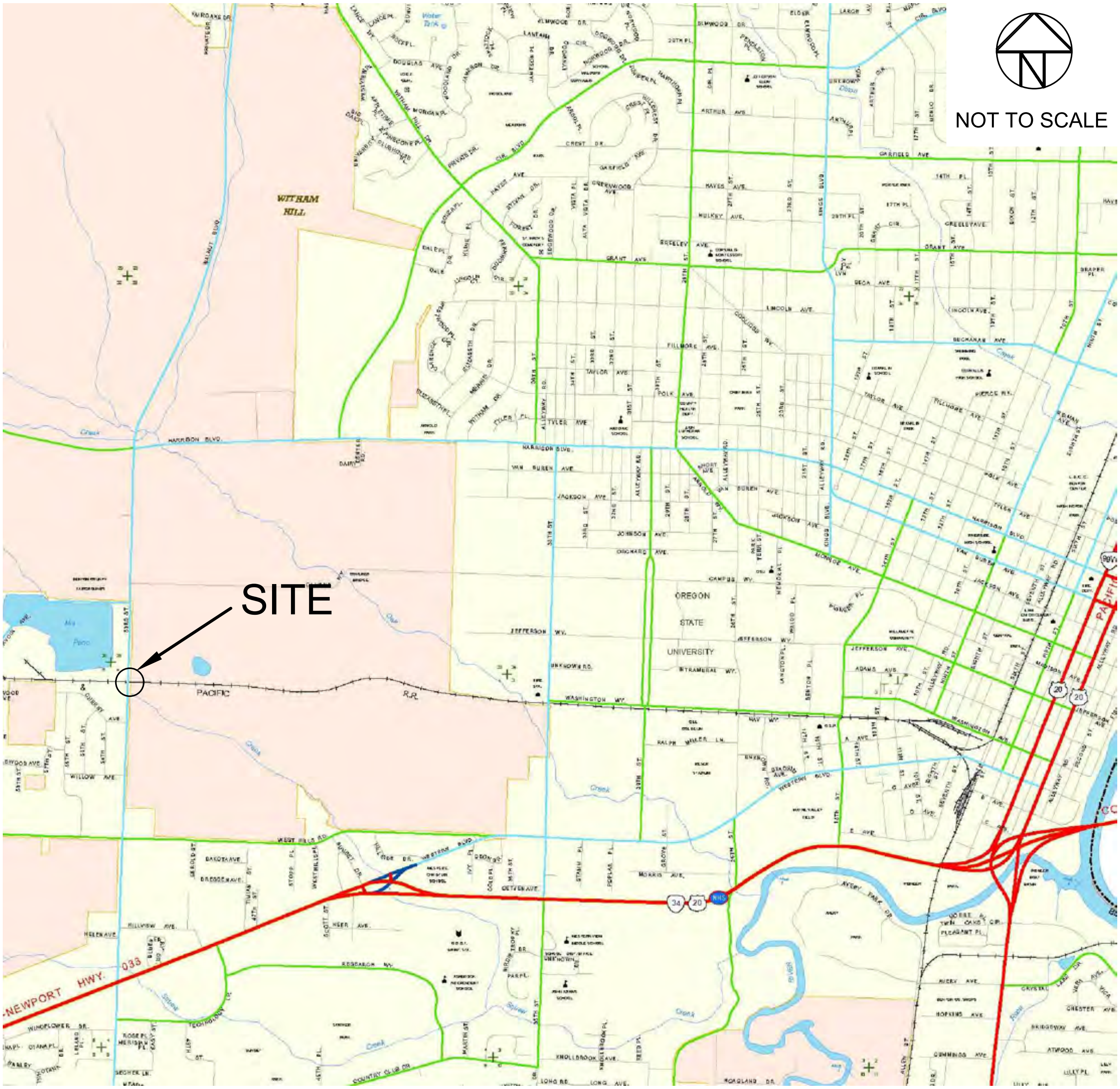
Appendix A

Figures and Tables

Professional
Geotechnical
Services

Foundation Engineering, Inc.



BASEMAP PROVIDED BY OREGON DEPARTMENT OF TRANSPORTATION (2011)
http://www.oregon.gov/ODOT/TD/TDATA/gis/odotmaps.shtml

DATE DEC. 2014
DWN. BZH
APPR.
REVIS.
PROJECT NO.
2141009

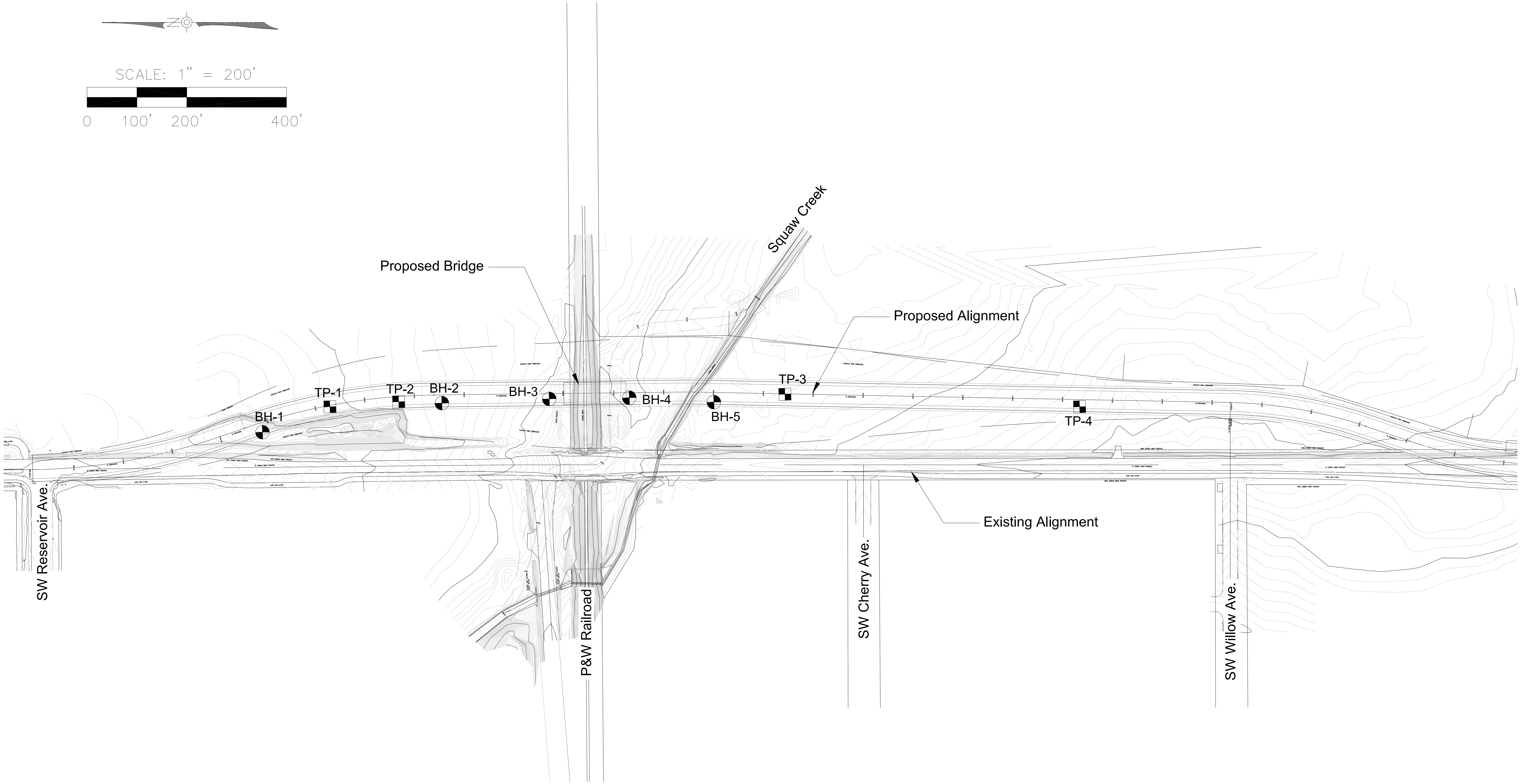
FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 NW CORNELL AVENUE
CORVALLIS, OR 97330-4517
BUS. (541) 757-7645 FAX (541) 757-7650

VICINITY MAP

SW 53rd STREET RAILROAD CROSSING
BENTON COUNTY, OREGON

FIGURE NO.
1A

FILE NAME: FIG 1A



NOTES:
1. EXPLORATION LOCATIONS ARE APPROXIMATE ONLY.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY BENTON COUNTY PUBLIC WORKS DEPARTMENT.

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 NW CORNELL AVENUE
CORVALLIS, OR 97330-4517
BUS. (541) 757-7645 FAX (541) 757-7650

DATE OCT. 2015
DWN. MDM
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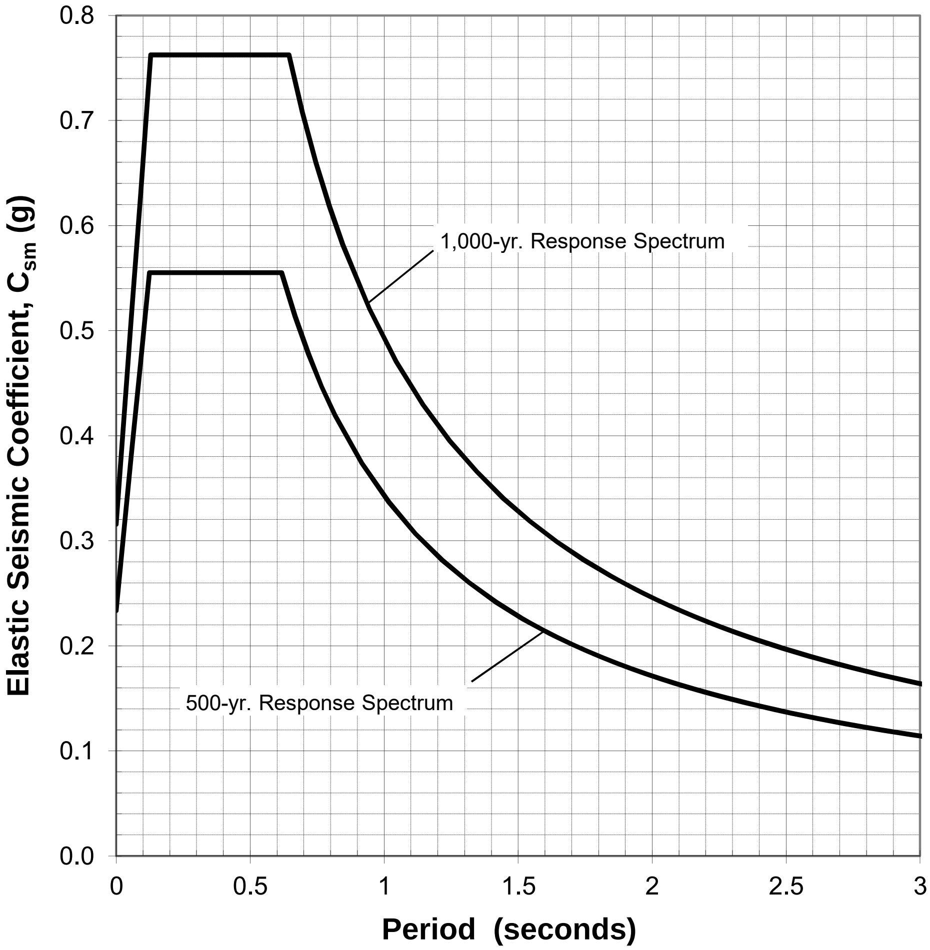
2141009

EXISTING AND PROPOSED ALIGNMENTS
AND EXPLORATION LOCATIONS
SW 53RD STREET RAILROAD CROSSING
BENTON COUNTY, OREGON

FIGURE NO.
2A



Notes:

1. The Design Response Spectrum is based on AASHTO 2012 Section 3.10.3 using the following parameters:

Site Class= D Damping = 5%

1,000-yr.	PGA =	0.24	F_{pga} =	1.32	A_s =	0.32
	S_S =	0.57	F_a =	1.34	S_{DS} =	0.76
	S_1 =	0.26	F_v =	1.87	S_{D1} =	0.49
500-yr.	PGA =	0.16	F_{pga} =	1.49	A_s =	0.23
	S_S =	0.37	F_a =	1.50	S_{DS} =	0.56
	S_1 =	0.16	F_v =	2.17	S_{D1} =	0.34

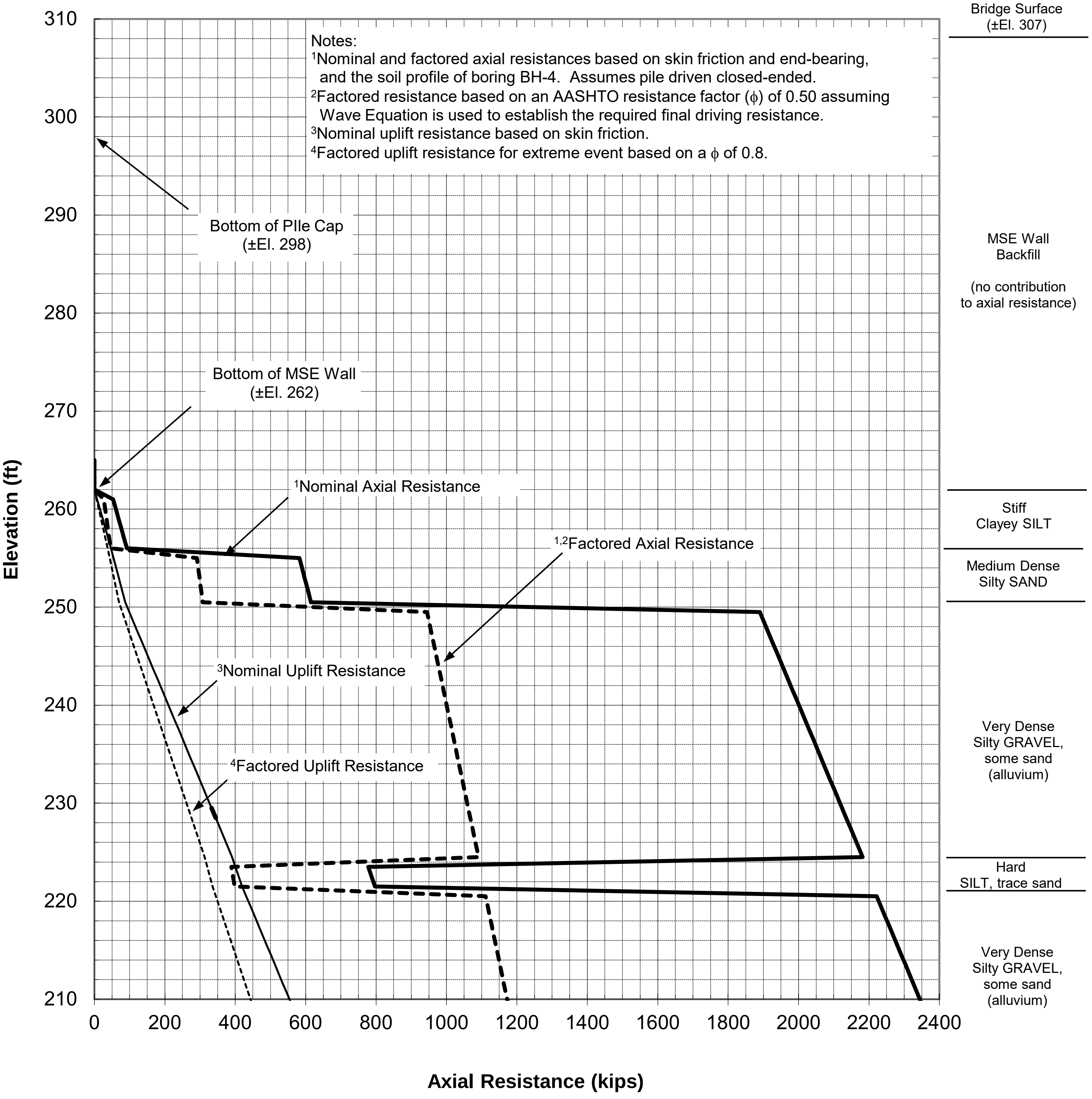
2. PGA, S_S and S_1 values are based on USGS 2002 maps and mapping software included in AASHTO 2012. The 1,000-yr. values assume 7% probability of exceedence in 75 years. The 500-yr. values assume 10% probability of exceedence in 50 years.

3. F_{pga}, F_a and F_v were established based on AASHTO 2008, Tables 3.10.3.2-1, 3.10.3.2-2 and 3.10.3.2-3 using the selected PGA, S_s and S_1 values, respectively.

4. Site location is: Latitude 44.5622, Longitude -123.3115.

FIGURE 3A.
AASHTO 2012 GENERAL PROCEDURE RESPONSE SPECTRA

SW 53rd Street Railroad Crossing
Benton County, Oregon
Project No. 2141009



Notes:
[1] Nominal and factored axial resistances based on skin friction and end-bearing, and the soil profile of boring BH-4. Assumes pile driven closed-ended.
[2] Factored resistance based on an AASHTO resistance factor (ϕ) of 0.50 assuming Wave Equation is used to establish the required final driving resistance.
[3] Nominal uplift resistance based on skin friction.
[4] Factored uplift resistance for extreme event based on a ϕ of 0.8.

FIGURE 4A
AXIAL RESISTANCE vs. ELEVATION - BENT 1
PP24x0.5 (Closed-Ended)
SW 53rd Street Railroad Crossing
Benton County, Oregon
Project No. 2141009

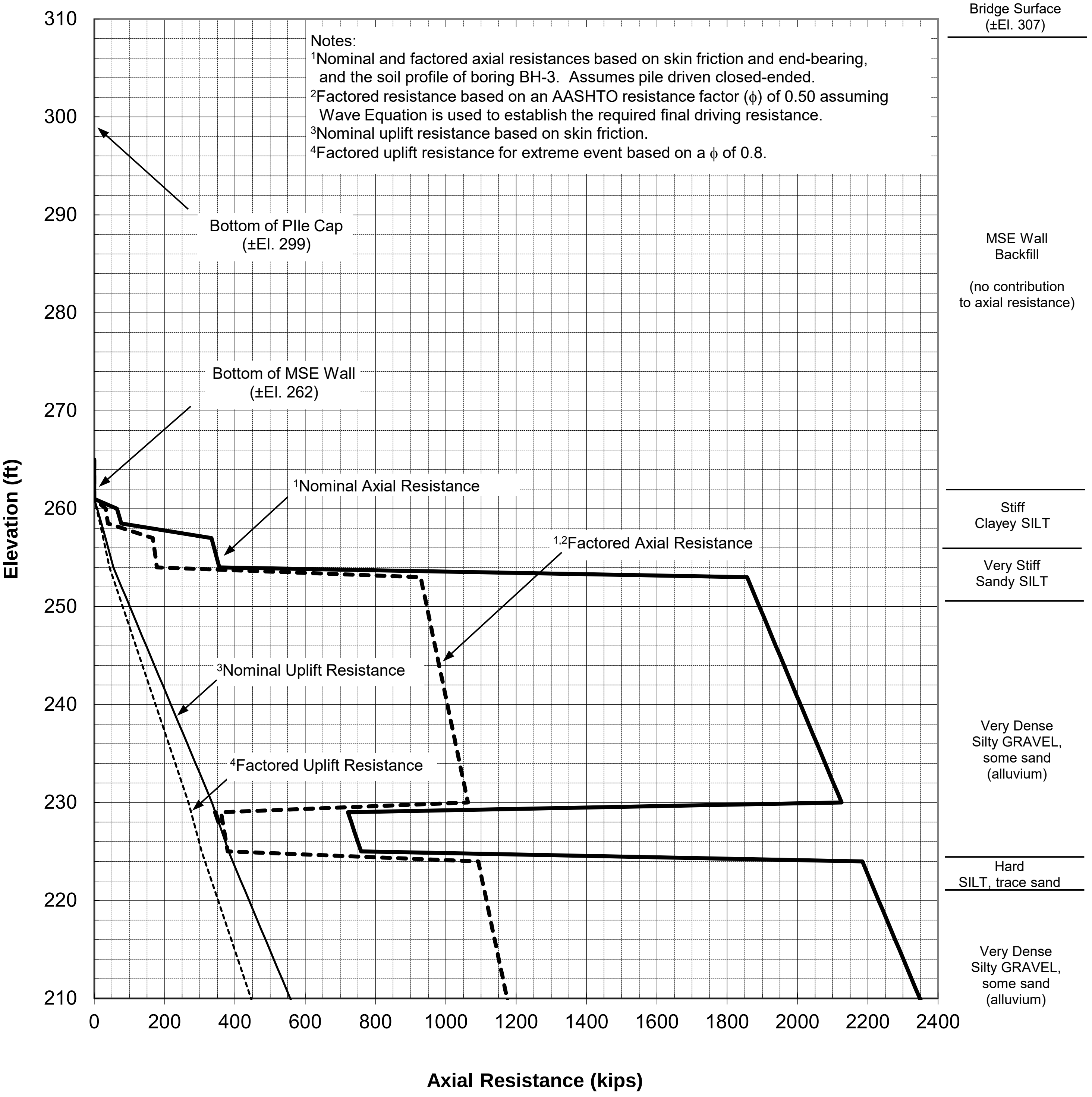


FIGURE 5A
AXIAL RESISTANCE vs. ELEVATION - BENT 2
PP24x0.5 (Closed-Ended)
SW 53rd Street Railroad Crossing

Benton County, Oregon
Project No. 2141009

Table 1A. Recommended WEAP Input Parameters

Bent	Pile Type	Pile Length (ft)	Quake (in)		Damping (sec/ft)		Distribution	% skin (ITYS)	R_n (kips)
			Skin	Toe	Skin	Toe			
Bent 1	PP24x0.5	60	0.10	0.13	0.15	0.15	Triangular	10	845
Bent 2	PP24x0.5	60	0.10	0.13	0.15	0.15	Triangular	10	845

Appendix B

Boring and Test Pit Logs

Professional
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Foundation Engineering, Inc.

DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for each boring or test pit by our field representative. The log contains information concerning sampling depths and the presence of various materials such as gravel, cobbles, and fill, and observations of ground water. It also contains our interpretation of the soil conditions between samples. The final logs presented in this report represent our interpretation of the contents of the field logs and the results of the laboratory examinations and tests. Our recommendations are based on the contents of the final logs and the information contained therein and not on the field logs.

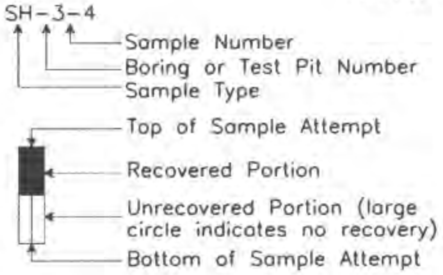
VARIATION IN SOILS BETWEEN TEST PITS AND BORINGS

The final log and related information depict subsurface conditions only at the specific location and on the date indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ. Actual foundation or subgrade conditions should be confirmed by us during construction.

TRANSITION BETWEEN SOIL OR ROCK TYPES

The lines designating the interface between soil, fill or rock on the final logs and on subsurface profiles presented in the report are determined by interpolation and are therefore approximate. The transition between the materials may be abrupt or gradual. Only at boring or test pit locations should profiles be considered as reasonably accurate and then only to the degree implied by the notes thereon.

SAMPLE OR TEST SYMBOLS



S – Grab Samples
SS – Standard Penetration Test Sample (split-spoon)
SH – Thin-walled Shelby Tube Sample
C – Core Sample
CS – Continuous Sample

▲ Standard Penetration Test Resistance equals the number of blows a 140 lb. weight falling 30 in. is required to drive a standard split-spoon sampler 1 ft. Practical refusal is equal to 50 or more blows per 6 in. of sampler penetration.

● Water Content (%).

UNIFIED SOIL CLASSIFICATION SYMBOLS

G – Gravel
S – Sand
M – Silt
C – Clay
Pt – Peat

W – Well Graded
P – Poorly Graded
L – Low Plasticity
H – High Plasticity
O – Organic

FIELD SHEAR STRENGTH TEST

Shear strength measurements on test pit side walls, blocks of soil or Shelby tube samples are typically made with Torvane or pocket penetrometer devices.

TYPICAL SOIL/ROCK SYMBOLS

□	Sand	▥	Silt
▨	Clay	⁘	Gravel
▩	Basalt	▤	Siltstone

WATER TABLE

▼	Water Table Location
(1/31/00)	Date of Measurement
▤	Piezometer Tip Location (if used)

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES

820 NW CORNELL AVE.
CORVALLIS, OR 97330-4517
BUS. (541) 757-7645 FAX (541) 757-7650

SYMBOL KEY
BORING AND TEST PIT LOGS

Explanation of Common Terms Used in Soil Descriptions

Field Identification	Cohesive Soils			Granular Soils	
	SPT	S_u^* (tsf)	Term	SPT	Term
Easily penetrated several inches by fist.	0 – 1	< 0.125	Very Soft	0 – 4	Very Loose
Easily penetrated several inches by thumb.	2 – 4	0.125–0.25	Soft	5 – 10	Loose
Can be penetrated several inches by thumb with moderate effort.	5 – 8	0.25 – 0.50	Medium Stiff (Firm)	11 – 30	Medium Dense
Readily indented by thumb but penetrated only with great effort.	9 – 15	0.50 – 1.0	Stiff	31 – 50	Dense
Readily indented by thumbnail.	16 – 30	1.0 – 2.0	Very Stiff	> 50	Very Dense
Indented with difficulty by thumbnail.	31 – 60	> 2.0	Hard		

* Undrained shear strength

Term	Soil Moisture Field Description
Dry	Absence of moisture. Dusty. Dry to the touch.
Damp	Soil has moisture. Cohesive soils are below plastic limit and usually moldable.
Moist	Grains appear darkened, but no visible water. Silt/clay will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grain surfaces. Sand and cohesionless silt exhibit dilatancy. Cohesive silt/clay can be readily remolded. Soil leaves wetness on the hand when squeezed. "Wet" indicates that the soil is wetter than the optimum moisture content and above the plastic limit.

Term	PI	Plasticity Field Test
Nonplastic	0 – 3	Cannot be rolled into a thread.
Low Plasticity	3 – 15	Can be rolled into a thread with some difficulty.
Medium Plasticity	15 – 30	Easily rolled into thread.
High Plasticity	> 30	Easily rolled and rerolled into thread.

Term	Soil Structure Criteria
Stratified	Alternating layers at least 1 inch thick – describe variation.
Laminated	Alternating layers at less than 1 inch thick – describe variation.
Fissured	Contains shears and partings along planes of weakness.
Slickensides	Partings appear glossy or striated.
Blocky	Breaks into lumps – crumbly.
Lensed	Contains pockets of different soils – describe variation.

Term	Soil Cementation Criteria
Weak	Breaks under light finger pressure.
Moderate	Breaks under hard finger pressure.
Strong	Will not break with finger pressure.

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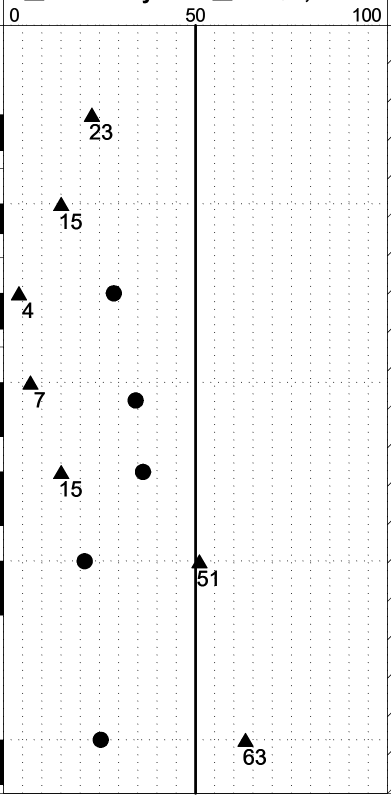
COMMON TERMS
SOIL DESCRIPTIONS

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	▲ SPT, N-Value ● Moisture, % ⬚ Recovery ⊞ RQD., %	Installations/ Water Table
1 2 3 4 5 6 7	Sandy GRAVEL, some silt, (GP); grey, medium plasticity silt, moist, medium dense, fine to coarse sand, fine, angular to subrounded gravel, (fill).		283.31 0.0	SS-1-1 SS-1-2		Backfilled with bentonite chips
8 9	SILT, (ML); grey, low plasticity, wet, soft to medium stiff, (fill).		275.8 7.5	SS-1-3		
10 11 12	Clayey SILT, (MH); light brown, medium plasticity, moist, medium stiff to stiff, (Willamette Silt).		273.3 10.0	SS-1-4		
13 14 15	Clayey SILT, trace sand, (MH); brown, medium plasticity, damp to moist, stiff, fine to coarse sand, (alluvium). Trace gravel below ±13 feet.		270.8 12.5	SS-1-5		
16 17 18 19 20 21	Silty GRAVEL, some sand, (GM); brown, iron-stained, low plasticity silt, damp to moist, very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).		268.3 15.0	SS-1-6 SS-1-7		
	BOTTOM OF BORING		261.8 21.5			



Project No.: 2141009

Surface Elevation: 283.31 feet

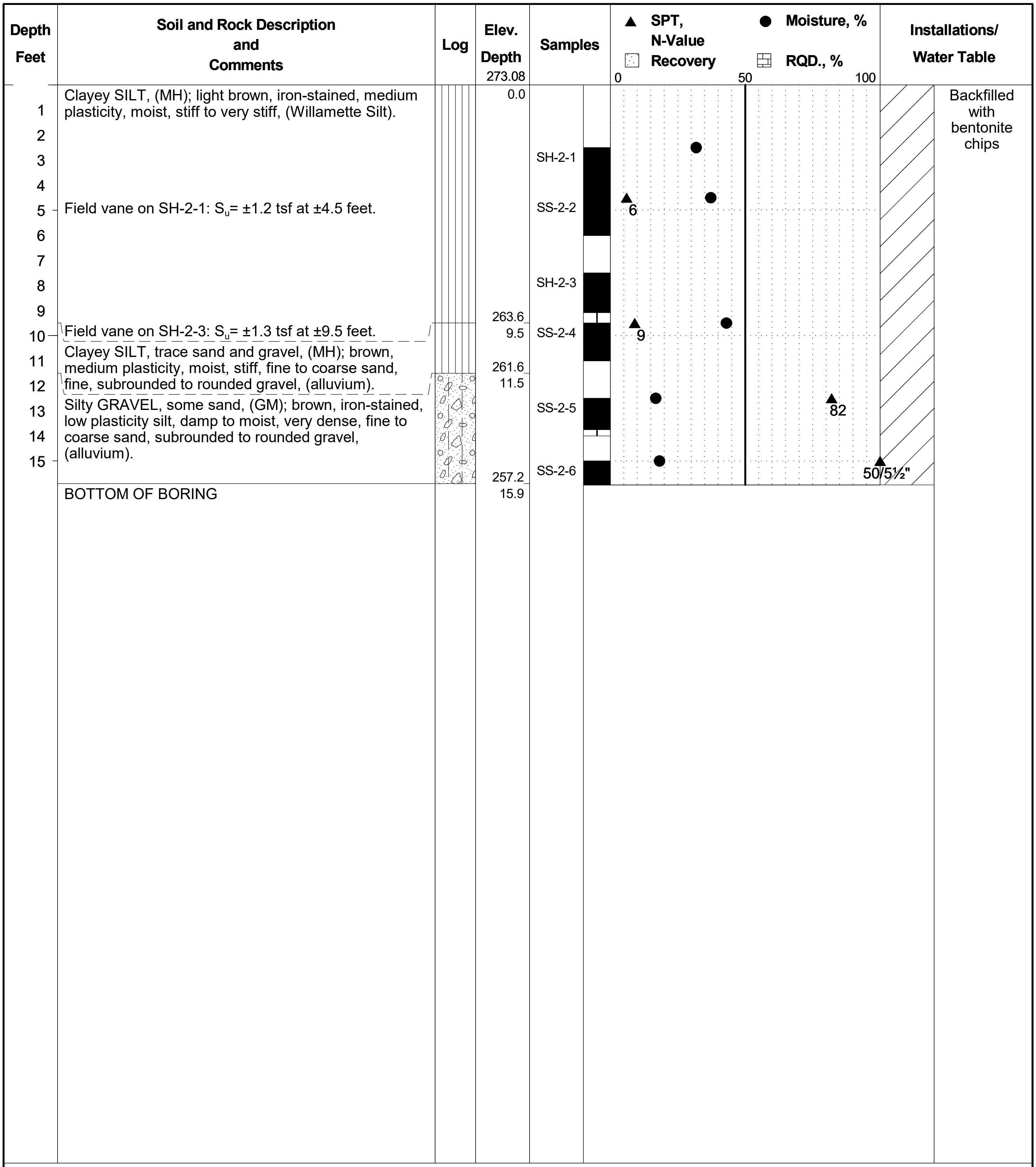
Date of Boring: October 28, 2014

Foundation Engineering, Inc.

Boring Log: BH-1

SW 53rd Street Railroad Crossing

Benton County, Oregon



Depth Feet	Soil and Rock Description and Comments	Elev. Depth 273.08	Samples	SPT, N-Value	Installations/ Water Table
	Clayey SILT, (MH); light brown, iron-stained, medium plasticity, moist, stiff to very stiff, (Willamette Silt).	0.0			Backfilled with bentonite chips
3			SH-2-1		
5	Field vane on SH-2-1: S_u= ±1.2 tsf at ±4.5 feet.		SS-2-2	6	
8			SH-2-3		
10	Field vane on SH-2-3: S_u= ±1.3 tsf at ±9.5 feet.	263.6 9.5	SS-2-4	9	
	Clayey SILT, trace sand and gravel, (MH); brown, medium plasticity, moist, stiff, fine to coarse sand, fine, subrounded to rounded gravel, (alluvium).				
	Silty GRAVEL, some sand, (GM); brown, iron-stained, low plasticity silt, damp to moist, very dense, fine to coarse sand, subrounded to rounded gravel, (alluvium).	261.6 11.5			
13			SS-2-5	82	
15			SS-2-6	50/5½"	
	BOTTOM OF BORING	257.2 15.9			

Project No.: 2141009

Surface Elevation: 273.08 feet

Date of Boring: October 28, 2014

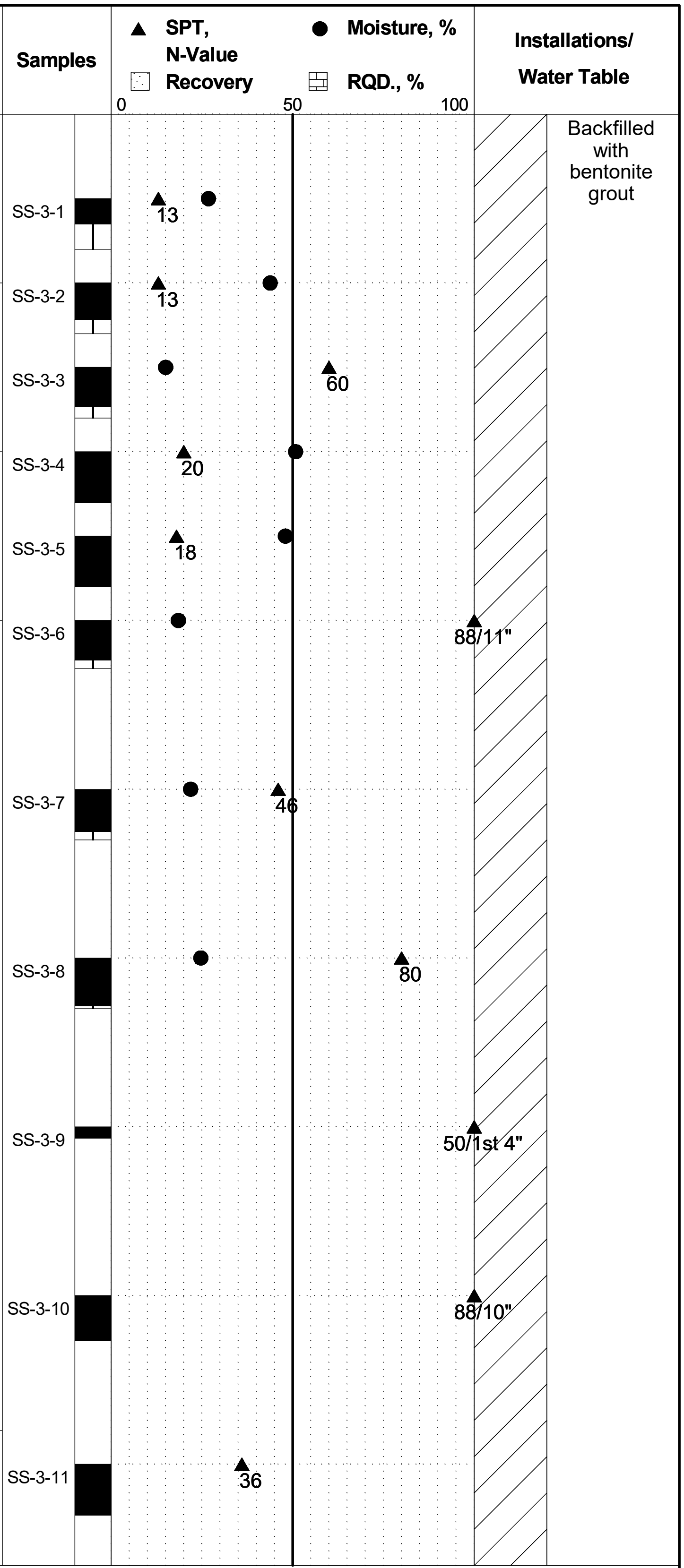
Foundation Engineering, Inc.

Boring Log: BH-2

SW 53rd Street Railroad Crossing

Benton County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth 269.5
0	Clayey SILT, (MH); light brown, iron-stained, medium plasticity, damp, stiff, (Willamette Silt).	0.0
5	Clayey SILT, trace sand and gravel, (MH); brown, iron and manganese-stained, low to medium plasticity, damp, stiff, fine to coarse sand, fine, subrounded to rounded gravel, (alluvium).	264.5 5.0
10	Sandy SILT, (ML); brown, iron-stained, low plasticity, damp, very stiff, fine sand, (alluvium).	259.5 10.0
15	Silty GRAVEL, some sand (GM): brown, iron-stained, low plasticity silt, moist, very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).	254.5 15.0
20	Dense at ±20 feet.	
39	SILT, trace sand, (ML); blue-grey, iron-stained, low plasticity, damp to moist, hard, fine sand, (alluvium).	230.5 39.0



Samples	SPT, N-Value
SS-3-1	13
SS-3-2	13
SS-3-3	60
SS-3-4	20
SS-3-5	18
SS-3-6	88/11"
SS-3-7	46
SS-3-8	80
SS-3-9	50/1st 4"
SS-3-10	88/10"
SS-3-11	36

Project No.: 2141009

Surface Elevation: 269.50 feet

Date of Boring: October 28, 2014

Foundation Engineering, Inc.

Boring Log: BH-3

SW 53rd Street Railroad Crossing

Benton County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	▲ SPT, N-Value (0 – 50 – 100)
		226.5		
43.5	Silty GRAVEL, some sand, (GM); dark grey, low plasticity silt, moist, very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).	226.0 43.5		
45			SS-3-12	79
50			SS-3-13	80/9"
55			SS-3-14	50/1st 5"
60			SS-3-15	50/1st 5½"
65			SS-3-16	50/5"
70			SS-3-17	50/1st 5"
72	SILT, trace sand, (ML); blue-grey, low plasticity, moist, hard, fine sand, (alluvium).	197.5 72.0		
75			SS-3-18	35
77	Silty GRAVEL, some sand, (GM); dark grey, low plasticity silt, moist, very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).	192.5 77.0		
80			SS-3-19	50/5"
80.9	BOTTOM OF BORING	188.6 80.9		

Legend: ▲ SPT, N-Value; ● Moisture, %; ⬚ Recovery; ⊞ RQD., %; Installations/Water Table

Project No.: 2141009

Surface Elevation: 269.50 feet

Date of Boring: October 28, 2014

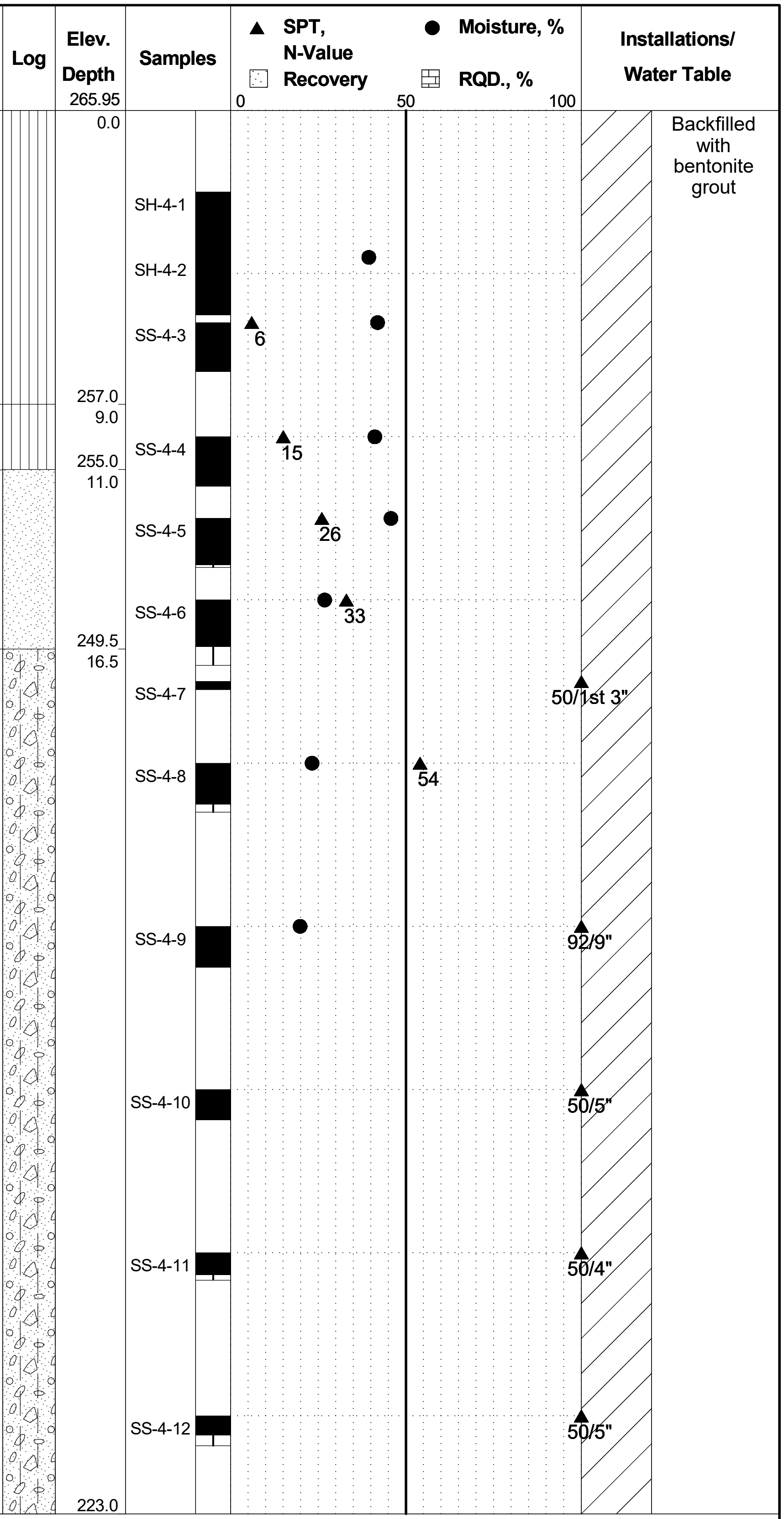
Foundation Engineering, Inc.

Boring Log: BH-3

SW 53rd Street Railroad Crossing

Benton County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Installations/ Water Table
	Clayey SILT, (MH); brown, medium plasticity, moist, stiff, (Willamette Silt).	265.95 0.0			Backfilled with bentonite grout
3			SH-4-1		
5	Field vane on SH-4-1: S_u= ±0.8 tsf at ±4.5 feet. Scattered wood debris at ±5 feet.		SH-4-2		
7	Field vane on SH-4-2: S_u= ±0.85 tsf at ±6.5 feet.		SS-4-3	6	
9	Clayey SILT, trace sand and gravel, (MH); grey-brown, medium plasticity, damp to moist, stiff, fine to coarse sand, fine, subrounded to rounded gravel, (alluvium).	257.0 9.0			
10			SS-4-4	15	
11	Silty SAND, (SM); brown, iron-stained, low plasticity silt, damp to moist, medium dense to dense, fine sand, (alluvium). Some gravel below ±12.5 feet.	255.0 11.0			
13			SS-4-5	26	
15			SS-4-6	33	
17	Silty GRAVEL, some sand, (GM); brown, iron-stained, low plasticity silt, moist, very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).	249.5 16.5			
18			SS-4-7	50/1st 3"	
20			SS-4-8	54	
25	Grey below ±25 feet.		SS-4-9	92/9"	
30			SS-4-10	50/5"	
35			SS-4-11	50/4"	
40			SS-4-12	50/5"	
42		223.0			



Project No.: 2141009

Surface Elevation: 266.0 feet (Approx.)

Date of Boring: October 28, 2014

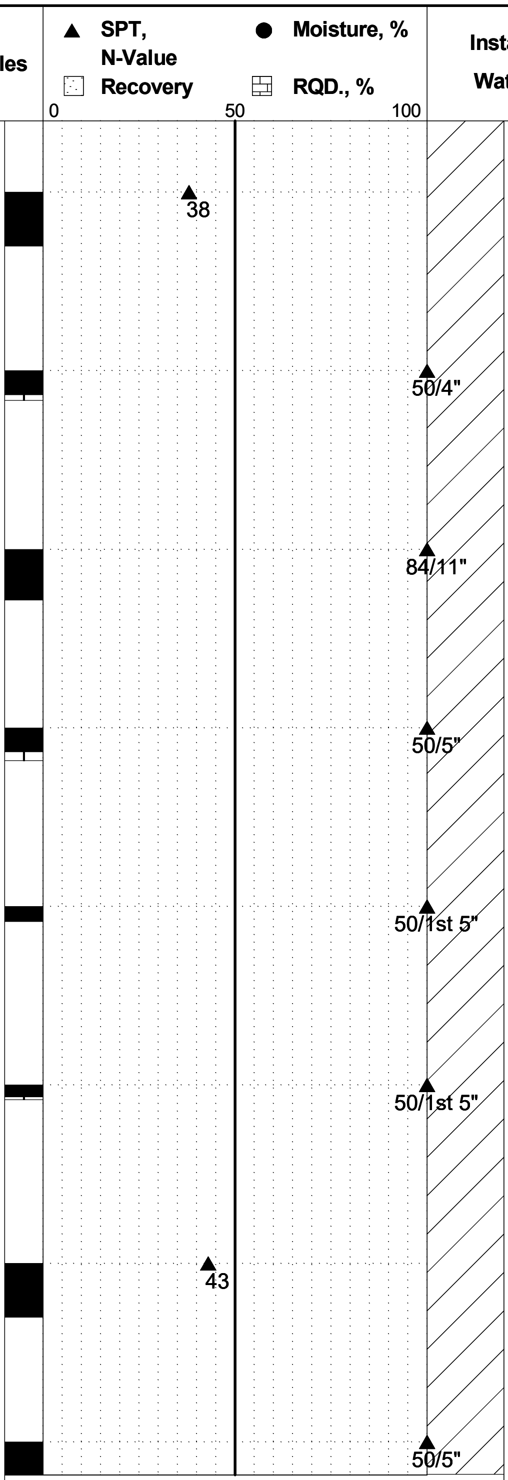
Foundation Engineering, Inc.

Boring Log: BH-4

SW 53rd Street Railroad Crossing

Benton County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth 222.95	Samples	SPT, N-Value
44	SILT, trace sand, (ML); blue-grey, low plasticity, moist, very stiff, fine sand, (alluvium).	43.0		
45	Wood debris (±2 inches thick) at ±45.5 feet.		SS-4-13	38
46	Silty GRAVEL, some sand, (GM); grey, low plasticity silt, moist, very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).	220.0 46.0		
50			SS-4-14	50/4"
55			SS-4-15	84/11"
60			SS-4-16	50/5"
63	Drilling action suggests scattered very thin silt lenses from ±50 to ±80 feet.			
65			SS-4-17	50/1st 5"
70			SS-4-18	50/1st 5"
73	SILT, trace to some sand, trace gravel, (ML); grey, low plasticity, moist, hard, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).	193.5 72.5		
75			SS-4-19	43
77	Silty GRAVEL, some sand, (GM); grey, low plasticity silt, moist, very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium). Drilling action suggests scattered very thin silt lenses from ±50 to ±80 feet.	189.5 76.5		
80			SS-4-20	50/5"
	BOTTOM OF BORING	185.0 80.9		



Project No.: 2141009

Surface Elevation: 266.0 feet (Approx.)

Date of Boring: October 28, 2014

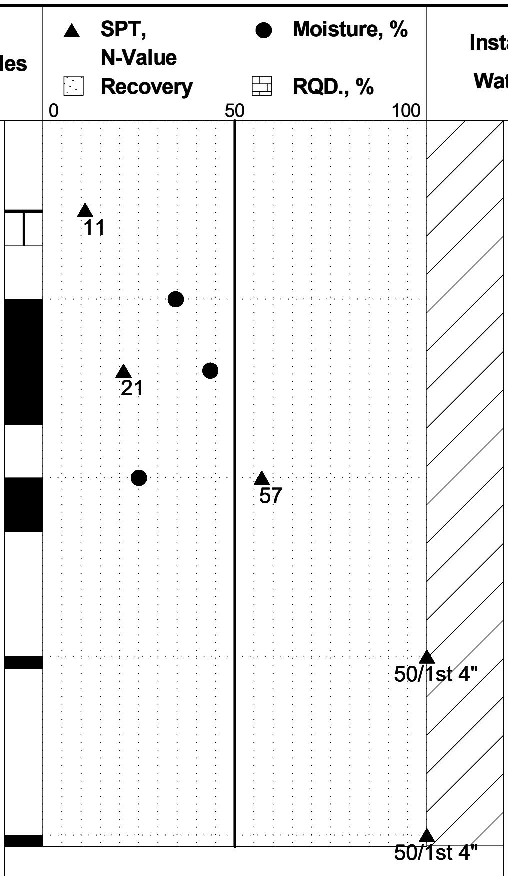
Foundation Engineering, Inc.

Boring Log: BH-4

SW 53rd Street Railroad Crossing

Benton County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	Installations/ Water Table
	Clayey SILT, (MH); grey, iron-stained, medium plasticity, moist, stiff to very stiff, (Willamette Silt).	263.43 0.0		Backfilled with bentonite chips
1				
2				
3			SS-5-1	
4				
5			SH-5-2	
6				
7	Field vane on SH-5-2: S_u= ±1.1 tsf at ±7 feet.		SS-5-3	
8	Silty GRAVEL, some sand, (GM); brown, iron-stained, low plasticity, damp to moist, very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).	255.4 8.0		
9				
10			SS-5-4	
11				
12				
13				
14				
15			SS-5-5	
16				
17				
18				
19				
20	BOTTOM OF BORING	243.2 20.3	SS-5-6	



Project No.: 2141009

Surface Elevation: 263.43 feet

Date of Boring: October 28, 2014

Foundation Engineering, Inc.

Boring Log: BH-5

SW 53rd Street Railroad Crossing

Benton County, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.		S-1-1	■					SILT, scattered organics, (ML); brown, low to medium plasticity, moist, medium stiff, organics consist of fine roots, blocky structure, (topsoil).
	1							
	2	S-1-2	■			0.50 1.60		Clayey SILT, (MH); light brown, iron-stained, medium plasticity, moist, stiff, (Willamette Silt).
	3							
	4	S-1-3	■			1.00		
No seepage or ground water encountered to the limit of excavation.	5							
	6							
	7							
	8							
	9	S-1-4	■					Silty GRAVEL, trace sand, scattered cobbles, (GM); brown, low to medium plasticity silt, dense to very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, cobbles up to ±4 inches in diameter, (alluvium).
	10							
	11							BOTTOM OF TEST PIT
	12							

Project No.: 2141009

Surface Elevation: 275.64 feet

Date of Test Pit: November 7, 2014

Test Pit Log: TP-1

SW 53rd Street Railroad Crossing

Benton County, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								SILT, scattered organics, (ML); brown, low to medium plasticity, moist, medium stiff, organics consist of fine roots, blocky structure, (topsoil).
	1					0.35		
	2	S-2-1	■			0.50 0.60		Clayey SILT, (MH); light brown, iron-stained, medium plasticity, moist, stiff to very stiff, (Willamette Silt).
	3							
	4					1.20		
	5							
No seepage or ground water encountered to the limit of excavation.	6							
	7							
	8	S-2-2	■					
	9	S-2-3	■					Clayey SILT, trace to some sand and gravel, (MH); brown, medium plasticity, moist, stiff, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).
	10							
	11	S-2-4	■					Silty GRAVEL, some sand, (GM); brown, iron-stained, low to medium plasticity silt, moist, dense to very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).
	12							BOTTOM OF TEST PIT

Project No.: 2141009

Surface Elevation: 273.76 feet

Date of Test Pit: November 7, 2014

Test Pit Log: TP-2

SW 53rd Street Railroad Crossing

Benton County, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								Clayey SILT, trace gravel, scattered organics, (MH); brown, medium plasticity, moist to wet, medium stiff, fine to coarse, subrounded to rounded gravel, organics consist of fine roots, (topsoil).
	1					0.50		
	2	S-3-1	■					Clayey SILT, (MH); grey-brown, iron-stained, medium plasticity, moist to wet, stiff to very stiff, blocky structure, (Willamette Silt).
	3	S-3-2	■			0.80		
	4					1.20		
	5							
	6							
	7	S-3-3	■					
No seepage or ground water encountered to the limit of excavation. ±15 inch diameter PVC storm line was encountered at the south end of the test pit at ±9 feet.	8							
	9							BOTTOM OF TEST PIT
	10							
	11							
	12							

Project No.: 2141009

Surface Elevation: 262.74 feet

Date of Test Pit: November 7, 2014

Test Pit Log: TP-3

SW 53rd Street Railroad Crossing

Benton County, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.		S-4-1	■					SILT, scattered organics, (ML); brown, low to medium plasticity, damp, medium stiff to stiff, organics consist of fine to medium roots, blocky structure, (topsoil).
	1							
	2	S-4-2	■			1.40		Clayey SILT, (MH); light brown, iron-stained, medium plasticity, damp, stiff to very stiff, (Willamette Silt).
	3							
	4							
	5							
No seepage or ground water encountered to the limit of excavation.	6							
	7	S-4-3	■					
	8							
	9							
	10							Silty GRAVEL, trace sand, (GM); brown, iron-stained, low to medium plasticity silt, moist, dense to very dense, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).
	11	S-4-4 S-4-5	■					
	12							BOTTOM OF TEST PIT

Project No.: 2141009

Surface Elevation: 268.13 feet

Date of Test Pit: November 7, 2014

Test Pit Log: TP-4

SW 53rd Street Railroad Crossing

Benton County, Oregon

Appendix C

Laboratory Test Results

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Foundation Engineering, Inc.

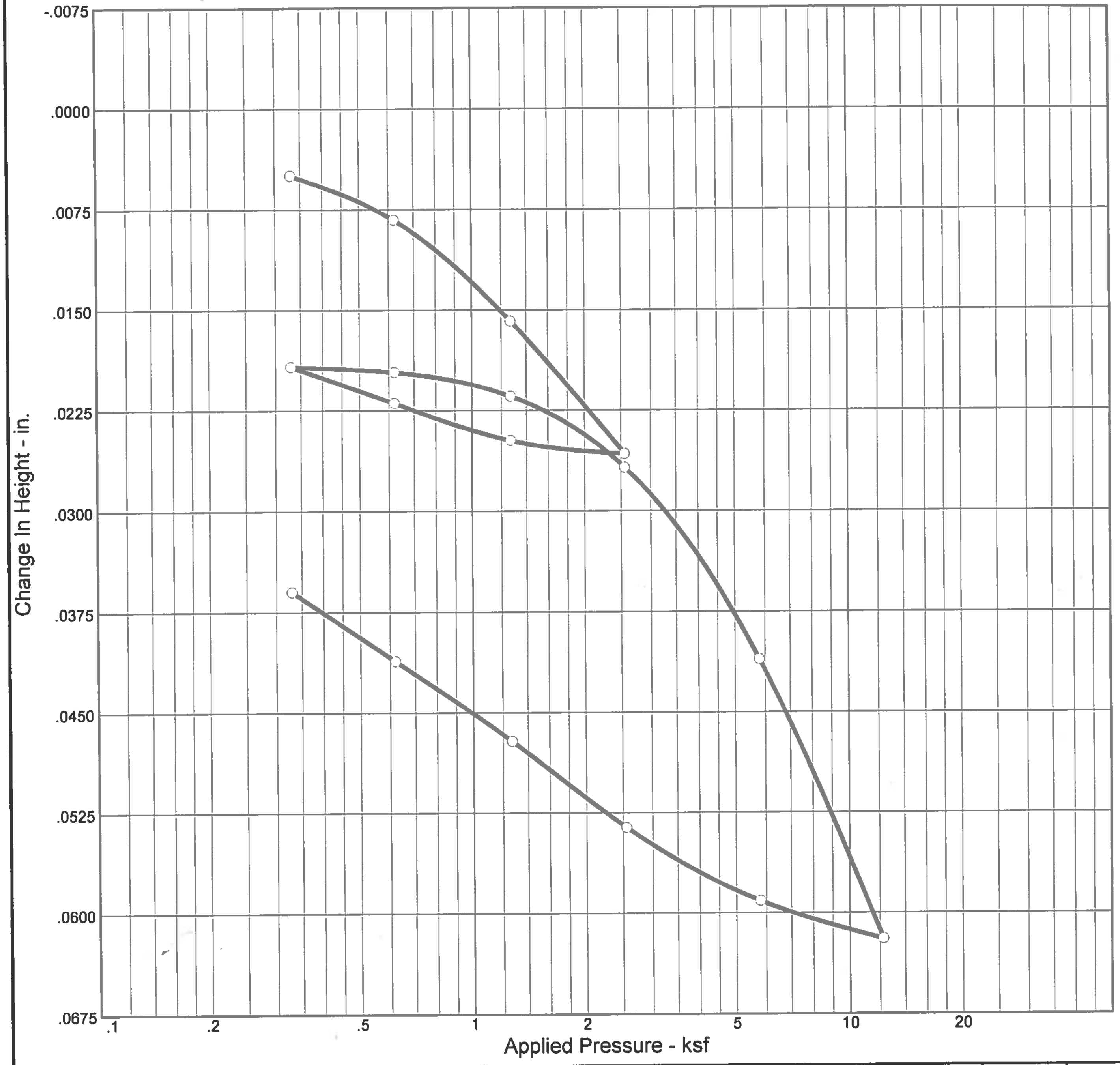
Table 1C. Atterberg Limits, Natural Water Contents, and Percent Fines

Sample Number	Sample Depth (feet)	Natural Water Content (percent)	LL	PL	PI	USCS Classification	Percent Fines
SS-1-3	7.5-9	28.7					
SS-1-4	10-11.5	34.4					
SS-1-5	12.5-14	36.3					
SS-1-6	15-16.5	21.1					
SS-1-7	20-21.5	25.3					
SH-2-1	2.5-4.5	31.8	52	33	19	MH	
SS-2-2	4.5-6	37.2					
SS-2-4	9.5-11	43.0					
SS-2-5	12.5-14	16.8					
SS-2-6	15-15.9	18.2					
SS-3-1	2.5-4	26.8					
SS-3-2	5-6.5	43.8					
SS-3-3	7.5-9	15.0					
SS-3-4	10-11.5	50.8					52.0
SS-3-5	12.5-14	48.0					
SS-3-6	15-16.5	18.5					
SS-3-7	20-21.5	21.9					
SS-3-8	25-26.5	24.7					
SH-4-2	4.5-6.5	39.4	57	34	23	MH	
SS-4-3	6.5-8	41.9					
SS-4-4	10-11.5	41.1					

Table 1C. Atterberg Limits, Natural Water Contents, and Percent Fines

Sample Number	Sample Depth (feet)	Natural Water Content (percent)	LL	PL	PI	USCS Classification	Percent Fines
SS-4-5	12.5-14	45.7					26.9
SS-4-6	15-16.5	26.8					
SS-4-8	20-21.5	23.2					
SS-4-9	25-26.3	19.8					
SH-5-2	5-7.0	34.6	51	28	23	MH-CH	
SS-5-3	7-8.5	43.6					
SS-5-4	10-11.5	25.0					

CONSOLIDATION TEST REPORT - ASTM D2435



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
82.5 %	23.1 %	95.0			2.65	0.41	2.10	0.15	0.03			0.742

MATERIAL DESCRIPTION	USCS	AASHTO
Light Brown Clayey SILT	MH	

Project No. 2146001-517 **Client:** Foundation Engineering Inc.; Project #2141009

Project: SW 53rd Street Railroad Crossing
Benton County, Oregon

Source: 5636 **Sample No.:** SH-2-1 **Elev./Depth:** 3.5'

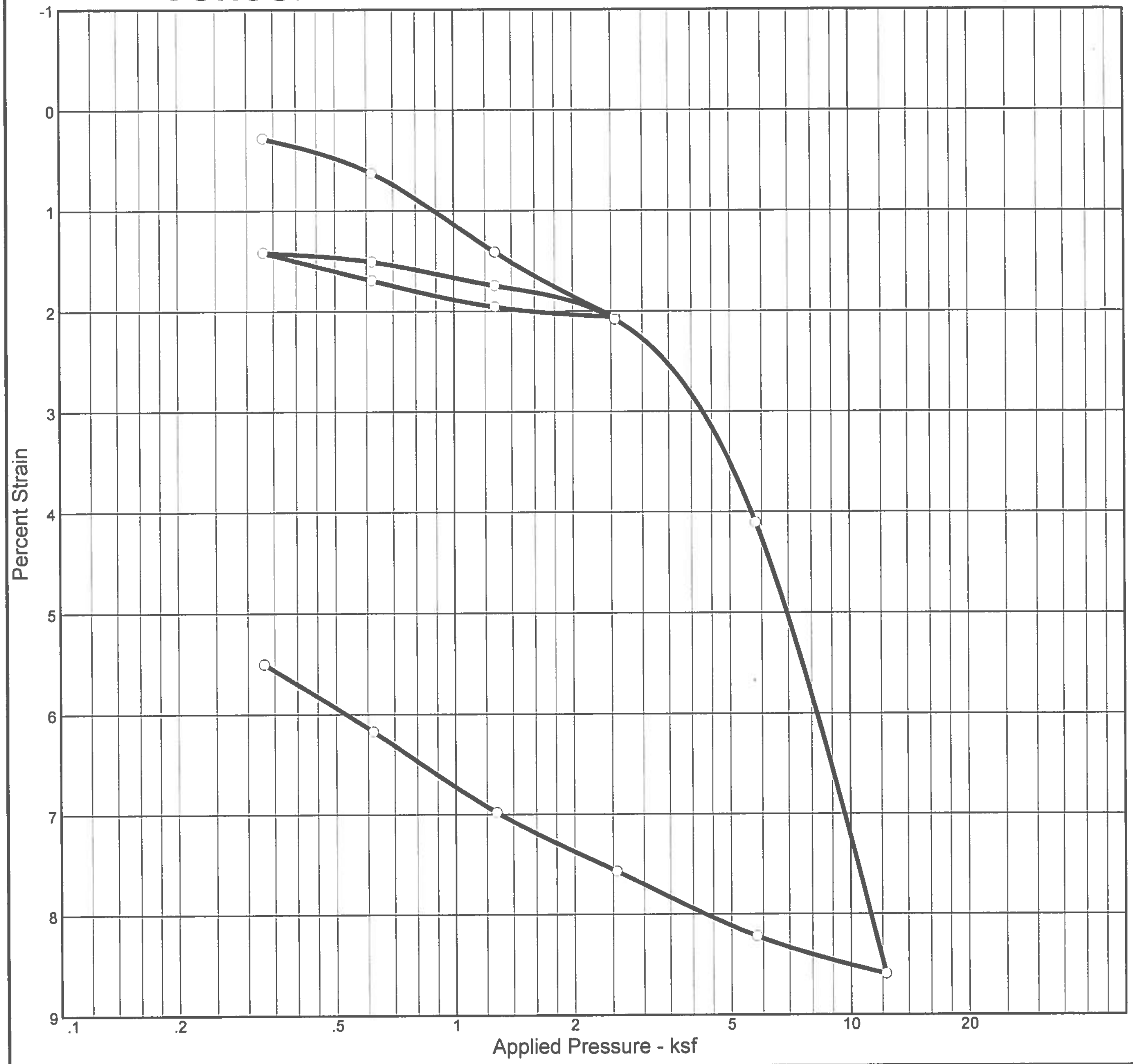
Remarks:

CONSOLIDATION TEST REPORT - ASTM D2435

FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 1C

CONSOLIDATION TEST REPORT - ASTM D2435



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
93.1 %	33.0 %	85.4			2.65	0.69	4.72	0.28	0.03			0.938

MATERIAL DESCRIPTION	USCS	AASHTO
Brown Clayey SILT	MH	

Project No. 2146001-517 **Client:** Foundation Engineering Inc.; Project #2141009

Project: SW 53rd Street Railroad Crossing
Benton County, Oregon

Source: 5636 **Sample No.:** SH-4-2 **Elev./Depth:** 6'

Remarks:

CONSOLIDATION TEST REPORT - ASTM D2435
FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 2C

Appendix D: Final Pavement Design Memorandum

Foundation Engineering, Inc.
Professional Geotechnical Services

Memorandum

Date: January 28, 2016

To: Anthony Calcagno, P.E.
David Evans and Associates, Inc.

From: David L. Running, P.E., G.E.
Senior Engineer

Subject: Pavement Design Memorandum

Project: SW 53rd Street Railroad Crossing
Benton County, Oregon
Project 2141009

REGISTERED PROFESSIONAL ENGINEER
62647PE
OREGON
NOV. 9, 1999
DAVID L. RUNNING
Expires: 12/31/16

This memorandum summarizes our analyses and design recommendations for the construction of new approach pavements for the above-referenced project.

BACKGROUND

A new bridge is planned crossing over the Portland & Western Railroad (PNWR) mainline tracks at SW 53rd Street in Corvallis, Oregon. The site location is shown on Figure 1A (attached).

SW 53rd Street currently crosses under the railroad tracks, which are supported on a timber trestle bridge. For the new crossing, the street will be shifted to the east of its current alignment and will cross over the tracks. At the planned crossing, the railroad tracks are laid on an embankment elevated ±10 feet above the surrounding terrain. New approach embankments up to ±44 feet tall and ±1,100 to 1,800 feet long will be required to raise the street above the existing track. Mechanically Stabilized Earth (MSE) retaining walls up to ±44 feet tall are planned to retain the approach fill at the abutments. The MSE walls will extend ±85 feet back from the abutments along the sides of the approaches parallel to the street. The new bridge will be a 60.9-foot wide by 113-foot long, single-span concrete structure.

Benton County is the project owner and David Evans and Associates, Inc. (DEA) is the prime designer. Foundation Engineering, Inc. was retained by DEA as the geotechnical consultant.

We completed an investigation in 2002/2003 for pavement reconstruction on SW 53rd Street extending from Harrison Boulevard south to the intersection with SW Reservoir Road. A portion of that project was adjacent to the proposed north approach for the new bridge. The findings were presented in a report dated April 8, 2003. Information from that investigation was used to supplement the current work.

FIELD EXPLORATION

The field exploration for the current project included five exploratory boreholes drilled between October 27 and 29, 2014, and four exploratory test pits dug on November 7, 2014. The exploration locations are shown on Figure 2A (attached). Discussions of the explorations and boring and test pit logs are provided in the Foundation Report dated October 19, 2015.

The previous explorations for SW 53rd Street included four test pits dug on the shoulders of 53rd Street. Two test pits were located adjacent to the planned north approach. The test pit locations are shown on Figure 1 (attached)

DISCUSSION OF SUBGRADE CONDITIONS

The borings and test pits completed along the planned alignment indicate the surficial soils typically consist of stiff to very stiff, medium plasticity clayey silt and silty clay (Willamette Silt). These soil conditions are consistent with the soils encountered in our explorations completed adjacent to the north approach in 2002, for the previous pavement reconstruction on SW 53rd Street. We anticipate these soil conditions will be representative of the subgrade at the north and south ends of the project, where the pavements will tie into the existing 53rd Street pavements.

The bridge approaches will be raised above the existing terrain, reaching a maximum height of ±44 feet at the abutments. The source of the new approach fill has not been established. Considering the large volume of material required, we anticipate the fill will be comprised of materials from various sources and may include fine-grained and granular soils. At the bridge abutments, we anticipate the subgrade will consist of predominantly MSE Granular Backfill.

SUBGRADE STRENGTH

For evaluating subgrade strength for pavement design, we assumed two scenarios.

- Pavements constructed on the native soil at the tie-in with the existing SW 53rd Street Pavement
- Pavements constructed on the new approach fill

Our 2002/2003 investigation for SW 53rd Street included two California Bearing Ratio (CBR) tests completed on subgrade samples. The test results indicate CBR values ranging from 3.3 to 3.9. These values correspond to resilient moduli (M_r) ranging from 4,950 psi to 5,850 psi (based on the AASHTO correlation $M_r = 150 \times CBR$). To account for potential variability, a M_r value of 4,500 psi was used for the designing approach pavements at the tie-in to the existing 53rd Street pavement.

As previously noted, the source and type of material used to construct the approaches have not been established. We anticipate the fill will include both fine-grained and granular soil. We assumed a M_r value of 6,000 psi for evaluating the minimum thicknesses of pavements built on the approach fill. This value will be conservative where granular fill is used, particularly at the abutments where the subgrade will consist of MSE Granular Backfill.

TRAFFIC DATA

Available traffic included a detailed breakdown of traffic distributions from November 2002 and January 2003. Additionally, the County provided an Average Daily Traffic (ADT) of 11,518 vehicles with 6.46% trucks recorded in 2012. The 2002/2003 and 2012 ADT values were used to calculate an annual growth rate of ±0.71%. The County indicated the truck traffic percentage may increase to ±8 to 10% after the new bridge is built, since the current height restrictions on SW 53rd Street will be eliminated.

We estimated a design traffic using the 2012 ADT along with an assumed vehicle distribution based on the average of the 2002/2003 data to estimate the design traffic. The traffic was adjusted to include 8% and 10% trucks, by adding trucks to the original traffic counts. The annual growth rate was used to project the traffic into the future. The available ADT values include two-way traffic. For design, we assumed 55% of the two-way ADT's to reflect the directional ADT for one-way traffic.

We assumed a start date of 2018 and assumed a 30-year design for pavements within 200 feet of the bridge abutments, as recommended in the ODOT Pavement Design Guide (2011). We assumed a 20-year design for pavements more than 200 feet away from the bridge abutments. The assumed traffic and design calculations are summarized on the attached calculation sheets.

PAVEMENT DESIGN

For pavement design, we used the ODOT (2011) procedure (based on AASHTO 1993) and assumed the following parameters:

- reliability of 85%
- overall deviation of 0.49
- initial serviceability of 4.2
- terminal serviceability of 2.5
- layer coefficient of 0.42 for new AC
- layer coefficient of 0.10 for Base Aggregate
- subgrade resilient modulus, M_r, of 4,500 psi (at the tie-in with 53rd Street)
- subgrade resilient modulus, M_r, of 6,000 psi (on the approach fill)
- drainage coefficient of 1.0
- 30-year design life (within 200 feet of the bridge abutments)
- 20-year design life (further than 200 feet from the abutments)

The following steps were taken to determine the minimum pavement section:

1. The required structural number (S_N) for the AC surface course was determined based on the design traffic and the ODOT-recommended resilient modulus of 20,000 psi for the Base Aggregate. The AC thickness was determined assuming a layer coefficient of 0.42 and a drainage coefficient of 1.0.

2. The required S_N for the Base Aggregate was determined by subtracting the S_N for the AC (Step 1) from the total required S_N, for the pavement section. The minimum thickness of the Base Aggregate was calculated assuming a layer coefficient of 0.10 and drainage coefficient of 1.0 for the Base Aggregate and the subgrade resilient modulus values listed above.

The calculations (attached) indicate the minimum pavement sections summarized in Table 1. We assume the County will select the appropriate pavement sections based on the anticipated truck traffic. For each section, a Subgrade Geotextile is recommended to provide separation between the base rock and subgrade. The Subgrade Geotextile may be eliminated where the subgrade consists of MSE Granular Wall Backfill or relatively clean granular fill.

Table 1. Minimum Pavement Sections

Percent Truck Traffic	8%		10%	
Location	AC Thickness (in)	Base Thickness (in)	AC Thickness (in)	Base Thickness (in)
Tie-In with 53rd Street (M_r = 4,500 psi - 20-yr Design)	7.5	22	8	22
Approach Embankment (M_r = 6,000 psi - 20-yr Design)	7.5	17	8	17
Approach Embankment within 200 feet of Abutments (M_r = 6,000 psi - 30-yr Design)	8	18	8.5	18

RECOMMENDATIONS

All specifications contained herein refer to ODOT's Oregon Standard Specifications for Construction (2015). It is also assumed these specifications will be referred to for general or specific items not addressed in this memorandum.

Based on the ODOT (2011) guidelines, the following pavement sections and mix designs are recommended for the new approach pavements, unless local County practice or experience warrants modifications.

- 2-inch thick (minimum) Wearing Course of Level 2, ½-inch Dense-Graded HMAC with PG 64-22 binder
- 2 to 3-inch thick lifts of Level 2, ½-inch or ¾-inch Dense-Graded HMAC Base Course, with PG 64-22 binder
- 1 inch – 0 Dense-Graded Base Aggregate

Section 10.4 (Table 5) of the ODOT (2011) guidelines indicates the project location does not mandate the use of anti-stripping additives in the HMAC.

The 1 inch – 0 Base Aggregate should conform to the material requirements of Section 02630 and grading requirements of Table 02630-1.

The Subgrade Geotextile should be a woven geotextile meeting the material requirements in Table 02320-4.

We recommend moisture-conditioning and compacting the subgrade prior to paving in accordance with Section 00330.43. The finished subgrade should be proof-rolled with a loaded dump truck or other approved heavy construction vehicle prior to placing the Base Aggregate to identify any soft areas. Any soft or pumping subgrade should be reworked or overexcavated and replaced with Base Aggregate.

LIMITATIONS

The analysis, conclusions and recommendations contained herein assume the subsurface profiles encountered in the borings and test and imported fill assumptions are representative of the site conditions within the identified construction limits. The above recommendations assume we will have the opportunity to review final drawings and be present during construction. No changes in the enclosed recommendations should be made without our approval. We will assume no responsibility or liability for any engineering judgment, inspection or testing performed by others.

This memorandum was prepared for the exclusive use of David Evans and Associates and Benton County Public Works for the design of the approach pavements as part of the SW 53rd Railroad Crossing project in Benton County, Oregon. Information contained herein should not be used for other sites or for unanticipated construction without our written consent.

This report is intended for planning and design purposes. Contractors using this information to estimate construction quantities or costs do so at their own risk. Our services do not include any survey or assessment of potential surface contamination or contamination of the soil or ground water by hazardous or toxic materials. We assume those services, if needed, have been completed by others.

We trust this information meets your present needs. Please do not hesitate to call if you have questions.

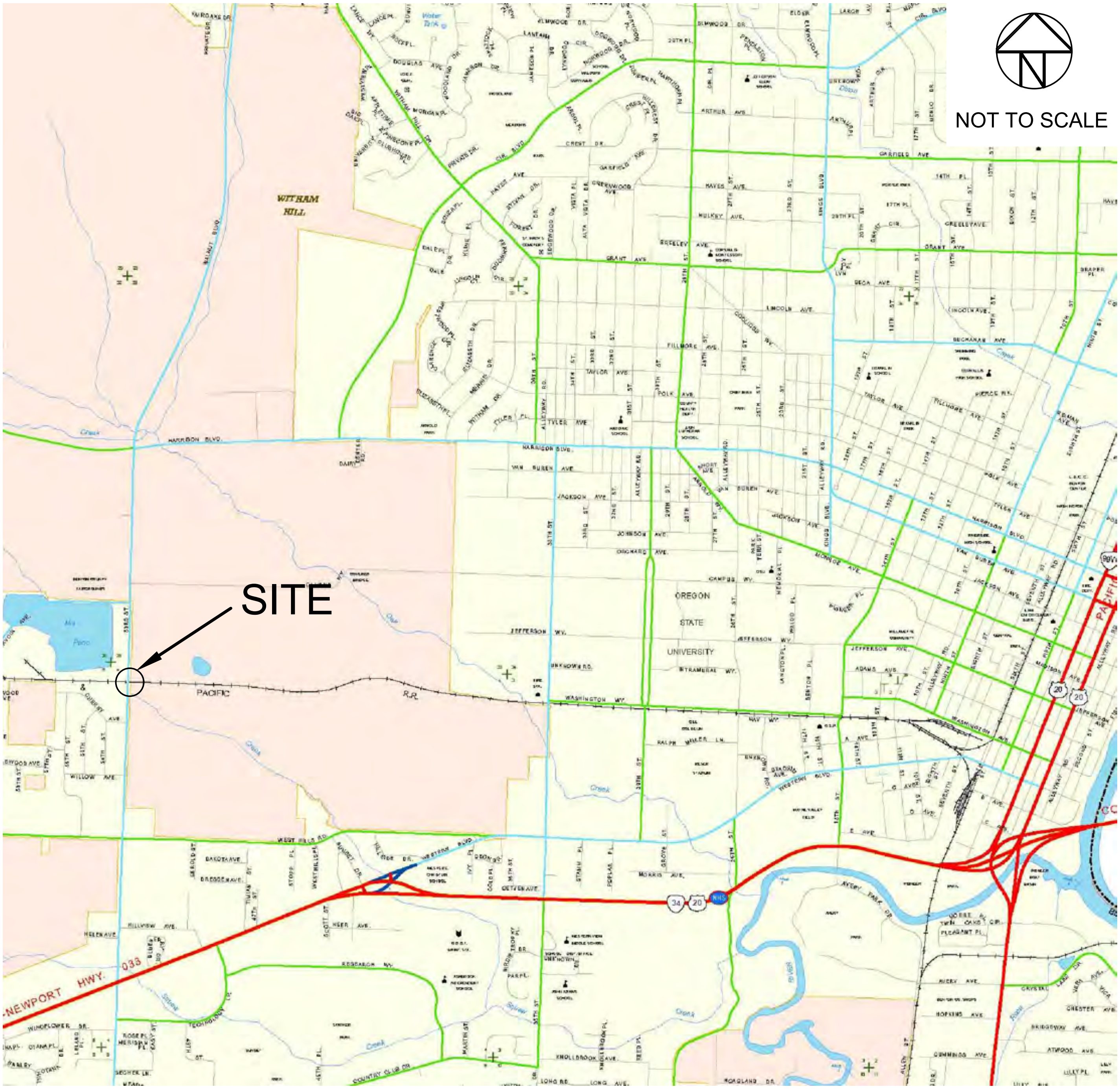
DLR/wg
Attachments

REFERENCES

American Association of State Highway and Transportation Officials (1993); AASHTO Guide for Design of Pavement Structures.

Oregon Department of Transportation, Highway Division (2015); Oregon Standard Specifications for Construction*.*

Oregon Department of Transportation, Pavement Services Unit (August 2011); ODOT Pavement Design Guide*.*



BASEMAP PROVIDED BY OREGON DEPARTMENT OF TRANSPORTATION (2011)
http://www.oregon.gov/ODOT/TD/TDATA/gis/odotmaps.shtml

DATE DEC. 2014
DWN. BZH
APPR.
REVIS.
PROJECT NO.
2141009

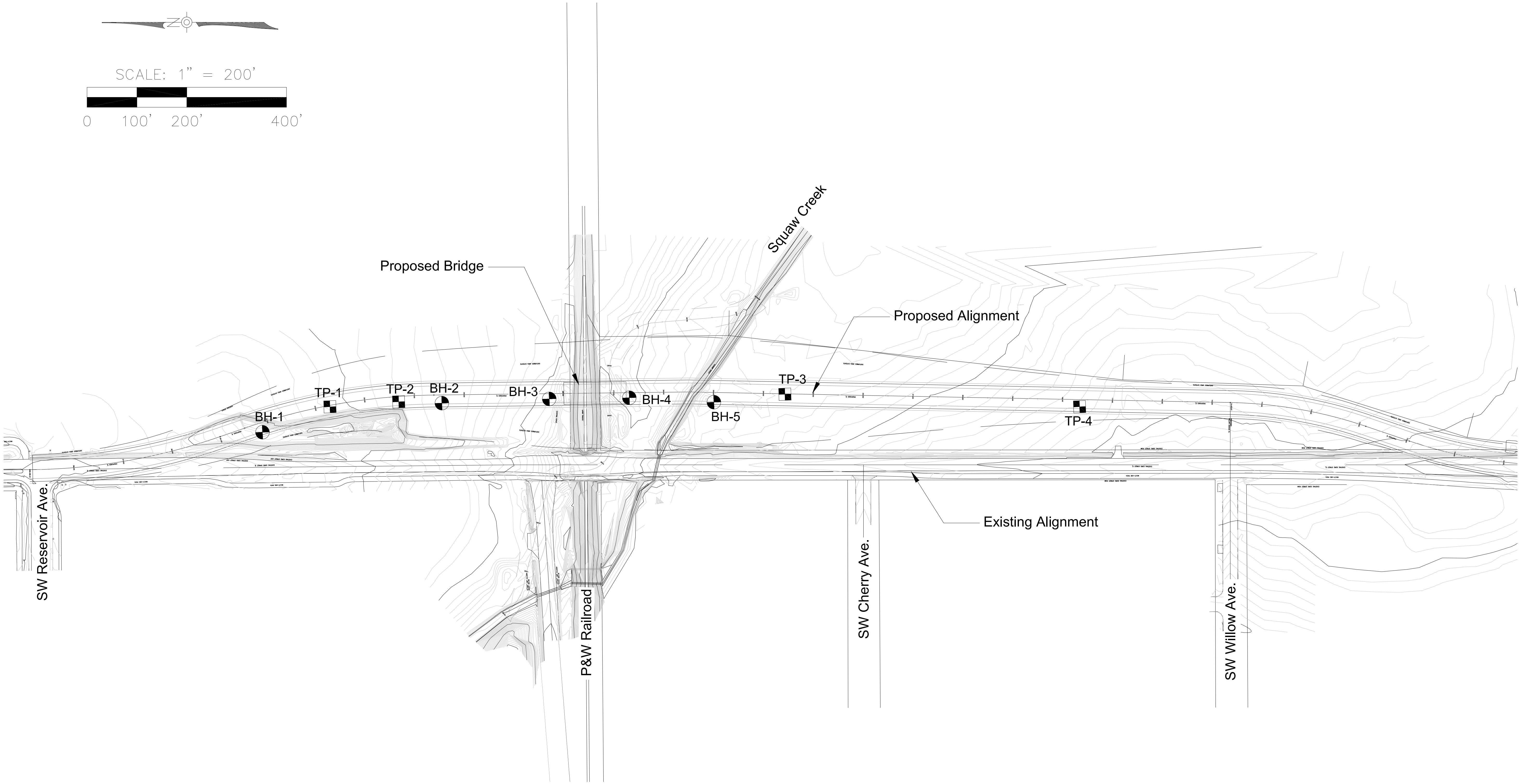
FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 NW CORNELL AVENUE
CORVALLIS, OR 97330-4517
BUS. (541) 757-7645 FAX (541) 757-7650

VICINITY MAP

SW 53rd STREET RAILROAD CROSSING
BENTON COUNTY, OREGON

FIGURE NO.
1A

FILE NAME: FIG 1A



NOTES:

1. EXPLORATION LOCATIONS ARE APPROXIMATE ONLY.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY BENTON COUNTY PUBLIC WORKS DEPARTMENT.

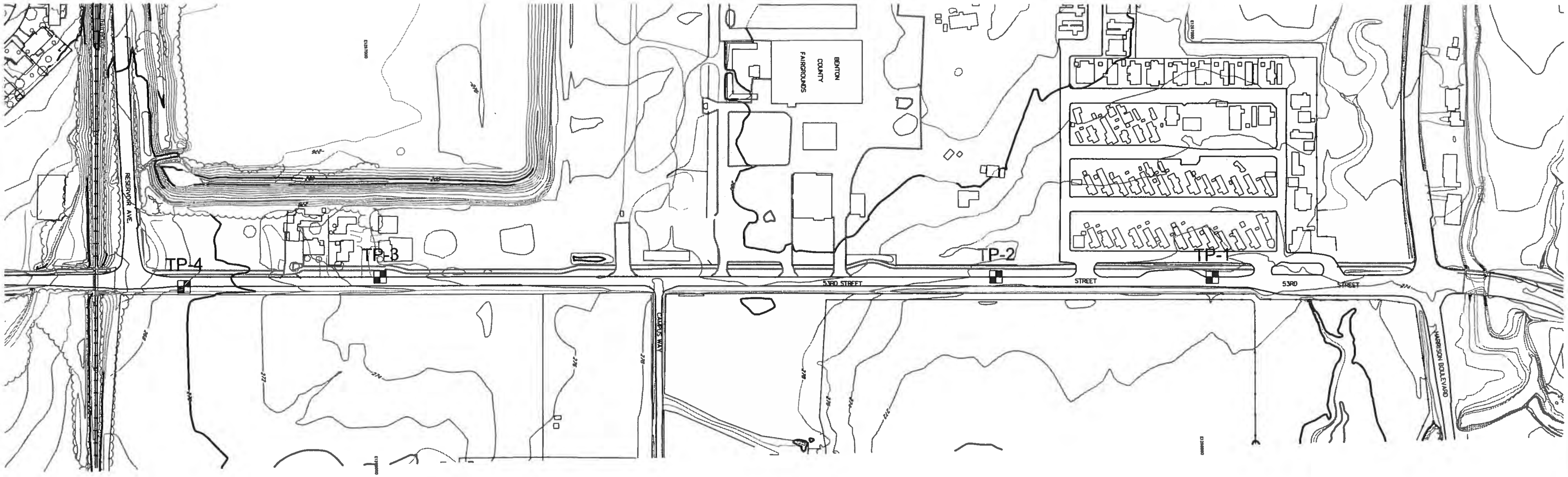
FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 NW CORNELL AVENUE
CORVALLIS, OR 97330-4517
BUS. (541) 757-7645 FAX (541) 757-7650

DATE OCT. 2015
DWN. MDM
APPR.
REVIS.
PROJECT NO.
2141009

EXISTING AND PROPOSED ALIGNMENTS
AND EXPLORATION LOCATIONS

SW 53RD STREET RAILROAD CROSSING
BENTON COUNTY, OREGON

FIGURE NO.
2A



SCALE 1" = 250'

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 NW CORNELL AVENUE
CORVALLIS, OR 97330-4517
BUS. (541) 757-7645 FAX (541) 757-7650

DATE APR. 2003
DWN. JMM
APPR.
REVIS.
PROJECT NO.
2021117

SITE MAP
EXPLORATION LOCATIONS
53RD STREET IMPROVEMENTS
CORVALLIS, OREGON

FIGURE NO.
1

53rd Street Railroad Crossing
Foundation Engineering, Inc.
Project 2141009

AVAILABLE TRAFFIC DATA

Based on Traffic Count from Nov. 18, to Nov. 24, 2002

Total Traffic Count = 10,163 veh/day

Breakdown Vehicle Type	%	Two-Way ADT
motorcycles/bicycles	0.32	33
passenger cars	76.20	7744
other 2-axle, 4 wheel vehicles	21.28	2163
buses	0.01	2
2-axle, 6-tire single trailer trucks	0.62	63
3-axle, single unit trucks	0.39	40
4-axle, single unit trucks	0.02	2
4 or less axle, single trailer trucks	0.29	30
5-axle, single trailer trucks	0.68	69
6 or more axle, single trailer trucks	0.02	3
5-axle, multi-trailer trucks	0.05	5
6-axle, multi-trailer trucks	0.03	3
all other vehicles	0.08	8
total trucks	2.20	223
total vehicles	100.0	10163

Based on Traffic Count from Jan. 7, to Jan. 10, 2003

Total Traffic Count = 11,299 veh/day

Breakdown Vehicle Type	%	Two-Way ADT
motorcycles/bicycles	1.5	170
passenger cars	72.6	8204
other 2-axle, 4 wheel vehicles	19.0	2152
buses	0.1	12
2-axle, 6-tire single trailer trucks	0.6	68
3-axle, single unit trucks	0.9	102
4-axle, single unit trucks	0.1	12
4 or less axle, single trailer trucks	2.1	238
5-axle, single trailer trucks	0.1	12
6 or more axle, single trailer trucks	0.1	12
5-axle, multi-trailer trucks	0.7	80
6-axle, multi-trailer trucks	0.3	34
all other vehicles	1.8	204
total trucks	6.85	773
	100.0	11299

Average
10,731 veh/day

Ave. %	Two-Way ADT
0.95	101
74.31	7974
20.10	2157
0.06	7
0.61	66
0.66	71
0.06	7
1.25	134
0.38	40
0.07	7
0.39	42
0.17	19
0.99	106
4.64	334
100.0	10731

Based on Traffic Count on May 6, 2012

Location: MP 0.41 (just north of intersection with West Hills Road)
Two Day Count
One Way ADT = 5,759 vehicles
Est. Two Way ADT = 11,518 vehicles
Truck Percentage = 6.46%
County indicates truck percentage may go up (possibly to 8 or 10%) after the overhead clearance issue is removed.
Assume 8% trucks for design.

Calculate Annual Growth Rate

2002/2003 Average ADT = 10731
2012 ADT = 11518

10-yr Expansion Factor (E) = 1.07
Annual Growth Rate (R) = 0.71 %

Adjust 2002/2003 Traffic Breakdown to 2012 and Increase to 8% Trucks

	2002/2003 Ave. ADT 10731		Adjust for 2012 ADT 6.46% Trucks 11518		Adjust 2012 ADT 8.0% Trucks		Adjust for 2012 ADT 10.0% Trucks 11518	
Breakdown FHWA Vehicle Type	Two-Way ADT	%	Two-Way ADT		Two-Way ADT	%	Two-Way ADT	%
1. Motorcycles/bicycles	101	0.95%	107	0.93%	107	0.91%	107	0.89%
2 Passenger Cars	7974	74.31%	8390	72.84%	8390	71.65%	8390	70.09%
3. Pickup and other 2-axle, 4 Tire Trucks	2157	20.10%	2270	19.71%	2270	19.38%	2270	18.96%
4. Buses (RVs)	7	0.06%	9	0.08%	9	0.08%	9	0.08%
5. 2-Axle/6-Tire Trucks	66	0.61%	99	0.86%	125	1.07%	160	1.33%
6. 3 Axle Single Unit Trucks	71	0.66%	107	0.93%	135	1.15%	173	1.44%
7. 4 Axle (or more) Single Unit Trucks	7	0.06%	10	0.09%	13	0.11%	16	0.14%
8. 3 to 4-Axle Single Trailer	134	1.25%	202	1.75%	254	2.17%	325	2.72%
9. 5 Axle Single Trailer	40	0.38%	61	0.53%	76	0.65%	98	0.82%
10. 6 (or more) Axle Single Trailer	7	0.07%	11	0.09%	14	0.12%	17	0.15%
11. 5 Axle Double Trailers	42	0.39%	64	0.55%	80	0.69%	103	0.86%
12. 6+ Axle Double Trailers	19	0.17%	28	0.24%	35	0.30%	45	0.38%
13. 7+ Axle Double Trailers	106	0.99%	160	1.39%	201	1.72%	258	2.15%
		100.0%		100.0%		100.0%		100.0%
Two-Way ADT =	10731		11518		11710		11971	
Truck ADT =	492		742		934		1195	
% trucks =	4.6%		6.45%		8.0%		10.0%	

Adjustment to 8% trucks assumes trucks are added and increase the ADT.

By DR 1/26/16
✓ By JCH 1/26/16

AASHTO (1993) Flexible Pavement Design - Calculation of Design ESALs (8% Trucks)

8% Trucks

2012 Traffic - Adjusted for 8% Trucks

Total ADT= 11710 vehicles
Lane Distribution = 55%
Design ADT (per lane)= 6440.5 vehicles
%Trucks = 8.0%

Assumed Breakdown		Two-Way ADT	One-Way ADT	Rounded ADT	*Flexible Pavement* Annual ESAL Factor	Annual ESALs	FHWA Classification
91.9%	0.91%	107	59	59	0	0	1. Motorcycles
	71.65%	8390	4615	4615	0.3	1385	2. Passenger Cars
	19.38%	2270	1249	1249	4.5	5621	3. Pickups (4-tire, single unit)
8.1%	0.08%	9	5	5	246	1230	4. Buses (RV's)
	1.07%	125	69	69	104	7176	5. 2-axle, 6-tire single unit
	1.15%	135	74	74	284	21016	6. 3-axle, single unit
	0.11%	13	7	7	757	5299	7. 4-axle (or more), single unit
	2.17%	254	140	140	253	35420	8. 4-axle (or less), single trailer
	0.65%	76	42	42	466	19572	9. 5-axle, single trailer
	0.12%	14	7	7	561	3927	10. 6-axle (or more), single trailer
	0.69%	80	44	44	603	26532	11. 5-axle (or less), multi-trailer
	0.30%	35	19	19	546	10374	12. 6-axle, multitrailer
	1.72%	201	111	111	1037	115107	13. 7-axle (or more), multi-trailer
Total =	100.0%	11710	6441	6441		252658	

Estimated Traffic

2002/2003 Average ADT = 10731
2012 ADT = 11518
10-yr Expansion Factor (E) = 1.07
Annual Growth Rate (R) = 0.71

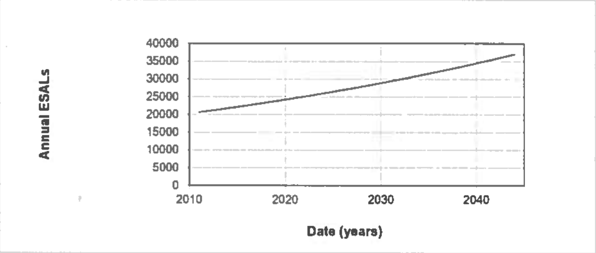
Available ADT=	2012	ADT =	11710
Projected Start Year ADT =	2018	ADT =	12218
Projected 20-Year ADT =	2038	ADT =	14076
Projected 30-Year ADT =	2048	ADT =	15108

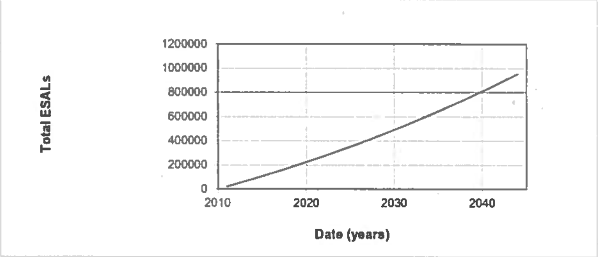
1. 2012 ADT and 8% trucks based on data provided by Benton County.
2. Truck percentage breakdown is based on 2002 and 2003 traffic counts adjusted for 8% trucks.
3. Annual ESAL factors are based on the ODOT Pavement Design Guide (2011).

Start Year = **2018**
Flexible Pavement Cummulative Traffic for:

	20-Year	30-Year
Initial ESAL's =	2,339,607	2,339,607
Final ESAL's =	8,104,455	11,307,708
Design ESAL's =	**5,764,848**	**8,968,102**

Year	Flexible Pavement Annual ESAL's	Sum	
2010	252,658	252,658	
2011	254,453	507,111	
2012	256,260	763,370	
2013	258,080	1,021,450	
2014	259,913	1,281,363	
2015	261,759	1,543,122	
2016	263,618	1,806,740	
2017	265,490	2,072,230	
2018	267,376	**2,339,607**	start
2019	269,275	2,608,882	
2020	271,188	2,880,069	
2021	273,114	3,153,183	
2022	275,054	3,428,237	
2023	277,007	3,705,244	
2024	278,975	3,984,218	
2025	280,956	4,265,175	
2026	282,952	4,548,126	
2027	284,961	4,833,087	
2028	286,985	5,120,072	
2029	289,023	5,409,096	
2030	291,076	5,700,172	
2031	293,144	5,993,316	
2032	295,226	6,288,542	
2033	297,323	6,585,864	
2034	299,434	6,885,298	
2035	301,561	7,186,859	
2036	303,703	7,490,562	
2037	305,860	7,796,422	
2038	308,032	**8,104,455**	20-year
2039	310,220	8,414,675	
2040	312,423	8,727,098	
2041	314,642	9,041,741	
2042	316,877	9,358,618	
2043	319,128	9,677,746	
2044	321,395	9,999,140	
2045	323,677	10,322,818	
2046	325,976	10,648,794	
2047	328,291	10,977,085	
2048	330,623	**11,307,708**	30-year
2049	332,971	11,640,680	
2050	335,336	11,976,016	





AASHTO (1993) Flexible Pavement Design - Minimum AC Thickness (20-yr Design - 8% Trucks)

(Assumes AC is over a Base Course Layer with a M_r of 20,000 psi)

Structural Number, SN_{req}	3.104	
Design ESAL's (W_{18})	5,765,000	5,765,000
Reliability (%)	85	Collector
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	**20,000**	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	
Standard Normal Deviate, Z_R	-1.036433	
Δ PSI	1.7	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	**3.15**
AC Layer Coefficient, a_{AC}	0.42
AC Drainage Coefficient, m_{AC}	1.00
Base Layer Coefficient, a_{base}	
Base Drainage Coefficient, m_{base}	

Layer Thickness

AC 7.5 in

$$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$$

AASHTO (1993) Flexible Pavement Design - Minimum AC and Base Thickness (20-yr Design - 8% Trucks)

(Assumes Conventional AC and Base Rock over subgrade with a M_r of 4,500 psi)

Structural Number, SN_{req}	**5.303**	
Design ESAL's (W_{18})	5,765,000	5,765,000
Reliability (%)	85	
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	**4,500**	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	
Standard Normal Deviate, Z_R	-1.03643339	
Δ PSI	1.7	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	5.35
AC Layer Coefficient, a_{AC}	0.42
AC Drainage Coefficient, m_{AC}	1.00
Base Layer Coefficient, a_{base}	0.1
Base Drainage Coefficient, m_{base}	1.0

Layer Thickness

AC	7.5 in
Base Rock	22 in

$$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$$

AASHTO (1993) Flexible Pavement Design - Minimum AC and Base Thickness (20-yr Design - 8% Trucks)

(Assumes Conventional AC and Base Rock over subgrade with a M_r of 6,000 psi)

Structural Number, SN_{req}	**4.823**	
Design ESAL's (W_{18})	5,765,000	5,765,000
Reliability (%)	85	
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	**6,000**	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

Standard Normal Deviate, Z_R	-1.03643339
Δ PSI	1.7

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	**4.85**
AC Layer Coefficient, a_{AC}	0.42
AC Drainage Coefficient, m_{AC}	1.00
Base Layer Coefficient, a_{base}	0.1
Base Drainage Coefficient, m_{base}	1.0

Layer Thickness

AC	7.5 in
Base Rock	17 in

$$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$$

AASHTO (1993) Flexible Pavement Design - Minimum AC Thickness (30-yr Design - 8% Trucks)

(Assumes AC is over a Base Course Layer with a M_r of 20,000 psi)

Structural Number, SN_{req}	**3.339**	
Design ESAL's (W_{18})	8,968,100	8,968,100
Reliability (%)	85	Collector
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	**20,000**	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	
Standard Normal Deviate, Z_R	-1.0364334	
Δ PSI	1.7	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	**3.36**
AC Layer Coefficient, a_{AC}	0.42
AC Drainage Coefficient, m_{AC}	1.00
Base Layer Coefficient, a_{base}	
Base Drainage Coefficient, m_{base}	

Layer Thickness

AC 8 in

$$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$$

AASHTO (1993) Flexible Pavement Design - Minimum AC and Base Thickness (30-yr Design - 8% Trucks)

(Assumes Conventional AC and Base Rock over subgrade with a M_r of 6,000 psi)

Structural Number, SN_{req}	**5.138**	
Design ESAL's (W_{18})	8,968,100	8,968,100
Reliability (%)	85	
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	**6,000**	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	
Standard Normal Deviate, Z_R	-1.0364334	
Δ PSI	1.7	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	**5.16**	***Layer Thickness***	
AC Layer Coefficient, a_{AC}	0.42	***AC***	8 in
AC Drainage Coefficient, m_{AC}	1.00	***Base Rock***	18 in
Base Layer Coefficient, a_{base}	0.1		
Base Drainage Coefficient, m_{base}	1.0		

$$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$$

AASHTO (1993) Flexible Pavement Design - Calculation of Design ESALs (10% Trucks)

10 % Trucks

2012 Traffic - Adjusted for 10% Trucks

Total ADT= 11971 vehicles
Lane Distribution = 55%
Design ADT (per lane)= 6584.05 vehicles
%Trucks = 10.0%

Assumed Breakdown		Two-Way ADT	One-Way ADT	Rounded ADT	*Flexible Pavement* Annual ESAL Factor	Annual ESALs	FHWA Classification
89.9%	0.89%	107	59	59	0	0	1. Motorcycles
	70.09%	8390	4615	4615	0.3	1385	2. Passenger Cars
	18.96%	2270	1249	1249	4.5	5621	3. Pickups (4-tire, single unit)
10.1%	0.08%	9	5	5	246	1230	4. Buses (RV's)
	1.34%	160	88	88	104	9152	5. 2-axle, 6-tire single unit
	1.45%	173	95	95	284	26980	6. 3-axle, single unit
	0.13%	16	9	9	757	6813	7. 4-axle (or more), single unit
	2.71%	325	179	179	253	45287	8. 4-axle (or less), single trailer
	0.82%	98	54	54	466	25164	9. 5-axle, single trailer
	0.14%	17	9	9	561	5049	10. 6-axle (or more), single trailer
	0.86%	103	57	57	603	34371	11. 5-axle (or less), multi-trailer
	0.38%	45	25	25	546	13650	12. 6-axle, multitrailer
	2.16%	258	142	142	1037	147254	13. 7-axle (or more), multi-trailer
Total =	100.0%	11971	6584	6586		321955	

Estimated Traffic

2002/2003 Average ADT = 10731
2012 ADT = 11518
10-yr Expansion Factor (E) = 1.07
Annual Growth Rate (R) = 0.71

Available ADT=	2012	ADT =	11971
Projected Start Year ADT =	2018	ADT =	12490
Projected 20-Year ADT =	2038	ADT =	14390
Projected 30-Year ADT =	2048	ADT =	15445

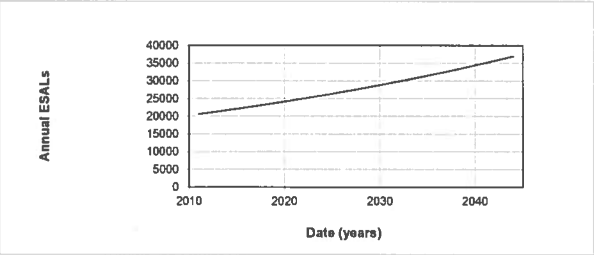
1. 2012 ADT and 10% trucks based on data provided by Benton County.
2. Truck percentage breakdown is based on 2002 and 2003 traffic counts adjusted for 10% trucks.
3. Annual ESAL factors are based on the ODOT Pavement Design Guide (2011).

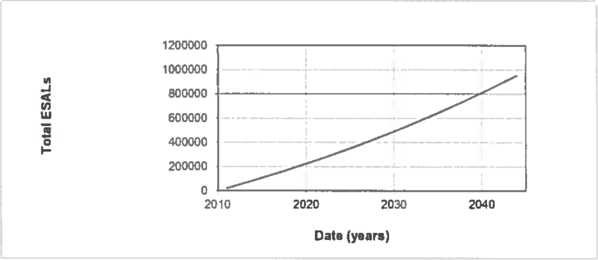
Year	Flexible Pavement Annual ESAL's	Sum	
2010	321,955	321,955	
2011	324,242	646,197	
2012	326,545	972,741	
2013	328,864	1,301,605	
2014	331,200	1,632,805	
2015	333,552	1,966,357	
2016	335,921	2,302,278	
2017	338,307	2,640,585	
2018	340,710	**2,981,295**	start
2019	343,130	3,324,425	
2020	345,567	3,669,992	
2021	348,021	4,018,013	
2022	350,493	4,368,506	
2023	352,982	4,721,488	
2024	355,490	5,076,978	
2025	358,014	5,434,992	
2026	360,557	5,795,549	
2027	363,118	6,158,667	
2028	365,697	6,524,365	
2029	368,295	6,892,659	
2030	370,910	7,263,569	
2031	373,545	7,637,114	
2032	376,198	8,013,312	
2033	378,870	8,392,182	
2034	381,561	8,773,743	
2035	384,271	9,158,013	
2036	387,000	9,545,013	
2037	389,749	9,934,762	
2038	392,517	**10,327,279**	20-year
2039	395,305	10,722,584	
2040	398,112	11,120,697	
2041	400,940	11,521,637	
2042	403,788	11,925,424	
2043	406,656	12,332,080	
2044	409,544	12,741,624	
2045	412,453	13,154,077	
2046	415,382	13,569,459	
2047	418,333	13,987,792	
2048	421,304	**14,409,095**	30-year
2049	424,296	14,833,392	
2050	427,310	15,260,701	

Start Year = **2018**

Flexible Pavement Cummulative Traffic for:

	20-Year	30-Year
Initial ESAL's =	2,981,295	2,981,295
Final ESAL's =	10,327,279	14,409,095
Design ESAL's =	**7,345,984**	**11,427,801**





AASHTO (1993) Flexible Pavement Design - Minimum AC Thickness (20-yr Design - 10% Trucks)

(Assumes AC is over a Base Course Layer with a M_r of 20,000 psi)

Structural Number, SN_{req}	**3.231**	
Design ESAL's (W_{18})	7,346,000	7,346,000
Reliability (%)	85	Collector
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	**20,000**	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	
Standard Normal Deviate, Z_R	-1.036433	
Δ PSI	1.7	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	**3.36**	***Layer Thickness***	
AC Layer Coefficient, a_{AC}	0.42	*AC*	8 in
AC Drainage Coefficient, m_{AC}	1.00		
Base Layer Coefficient, a_{base}			
Base Drainage Coefficient, m_{base}			

$$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$$

AASHTO (1993) Flexible Pavement Design - Minimum AC and Base Thickness (20-yr Design - 10% Trucks)

(Assumes Conventional AC and Base Rock over subgrade with a M_r of 4,500 psi)

Structural Number, SN_{req}	5.484	
Design ESAL's (W_{18})	7,346,000	7,346,000
Reliability (%)	85	
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	4,500	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	
Standard Normal Deviate, Z_R	-1.03643339	
Δ PSI	1.7	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	5.56
AC Layer Coefficient, a_{AC}	0.42
AC Drainage Coefficient, m_{AC}	1.00
Base Layer Coefficient, a_{base}	0.1
Base Drainage Coefficient, m_{base}	1.0

Layer Thickness

AC	8 in
Base Rock	22 in

$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$

AASHTO (1993) Flexible Pavement Design - Minimum AC and Base Thickness (20-yr Design - 10% Trucks)

(Assumes Conventional AC and Base Rock over subgrade with a M_r of 6,000 psi)

Structural Number, SN_{req}	**4.995**	
Design ESAL's (W_{18})	7,346,000	$\boxed{7,346,000}$
Reliability (%)	85	
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	**6,000**	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	
Standard Normal Deviate, Z_R	-1.03643339	
Δ PSI	1.7	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	**5.06**	***Layer Thickness***	
AC Layer Coefficient, a_{AC}	0.42	***AC***	8 in
AC Drainage Coefficient, m_{AC}	1.00	***Base Rock***	17 in
Base Layer Coefficient, a_{base}	0.1		
Base Drainage Coefficient, m_{base}	1.0		

$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$

AASHTO (1993) Flexible Pavement Design - Minimum AC Thickness (30-yr Design - 10% Trucks)

(Assumes AC is over a Base Course Layer with a M_r of 20,000 psi)

Structural Number, SN_{req}	3.475	
Design ESAL's (W_{18})	11,427,800	11,427,800
Reliability (%)	85	Collector
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	**20,000**	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	
Standard Normal Deviate, Z_R	-1.0364334	
Δ PSI	1.7	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	**3.57**
AC Layer Coefficient, a_{AC}	0.42
AC Drainage Coefficient, m_{AC}	1.00
Base Layer Coefficient, a_{base}	
Base Drainage Coefficient, m_{base}	

Layer Thickness

AC 8.5 in

$$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$$

AASHTO (1993) Flexible Pavement Design - Minimum AC and Base Thickness (30-yr Design - 10% Trucks)

(Assumes Conventional AC and Base Rock over subgrade with a M_r of 6,000 psi)

Structural Number, SN_{req}	5.316	
Design ESAL's (W_{18})	11,427,800	11,427,800
Reliability (%)	85	
Overall Deviation, S_o	0.49	
Resilient Modulus, M_R (psi)	**6,000**	
Initial Serviceability	4.2	
Terminal Serviceability	2.5	
Standard Normal Deviate, Z_R	-1.0364334	
Δ PSI	1.7	

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

**Use Goal Seek or Solver to change SN_{req} (cell B4) so cell C5 equals the design ESAL value.*

Structural Number, SN	5.37	*Layer Thickness*	
AC Layer Coefficient, a_{AC}	0.42	*AC*	8.5 in
AC Drainage Coefficient, m_{AC}	1.00	*Base Rock*	18 in
Base Layer Coefficient, a_{base}	0.1		
Base Drainage Coefficient, m_{base}	1.0		

$$SN = a_{AC} * D_{AC} + a_{base} * D_{base} * m_{base} + a_{sub} * D_{sub} * m_{sub}$$

Appendix E: Final Hydraulics and Scour Assessment Report

REGISTERED PROFESSIONAL ENGINEER
77018PE
OREGON
JANUARY 9, 2007
HANS RAYMOND HADLEY
EXPIRES: 6/30/17

Bridge Hydraulics and Scour Assessment Report
for
53rd Street Railroad Overpass Design

Prepared for:

David Evans & Associates, Inc.
530 Center Street, Suite 605
Salem, OR 97301
(503) 361-8635

Prepared by:

WEST
Consultants, Inc.

2601 25th St. SE, Suite 450
Salem, OR 97302
(503) 485-5490

March 2, 2016

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INTRODUCTION

The Benton County Department of Public Works (BCPW) is designing a new overpass along 53rd Street that passes over an existing railroad and Dunawi Creek near Corvallis, Oregon. A project location map is shown in **Figure 1** (all figures are located in **Appendix A**). As part of the project, portions of Old Reservoir Avenue and 53rd Street will be removed and Dunawi Creek will be re-aligned to flow eastward along a portion of the Old Reservoir Avenue, then southward underneath an existing railroad bridge, where 53rd Street is currently located, eastward beneath the new overpass, and southward to its connection with the existing channel. Additionally, a pedestrian bridge will be added across Dunawi Creek to the northwest of the 53rd Street railroad overpass. As part of this work, hydraulic and scour evaluations were performed to determine the hydraulic impacts of the proposed project and assist in the project designs.

All elevations in this report are referenced to the North American Vertical Datum of 1988 (NAVD 1988), unless stated otherwise.

REGULATORY

The project site is not located within a FEMA regulatory floodplain or floodway. However, there is a history of flooding at this location. The proposed project should not increase the computed 100-year flood elevation by greater than 1 foot and should not increase the flood risk to adjacent properties.

SITE INVESTIGATION

An investigation of the project area was conducted by Hans R. Hadley, P.E., WEST Consultants, Inc. on March 18, 2014.

The following items were observed during the field site investigation:

1) Lateral Channel Stability

   The channel banks in the study reach are steep but show little evidence of bank erosion and/or failures. The bank material in the vicinity of the bridge is comprised primarily of cohesive clay/silt sized material.

2) Aggradation/Degradation

   No signs of significant aggradation or degradation were observed in the channel. The channel appears to be slightly incised.

3) Manning's *n*

   Manning's *n* values vary with location. Manning’s *n* values of 0.04 were estimated for the main channel. The overbank area Manning’s n values range between 0.04 in open fields to 0.12 in dense vegetation. These values were selected based upon the investigator's judgment and experience.

4) Riprap

Small riprap was observed at the downstream end of the 53rd Street culvert to help support the roadway shoulder.

5) Bed Material

The stream bed material varies by location. Upstream of Old Reservoir Road, the bed material consists of cobble (D_{50} = 3 inches) and deposited silt-sized material. The cobble-sized material appears to have been placed as part of the previous channel realignment project. Between Old Reservoir Road and the railroad culvert, the bed material consists of fine gravel (D_{50} =0.25 inches) and is also partially vegetated with grass. Downstream of the railroad culvert, the bed material consists of fine gravel (D_{50} = 0.125 to 0.25 inches).

6) Evidence of Scour

Scour was observed at the entrance to the railroad culvert.

7) Pier Alignment

There are no existing bridge piers in the main channel.

8) Hydraulic Controls

No hydraulic controls were observed in the vicinity of the project site. However, beaver dams are known to sometimes be present downstream (personal communication with Gordon Kurtz, BCPW).

9) High Water Marks

No high water marks were observed. However, a web link to a video of high water flowing beneath the railroad trestle on January 19, 2012 was provided by Benton County. See https://www.youtube.com/watch?v=iQcX9ji3Qb0.

10) Debris

No significant debris was observed at the project site.

11) Dunes

No dune bed forms were observed.

A photographic log of site investigation observations is provided in **Appendix B**.

HYDROLOGY

Peak discharges for the 2-, 5-, 10-, 50-, and 100-year events for Dunawi Creek were obtained from the Reservoir Avenue Realignment Project Hydraulic Report (BCPW, 2011). Peak discharges for the 25- and 500-year events were determined by interpolation and extrapolation, respectively, from the Benton County values. The peak discharges for recurrence intervals ranging from 2 to

500 years are shown in Table 1.

Table 1. Peak Discharges for Dunawi Creek at Project Site

Recurrence Interval (years)	Peak Discharge (cfs)
2	24
5	39
10	64
25	78
50	82
100	114
500	170

HYDRAULICS

The U.S. Army Corps of Engineers River Analysis System standard-step backwater computer program (HEC-RAS Version 4.1) was used to compute the channel hydraulics (U.S. Army Corps of Engineers, 2010). HEC-RAS computes flow in one dimension based on input cross sectional geometry data. Cross section locations were selected to adequately model flow characteristics throughout the project area. Cross-section station-elevation information was extracted from the existing and proposed conditions elevation datasets. The existing conditions elevation dataset was developed based on a survey conducted by BCPW in February of 2015. Survey data was provided in the form of a digital terrain model (DTM). Because the survey did not cover portions of the overbank areas, supplemental elevation data was obtained from the Oregon Department of Geology and Mineral Industries (DOGAMI, 2009) to provide complete coverage of the project area for the existing conditions. Proposed conditions contours were provided by the BCPW for areas in which the ground elevations are to be altered. Due to the proposed channel realignment, the placement of cross sections was updated to reflect the proposed conditions. A total of 16 cross sections were used in the existing conditions and 20 cross sections were used in the proposed conditions. Due to the change in channel alignment and the addition of the 53rd Street railroad overpass embankment, XS 1052 is the only section upstream of the project area that uses the same geometry in both models. However, due to the lengthening of the channel under the proposed conditions, XS 1052 in the existing conditions is referred to as XS 1147 in the proposed conditions. The location of the cross sections for each condition is shown in **Figure 2** and **Figure 3**, respectively.

Channel and overbank resistance values were selected based upon the investigator's experience and judgment. Manning's n values of 0.04 were selected for the overbank areas of XS 1052, 1014, and 967 in the existing conditions model and XS 1147 and the left overbank of XS 1070 in the proposed conditions model. Manning's n values of 0.12 were selected for the remaining overbank areas in both the existing and proposed conditions models. A channel Manning's n value of 0.04 was selected for the existing and proposed conditions models. A slope of 0.005 was used in all of the models to determine a normal depth starting water surface elevation for backwater calculations. This slope was determined from field survey of the channel at the downstream end of the model.

Initial existing conditions modeling efforts revealed that flow is backed up by the railroad culvert. Once flow is sufficiently backed up to flow onto the Old Reservoir Avenue roadway, it flows eastward along Old Reservoir Avenue, then southward on 53rd Street before flowing back into Dunawi Creek downstream of the 53rd Street culvert. This flow breakout occurs during the 25-year and larger events. Therefore, a combined one-dimensional (1D) and two-dimensional (2D) HEC-RAS model was established to estimate the amount of flow that breaks out of the main channel for the existing conditions. The main channel and right overbank portion of Dunawi Creek were modeled using the 1D component of HEC-RAS. The left overbank portion of Dunawi Creek and the area near the intersection of Reservoir Avenue and 53rd Street were modeled using the 2D component of HEC-RAS. The 1D/2D model schematic is shown in **Figure 4**. The results of the combined 1D/2D modeling indicate that flow leaves the main channel of Dunawi Creek in the existing conditions model between XS 1052 and 967. During the 500-year event, a breakout flow of 5 cfs also occurs between XS 911 and 893. Flow change locations were added to the existing conditions 1D model to reflect the reduction in flow in the main channel as a result of the breakout flows. The breakout flows were added back into the model at XS 535, downstream of the 53rd Street culvert. The existing conditions flow change locations and associated flows at those locations are shown in Table 2.

Table 2. Flows at Flow Change Locations in Existing Conditions HEC-RAS Model

XS	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
1052	24	39	64	78	82	114	170
1014	24	39	64	74	76	98	119
911	24	39	64	74	76	98	114
535	24	39	64	78	82	114	170

The results of the hydraulic analyses for the existing and proposed conditions are provided in Table 3 and Table 4. For the existing conditions, the 25-, 50-, 100-, and 500-year flows all overtop the Old Reservoir Avenue roadway. Flow does not overtop the railroad bridge. The 100-year and 500-year flows overtop 53rd Street at the sag in the vertical curve. For the proposed conditions, flow does not overtop the proposed pedestrian bridge, the existing railroad bridge, nor the proposed 53rd Street railroad overpass.

The hydraulic modeling results for the existing and proposed conditions were compared to determine backwater effects. Due to the realignment of the channel, only XS 1052 (proposed conditions XS 1147) could be assessed to determine if backwater effects occurred as a result of the new bridge and channel configuration. The water surface elevations for this cross section are compared for the two conditions in Table 5. As seen in the table, the proposed conditions do not cause an increase in backwater at XS 1052 (proposed conditions XS 1147) and instead result in a decrease in the water surface elevation for all of the flows that were evaluated. Water surface elevation profile plots are presented for the existing and proposed conditions in **Figures 5 and 6**, respectively. Summary tables of HEC-RAS model output for the 1D model simulations of the existing and proposed conditions are presented in **Appendix C**.

Table 3. Existing Conditions Water Surface Elevations

Cross Section	Water Surface Elevations (feet, NAVD 88)						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
1052	266.27	267.03	268.42	268.76	268.77	269.06	269.62
1014	266.26	267.02	268.42	268.76	268.77	269.06	269.63
967	266.22	266.98	268.36	268.72	268.73	269.04	269.62
911	265.97	266.60	267.48	267.82	267.88	268.70	269.56
902	265.98	266.62	267.52	267.86	267.92	268.75	269.58
893	265.95	266.57	267.44	267.77	267.84	268.65	269.44
827	263.59	263.73	264.22	264.48	264.54	265.08	265.31
730	262.67	263.26	264.13	264.46	264.52	265.11	265.36
649	262.44	263.10	264.03	264.37	264.44	265.04	265.28
598	262.43	263.10	264.03	264.37	264.44	265.04	265.27
535	260.32	260.53	261.00	261.22	261.28	261.68	262.21
450	259.94	260.35	260.84	261.06	261.11	261.51	262.05
365	259.88	260.27	260.74	260.94	261.00	261.38	261.89
216	259.83	260.20	260.65	260.84	260.89	261.26	261.74
120	259.78	260.13	260.55	260.73	260.78	261.12	261.57
11	259.57	259.87	260.24	260.40	260.44	260.75	261.16

Table 4. Proposed Conditions Water Surface Elevations

Cross Section	Water Surface Elevations (feet, NAVD 88)						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
1147	264.39	264.61	264.89	265.02	265.06	265.32	265.71
1070	263.71	263.93	264.23	264.37	264.41	264.69	265.10
987	263.10	263.32	263.62	263.76	263.80	264.08	264.50
913	262.56	262.78	263.08	263.23	263.27	263.56	263.99
851	262.08	262.31	262.62	262.77	262.81	263.11	263.54
834	261.98	262.22	262.56	262.72	262.77	263.07	263.53
807	261.80	262.04	262.38	262.54	262.58	262.87	263.29
767	261.60	261.84	262.17	262.34	262.38	262.68	263.12
742	261.31	261.53	261.84	261.99	262.03	262.33	262.78
728	261.21	261.43	261.73	261.88	261.92	262.23	262.68
713	261.09	261.31	261.62	261.77	261.82	262.13	262.59
698	260.99	261.22	261.53	261.69	261.73	262.05	262.52
660	260.72	260.95	261.28	261.45	261.50	261.84	262.32
619	260.48	260.71	261.07	261.25	261.30	261.66	262.15
536	259.95	260.32	260.77	260.98	261.03	261.41	261.91
478	259.86	260.24	260.70	260.90	260.95	261.33	261.82
390	259.83	260.20	260.65	260.85	260.90	261.27	261.75
228	259.82	260.19	260.63	260.82	260.88	261.23	261.70
120	259.80	260.14	260.56	260.74	260.79	261.12	261.54
11	259.59	259.89	260.26	260.42	260.46	260.76	261.15

Table 5. Backwater Comparison at XS 1052 (Proposed Conditions XS 1147)

Condition	Water Surface Elevations and Changes (feet, NAVD 88)						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
Existing	266.27	267.03	268.42	268.76	268.77	269.06	269.62
Proposed	264.39	264.61	264.89	265.02	265.06	265.32	265.71
Difference	-1.88	-2.42	-3.53	-3.74	-3.71	-3.74	-3.91

SCOUR CALCULATIONS

Contraction Scour

Contraction scour was evaluated for the 500-year discharge for the proposed conditions. Flow is contracting at the railroad bridge because of the presence of a bridge pier in the main channel. Flow is contracting at the pedestrian bridge because the channel bank slopes are steeper here than at the approach section. Because flow is contained entirely within the realigned portion of the channel near the 53rd Street railroad overpass bridge and because no changes in the channel geometry occur between the approach section and this bridge, no contraction scour was assessed at this location.

To determine if live-bed or clear-water contraction scour would occur for the 500-year discharge at either the railroad bridge or the pedestrian bridge, Laursen's equation presented in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 18 (HEC-18; FHWA, 2012) was used:

$$V_c = 11.17(y_1)^{\frac{1}{6}}(D_{50})^{\frac{1}{3}}$$

where y_1 is the average depth of flow upstream of the bridge; D_{50} is the median diameter of the bed material, assumed to be medium sand in the absence of design information, 0.00164 feet; and V_c is the critical velocity for incipient motion of bed material in the approach section. Because the approach section velocity is greater than the critical velocity at both bridge locations, Laursen's live-bed scour equation (FHWA, 2012) was used to compute the contraction scour:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}}\left(\frac{W_1}{W_2}\right)^{k_1}$$

where y_2 is the depth in the contracted section; Q_1 and Q_2 are the flows in the upstream section that is transporting sediment and in the contracted section, 170 cfs; W_1 is the top width of the upstream main channel; W_2 is the top width in the contracted section; and k_1 is an exponent based on the ratio of the bed shear velocity at the approach section to the settling velocity of the bed material. The contraction scour is calculated using the following equation:

$$y_s = y_2 - y_0$$

where y_s is the contraction scour depth and y_0 is the existing depth at the contracted section. A summary of the variables used in the calculation of contraction scour at the pedestrian and railroad bridges is presented in Table 6.

Table 6. Summary of Contraction Scour Calculations for Project Area Bridges

Bridge	Approach Section	y_1 (ft)	V_1 (ft/s)	V_c (ft/s)	y_2 (ft)	W_1 (ft)	W_2 (ft)	k_1	y_0 (ft)	y_s (ft)
Pedestrian	XS 913	2.1	4.3	1.5	2.7	32.4	22.7	0.69	2.1	0.6
Railroad	XS 807	2.2	4.2	1.5	2.4	23.3	20.5	0.69	2.2	0.2

Aggradation/Degradation

No evidence of channel degradation was observed during the field reconnaissance. Also, the proposed channel will not contain any significant discontinuities in the longitudinal profile that would be expected induce headcutting. Therefore, long-term degradation is assumed to be 0.0 ft for all of the bridges. However, some long-term adjustment to the channel profile should be expected. Therefore, any riprap that is placed should incorporate the ODOT standard toe trench to help prevent potential future undermining.

Pier Scour

Only the existing railroad bridge has piers; therefore, pier scour was only evaluated for this bridge. The only pier impacted by flow during the 500-year event is the central pier of the railroad bridge. Because the proposed conditions configuration of this pier is unknown at this time, BCPW requested that a pier width of 3 feet be used in the hydraulic modeling and pier scour calculations. The pier scour depth (y_s) was estimated using the HEC-18 pier scour equation (FHWA, 2012):

$$y_s = 2.0 K_1 K_2 K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} y_1$$

where y_s is the computed pier scour depth; K_1 is a correction factor for pier nose shape, 1.1 for square-nosed piers; K_2 is a correction factor for angle of attack of flow, 1.0; K_3 is a correction factor for the bed condition, 1.1 for plane bed; y_1 is the maximum flow depth immediately upstream of the bridge, 2.2 feet; a is the pier width, 3 feet; Fr_1 is the Froude number immediately upstream of the bridge, 0.50 ($Fr_1 = V_1/(gy_1)^{0.5}$). The above calculation indicates a pier scour depth of 4.8 feet for the 500-year flood for the railroad bridge pier.

Total Scour

The total scour at each bridge is equal to the summation of the contraction scour, long-term degradation, and pier scour. Results of the scour evaluation are summarized in Table 7.

Table 7. Summary of Bridge Scour

Bridge	Contraction Scour	Long-term Degradation	Pier Scour	Total Scour	Channel Bed Elevation	Total Scour Elevation
Pedestrian	0.6 ft	0.0 ft	N/A	0.6 ft	261.4 ft	260.8 ft
Railroad	0.2 ft	0.0 ft	4.8 ft	5.0 ft	260.8 ft	255.8 ft
53rd Street Railroad Overpass	N/A	0.0 ft	N/A	0.0 ft	259.7 ft	259.7 ft

ABUTMENT RIPRAP

Abutment erosion protection was designed for each bridge in the project area. A riprap evaluation assuming Oregon Department of Transportation (ODOT, 2014) and HEC-11 (FHWA, 1989) criteria was conducted for the 100-year flood. The results of the riprap design were checked against the 500-year flood. Riprap size was computed using the following equation:

$$D_{50} = 0.001\, C\, V_a^3 / (d_{avg}^{0.5} K_1^{1.5})$$

where D_{50} is the median riprap particle size; V_a is the average channel velocity at the upstream bridge face; d_{avg} is the average flow depth at the upstream face of the bridge; C is a correction factor

$$C = (SF/1.2)^{1.5} = 1$$

where SF = 1.2; and

$$K_1 = (1-(\sin^2 \Theta / \sin^2 \Phi))^{0.5}$$

where Θ is the bank angle with the horizontal; and Φ is the riprap angle of repose, 41 degrees. The side slopes are 2.5H on 1V for the exiting railroad bridge and proposed overpass. The side slopes are 1.75H on 1V for the proposed pedestrian bridge.

The input values for the riprap calculations along with the results of the calculations are shown in Table 8. ODOT Class 50 English riprap is recommended for protection of all bridge abutments. A check against the 500-year flood indicates that this riprap size should be stable during a 500-year event as well. The longitudinal extents of the riprap should extend sufficiently upstream and downstream to prevent flanking of the riprap. Additionally, any riprap that is placed should incorporate the ODOT standard toe trench (**Figure 7**) to help prevent potential future undermining.

Table 8. Summary of Riprap Sizing for 100-year Flood for Project Bridges

Bridge	Approach Section	C	V_a	D_{avg}	Θ	Φ	K_1	D_{50}	Riprap Class
Pedestrian	XS 1070	1	3.8 ft/s	1.7 ft	29.7°	41°	0.65	0.08 ft	English 50
Railroad	XS 807	1	3.7 ft/s	1.8 ft	21.8°	41°	0.82	0.05 ft	English 50
53rd Street Railroad Overpass	XS 660	1	3.4 ft/s	1.8 ft	21.8°	41°	0.82	0.04 ft	English 50

SUMMARY

A hydraulic and scour evaluation for the construction of a new 53rd Street overpass bridge, a new pedestrian bridge, and the existing railroad bridge over Dunawi Creek was conducted. Scour calculations estimated a total scour depth of 5.2 feet for the existing railroad bridge and 0.6 feet for the proposed pedestrian bridge. The proposed 53rd Street bridge is not expected to induce any scour. However, some long-term adjustment to the longitudinal profile of the channel should be expected. A summary of the scour calculations is summarized in Table 7. Using the ODOT and HEC-11 criteria for riprap revetment, a D_{50} of 0.08 feet, 0.05 feet, and 0.04 feet was calculated for the proposed pedestrian bridge, existing railroad bridge, and proposed 53rd Street bridge abutments. This corresponds to ODOT Class 50 English riprap. The longitudinal extents of the riprap should extend sufficiently upstream and downstream to prevent flanking of the riprap. All riprap revetments should include the standard ODOT toe trench to help prevent potential future undermining that may occur as a result of long-term adjustment to the longitudinal profile.

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Oregon Department of Transportation (ODOT) Highway Division, Hydraulics Unit, Hydraulics Manual, ODOT, Salem, Oregon, 2014.

U.S. Army Corps of Engineers (USACE), HEC-RAS River Analysis System User's Manual, Version 4.1.0, January 2010.

APPENDIX A

FIGURES

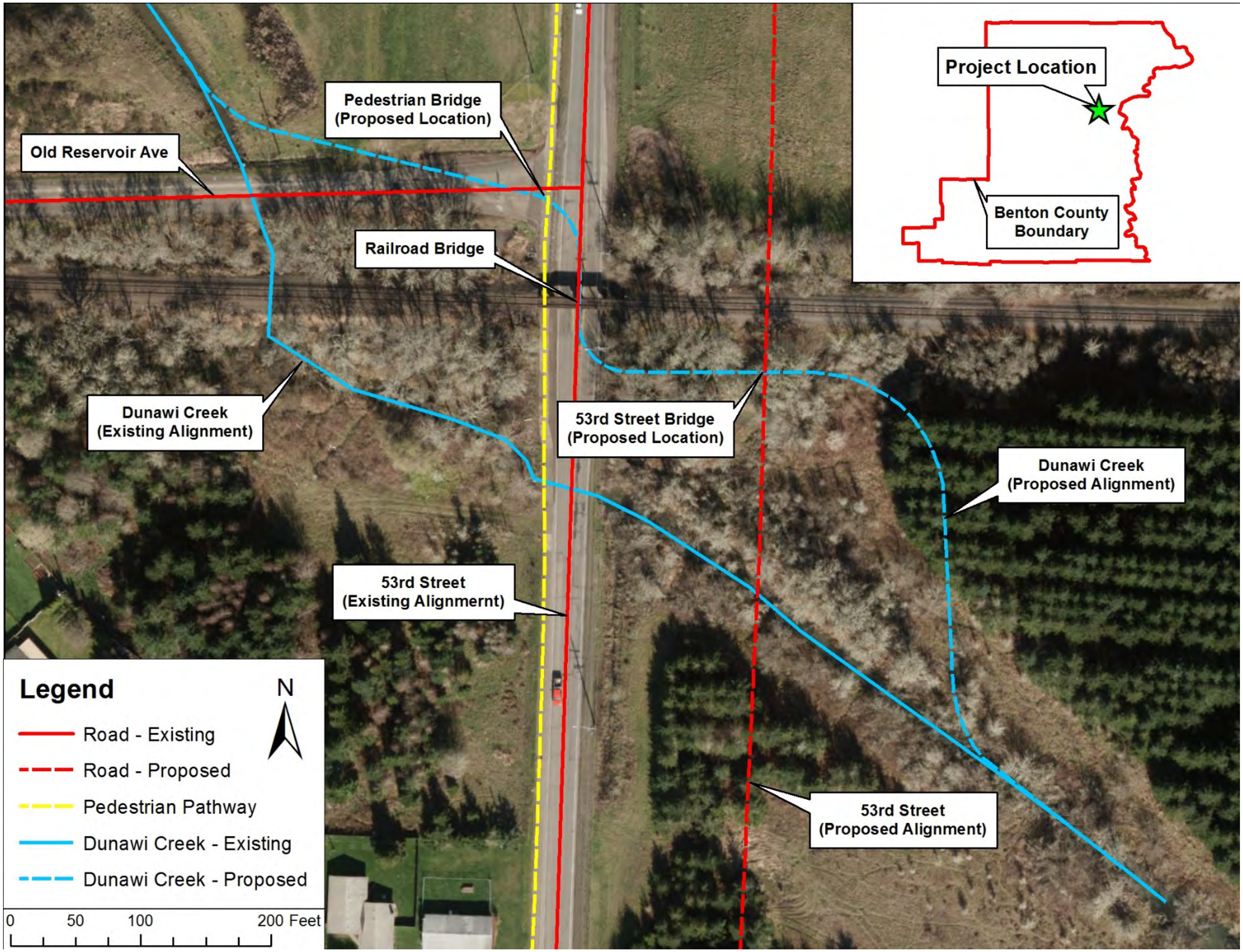


Figure 1. Project Location Map

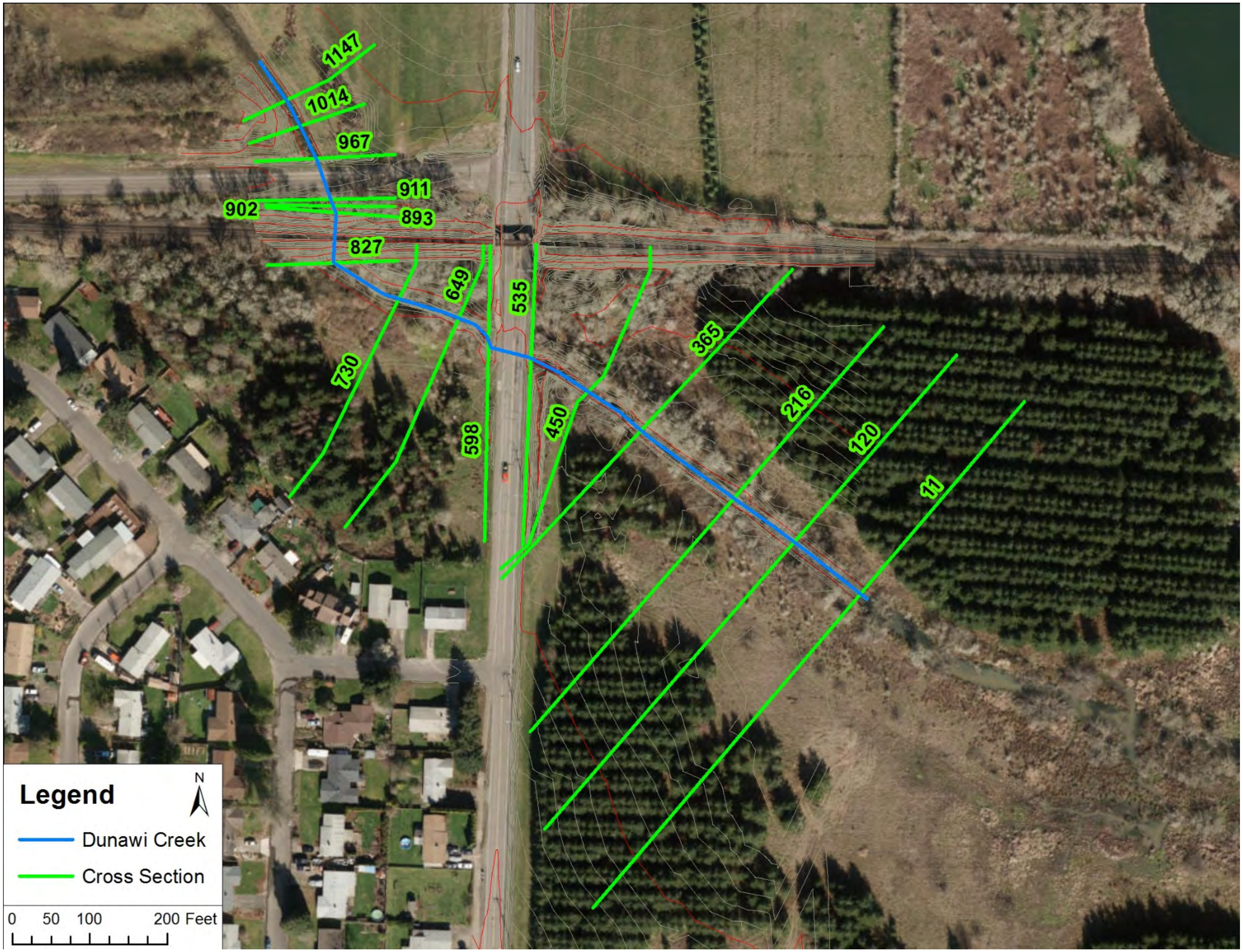


Figure 2. Existing Conditions Cross Section Layout Map

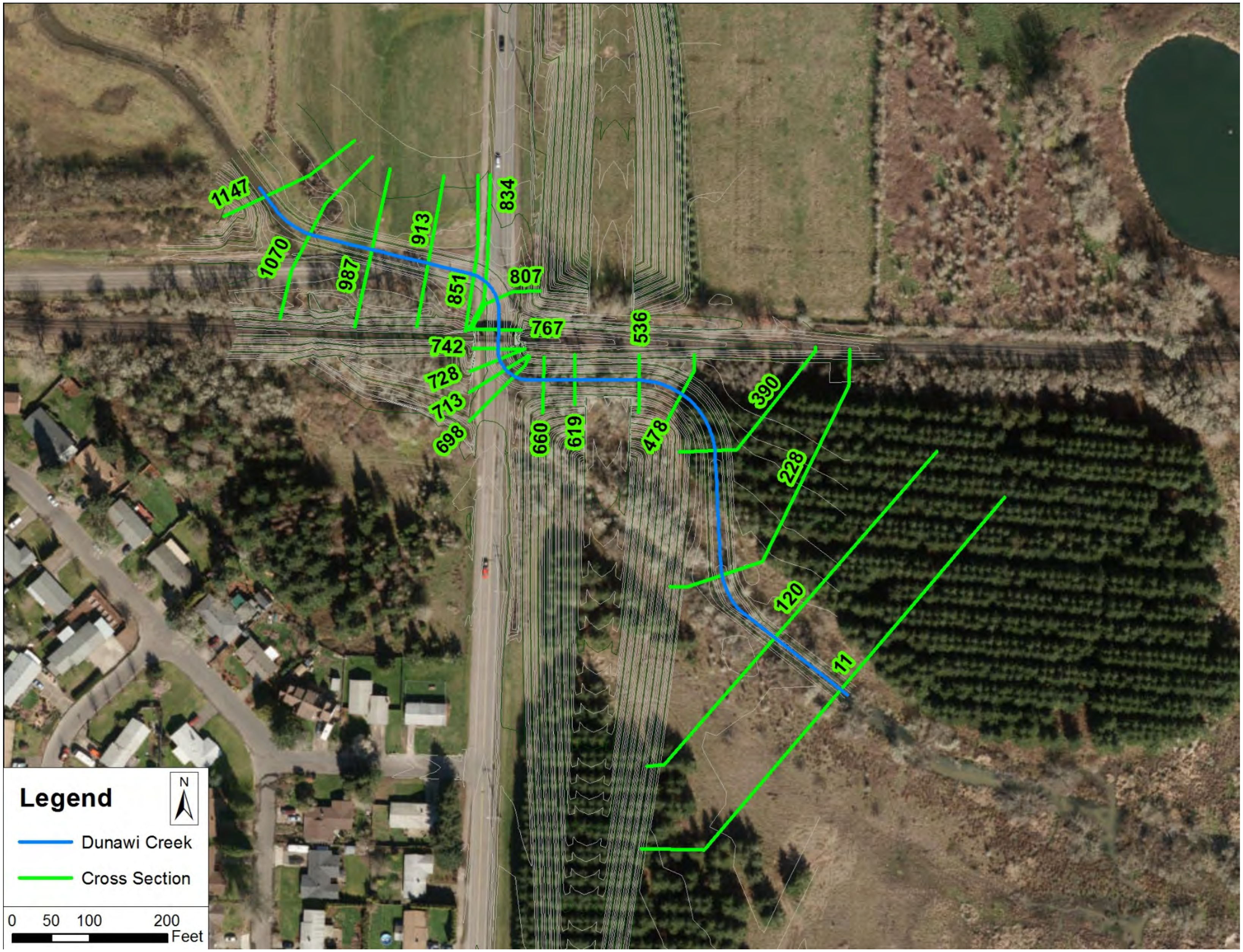


Figure 3. Proposed Conditions Cross Section Layout Map

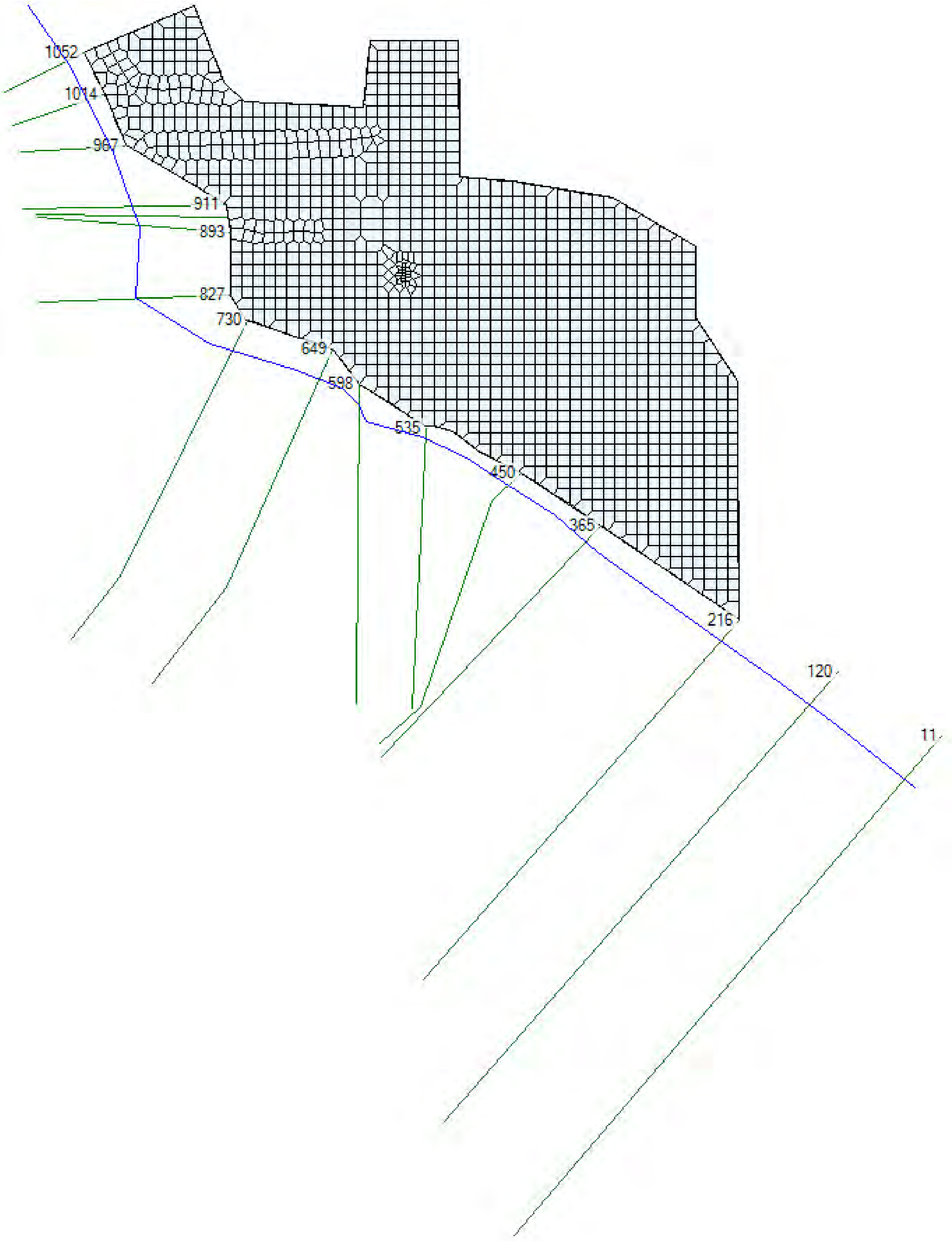


Figure 4. 1D/2D HEC-RAS Model Geometry Schematic

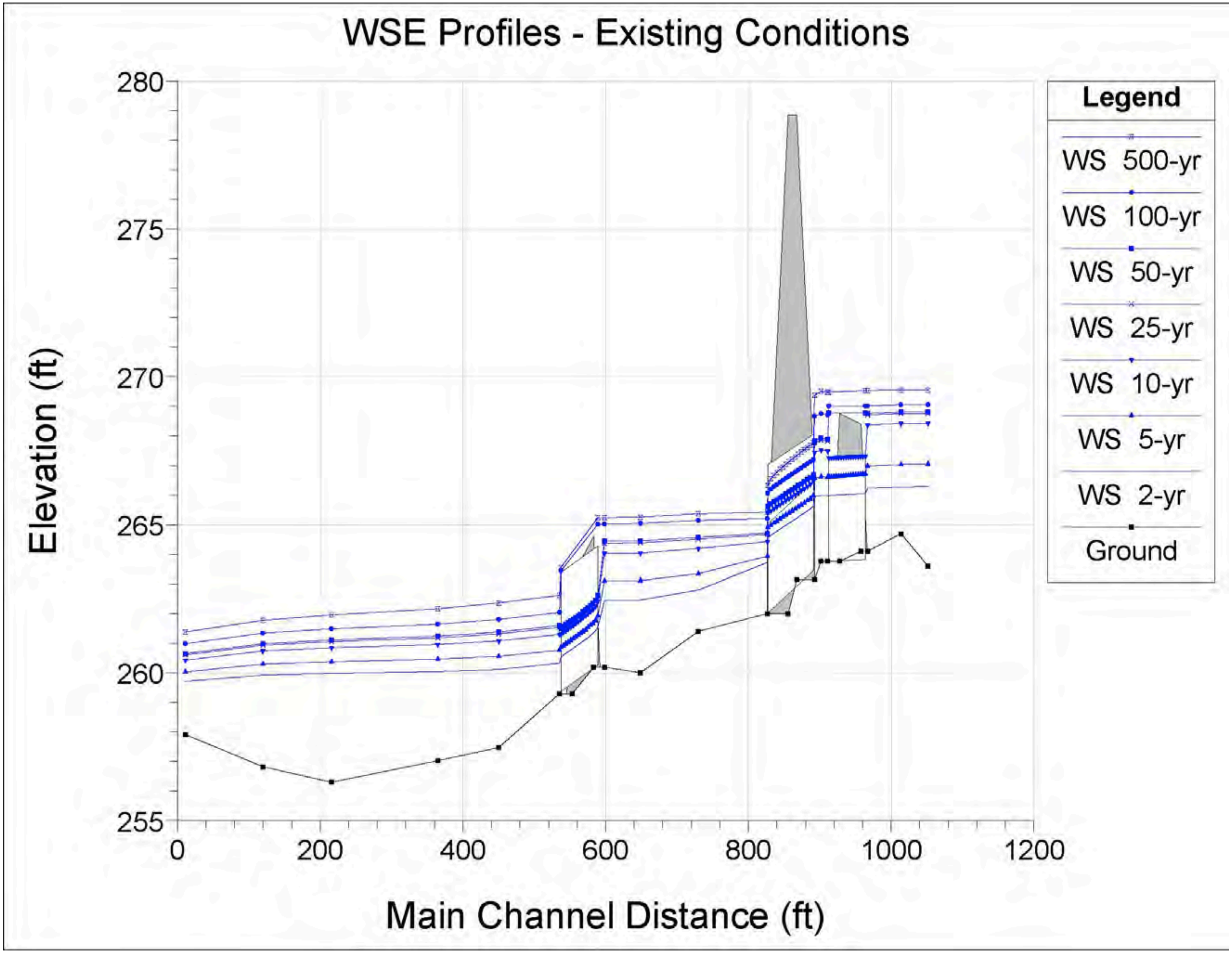


Figure 5. Existing Conditions Water Surface Profiles

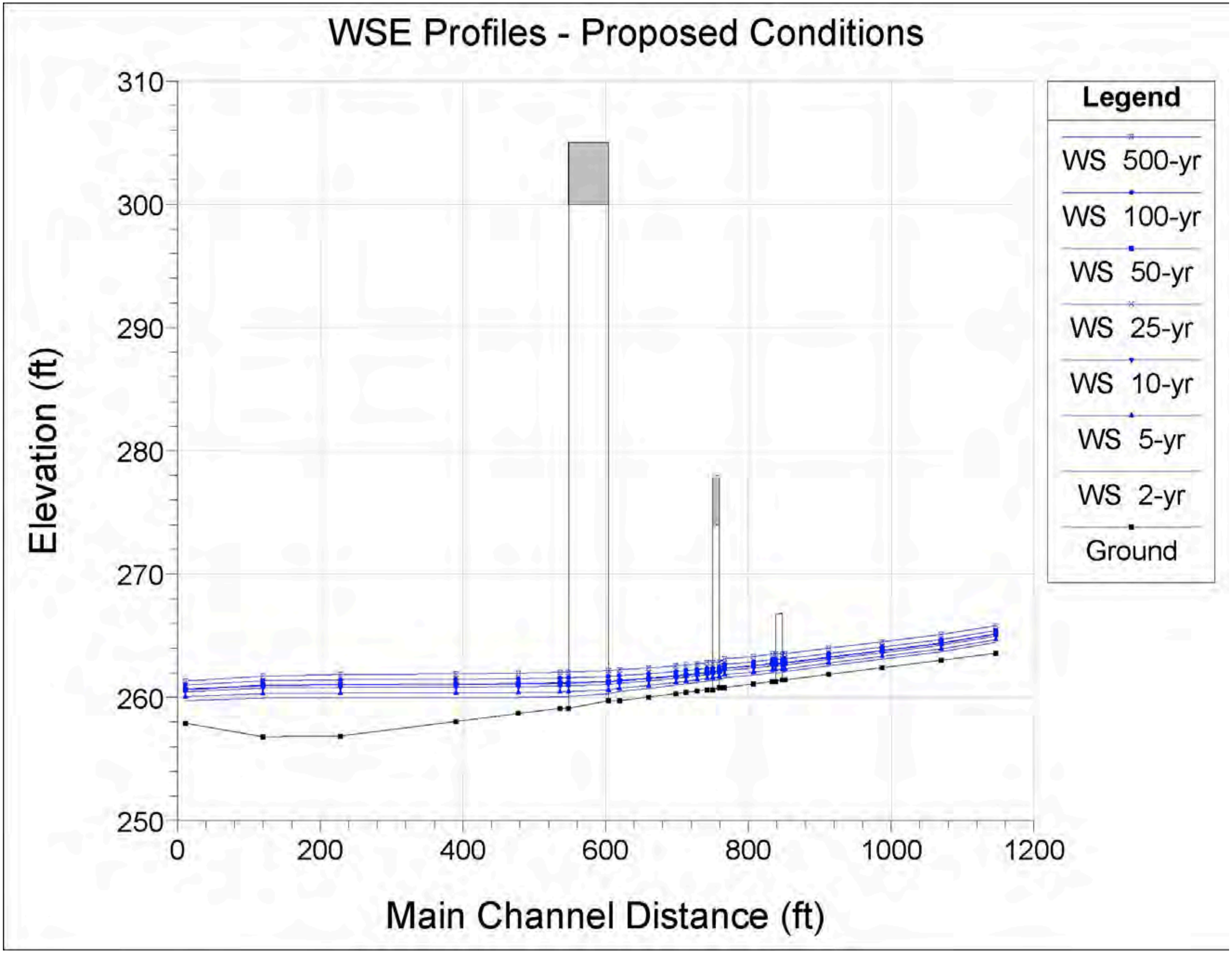
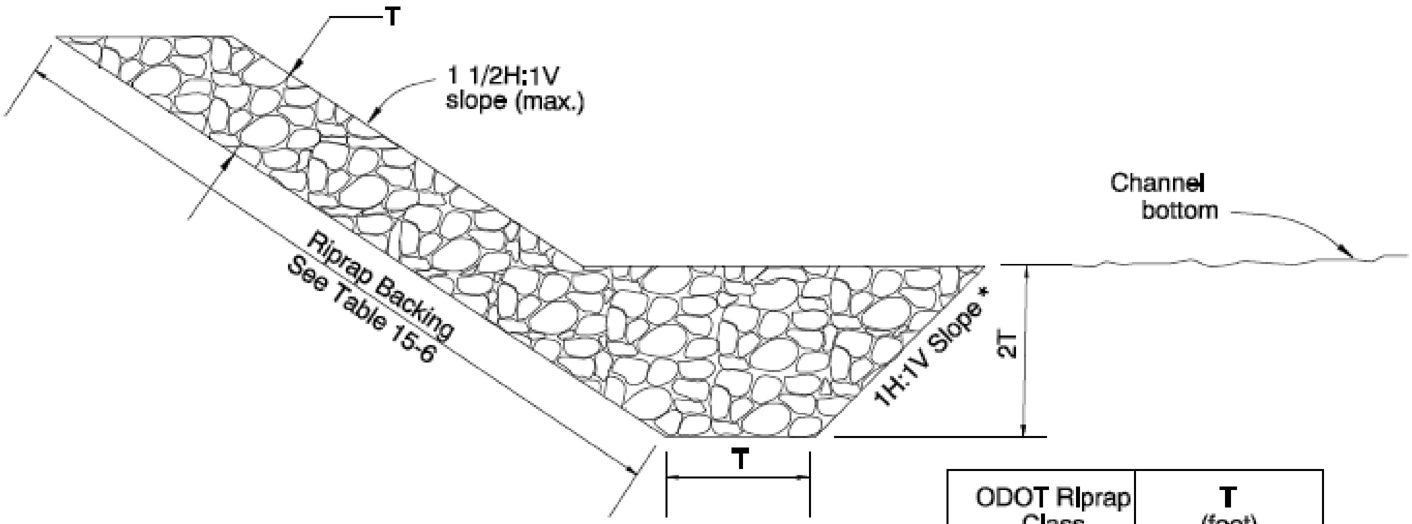


Figure 6. Proposed Conditions Water Surface Profiles



* A flatter slope may be used in materials that will not have a stable excavated face at a 1:1 slope, such as wet sands and gravels.

ODOT Riprap Class	**T** (feet)
50	1.0
100	1.5
200	2.0
700	3.0
2000	4.0

Figure 7 . ODOT Standard Riprap Section

APPENDIX B

PHOTOGRAPHIC LOG

53rd Street Railroad Overpass - Photo Log
Field Reconnaissance 3/18/14

Photo 1: Looking upstream from Old Reservoir Road

Photo 2: Inlet of Old Reservoir Road culvert

Photo 3: Inlet to railroad culvert

Photo 4: Outlet of Old Reservoir Road culvert

Photo 5: Inlet of railroad overflow culvert

Photo 6: Looking east along Old Reservoir Road

Photo 7: Railroad bridge over 53rd Street

Photo 8: Looking u/s from 53rd Street

Photo 9: Inlet of 53rd Street culvert

Photo 10: Looking d/s from 53rd Street

Photo 11: Outlet of 53rd Street culvert

Photo 12: Looking north along 53rd Street

Photo 13: Looking west along Old Reservoir Road

Photo 14: Looking South along pedestrian path and 53rd Street

APPENDIX C

HEC-RAS OUTPUT

Existing Conditions

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
53rd Street	1052	2-yr	24	263.59	266.27	264.24	266.27	0.000085	0.52	46.77	28.05	0.07
53rd Street	1052	5-yr	39	263.59	267.03	264.43	267.04	0.000069	0.58	71.75	39.16	0.06
53rd Street	1052	10-yr	64	263.59	268.42	264.69	268.42	0.000033	0.53	154.91	79.22	0.05
53rd Street	1052	25-yr	78	263.59	268.76	264.81	268.76	0.000033	0.56	182.72	85.02	0.05
53rd Street	1052	50-yr	82	263.59	268.77	264.84	268.77	0.000036	0.58	184.02	85.31	0.05
53rd Street	1052	100-yr	114	263.59	269.06	265.07	269.06	0.000051	0.73	209.51	93.51	0.06
53rd Street	1052	500-yr	170	263.59	269.62	265.41	269.63	0.000065	0.89	267.51	111.3	0.07
53rd Street	1014	2-yr	24	264.7	266.26	265.17	266.27	0.000342	0.73	32.82	36.63	0.12
53rd Street	1014	5-yr	39	264.7	267.02	265.32	267.03	0.00017	0.67	59.01	54.34	0.09
53rd Street	1014	10-yr	64	264.7	268.42	265.53	268.42	0.000023	0.37	216.29	116.38	0.04
53rd Street	1014	25-yr	74	264.7	268.76	265.61	268.76	0.000019	0.36	255.95	117.43	0.03
53rd Street	1014	50-yr	76	264.7	268.77	265.62	268.77	0.000019	0.37	257.75	117.47	0.04
53rd Street	1014	100-yr	98	264.7	269.06	265.76	269.06	0.000022	0.41	291.76	118.36	0.04
53rd Street	1014	500-yr	119	264.7	269.63	265.88	269.63	0.000017	0.4	359.41	120.11	0.03
53rd Street	967	2-yr	24	264.1	266.22	264.79	266.25	0.000445	1.25	19.2	109.62	0.16
53rd Street	967	5-yr	39	264.1	266.98	265.01	267.01	0.000406	1.48	26.42	152.46	0.16
53rd Street	967	10-yr	64	264.1	268.36	265.32	268.4	0.000283	1.61	39.64	168.72	0.14
53rd Street	967	25-yr	74	264.1	268.72	265.44	268.75	0.000348	1.45	73.4	179.44	0.15
53rd Street	967	50-yr	76	264.1	268.73	265.46	268.76	0.000355	1.47	74.95	179.97	0.15
53rd Street	967	100-yr	98	264.1	269.04	265.69	269.05	0.000525	1.17	116.71	181.67	0.16
53rd Street	967	500-yr	119	264.1	269.62	265.89	269.63	0.000134	0.72	222.28	181.67	0.09
53rd Street	943		Culvert									
53rd Street	911	2-yr	24	263.76	265.97		266.04	0.001969	2.13	11.28	18.82	0.3
53rd Street	911	5-yr	39	263.76	266.6		266.7	0.001706	2.47	15.76	34.77	0.29
53rd Street	911	10-yr	64	263.76	267.48		267.62	0.001506	2.91	22.03	77.04	0.29
53rd Street	911	25-yr	74	263.76	267.82		267.96	0.001431	3.03	24.4	100.84	0.29
53rd Street	911	50-yr	76	263.76	267.88		268.03	0.001418	3.06	24.86	104.81	0.29
53rd Street	911	100-yr	98	263.76	268.7		268.86	0.001172	3.2	30.66	124.14	0.27
53rd Street	911	500-yr	114	263.76	269.56		269.62	0.000703	2.29	142.7	180.28	0.2
53rd Street	902	2-yr	24	263.77	265.98	264.74	266.01	0.000855	1.32	19.1	18.11	0.18
53rd Street	902	5-yr	39	263.77	266.62	264.96	266.66	0.000742	1.52	29.24	41.51	0.17
53rd Street	902	10-yr	64	263.77	267.52	265.28	267.56	0.00064	1.75	43.49	108.46	0.17
53rd Street	902	25-yr	74	263.77	267.86	265.39	267.91	0.000604	1.82	48.87	133.56	0.17
53rd Street	902	50-yr	76	263.77	267.92	265.42	267.97	0.000598	1.83	49.91	136.13	0.17
53rd Street	902	100-yr	98	263.77	268.75	265.65	268.8	0.000489	1.89	62.99	144.94	0.16
53rd Street	902	500-yr	114	263.77	269.58	265.82	269.59	0.000122	1.06	271.15	153.79	0.08
53rd Street	893	2-yr	24	263.15	265.95	264.4	266	0.000694	1.75	17.05	30.68	0.2
53rd Street	893	5-yr	39	263.15	266.57	264.72	266.64	0.000788	2.17	23.7	50.64	0.22
53rd Street	893	10-yr	64	263.15	267.44	265.21	267.54	0.000847	2.67	32.99	90.58	0.24

Existing Conditions

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
53rd Street	893	25-yr	74	263.15	267.77	265.41	267.88	0.000844	2.81	36.52	127.44	0.24
53rd Street	893	50-yr	76	263.15	267.84	265.45	267.95	0.000844	2.84	37.2	132.83	0.24
53rd Street	893	100-yr	98	263.15	268.65	265.78	268.77	0.000754	3.01	45.89	171.44	0.24
53rd Street	893	500-yr	114	263.15	269.44	265.99	269.56	0.000612	2.99	54.34	171.44	0.22
53rd Street	862		Culvert									
53rd Street	827	2-yr	24	262	263.59	263.3	263.79	0.0101	3.57	6.72	10	0.63
53rd Street	827	5-yr	39	262	263.73	263.61	264.13	0.017336	5.1	7.65	24.04	0.85
53rd Street	827	10-yr	64	262	264.22	264.05	264.71	0.013426	5.7	12.93	45.16	0.79
53rd Street	827	25-yr	74	262	264.48	264.2	264.95	0.010418	5.56	16.18	51.09	0.71
53rd Street	827	50-yr	76	262	264.54	264.22	265	0.009855	5.52	16.91	52.38	0.7
53rd Street	827	100-yr	98	262	265.08	264.49	265.5	0.006753	5.39	24.12	82.93	0.6
53rd Street	827	500-yr	114	262	265.31	264.67	265.77	0.0067	5.68	27.28	89.09	0.61
53rd Street	730	2-yr	24	261.4	262.67	262.43	262.79	0.00988	2.74	8.76	13.28	0.59
53rd Street	730	5-yr	39	261.4	263.26	262.65	263.34	0.003443	2.23	17.52	16.25	0.38
53rd Street	730	10-yr	64	261.4	264.13	262.93	264.19	0.001386	1.93	33.88	23.56	0.26
53rd Street	730	25-yr	74	261.4	264.46	263.02	264.51	0.001031	1.87	42.36	28.24	0.23
53rd Street	730	50-yr	76	261.4	264.52	263.03	264.58	0.000975	1.85	44.27	29.19	0.22
53rd Street	730	100-yr	98	261.4	265.11	263.22	265.17	0.000688	1.83	64.98	41	0.2
53rd Street	730	500-yr	114	261.4	265.36	263.34	265.41	0.000683	1.93	75.48	45.27	0.2
53rd Street	649	2-yr	24	260	262.44	261.36	262.49	0.001744	1.78	13.45	9.56	0.27
53rd Street	649	5-yr	39	260	263.1	261.67	263.15	0.001489	1.91	20.38	11.45	0.25
53rd Street	649	10-yr	64	260	264.03	262.09	264.09	0.001077	1.98	33.47	18.39	0.22
53rd Street	649	25-yr	74	260	264.37	262.23	264.43	0.000912	1.99	40.14	20.52	0.21
53rd Street	649	50-yr	76	260	264.44	262.25	264.5	0.000883	2	41.58	21.03	0.21
53rd Street	649	100-yr	98	260	265.04	262.53	265.11	0.000746	2.09	56.81	50.97	0.2
53rd Street	649	500-yr	114	260	265.28	262.7	265.35	0.000791	2.24	64.83	81.73	0.21
53rd Street	598	2-yr	24	260.18	262.43	260.91	262.45	0.000274	0.89	26.95	26.88	0.12
53rd Street	598	5-yr	39	260.18	263.1	261.11	263.11	0.000273	1.05	37.03	30.14	0.12
53rd Street	598	10-yr	64	260.18	264.03	261.38	264.05	0.000256	1.23	52.12	33.71	0.12
53rd Street	598	25-yr	74	260.18	264.37	261.47	264.4	0.000243	1.28	57.77	35.01	0.12
53rd Street	598	50-yr	76	260.18	264.44	261.49	264.47	0.00024	1.29	58.91	35.27	0.12
53rd Street	598	100-yr	98	260.18	265.04	261.67	265.06	0.000476	1.36	72.65	52.91	0.16
53rd Street	598	500-yr	114	260.18	265.27	261.8	265.31	0.00053	1.43	81.75	73.4	0.17
53rd Street	568		Culvert									
53rd Street	535	2-yr	24	259.29	260.32	260.32	260.59	0.030931	4.19	5.73	10.85	1.02
53rd Street	535	5-yr	39	259.29	260.53	260.53	260.9	0.027664	4.89	7.97	12.52	1.01
53rd Street	535	10-yr	64	259.29	261		261.37	0.01401	4.86	13.16	16.46	0.78
53rd Street	535	25-yr	78	259.29	261.22		261.61	0.011859	5.01	15.58	17.03	0.74

Existing Conditions

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
53rd Street	535	50-yr	82	259.29	261.28		261.67	0.011471	5.06	16.21	17.19	0.73
53rd Street	535	100-yr	114	259.29	261.68		262.15	0.009953	5.53	20.61	18.29	0.71
53rd Street	535	500-yr	170	259.29	262.21		262.85	0.009631	6.43	26.46	19.92	0.73
53rd Street	450	2-yr	24	257.47	259.94	258.76	259.97	0.00106	1.41	17.02	12.52	0.21
53rd Street	450	5-yr	39	257.47	260.35	259.06	260.4	0.0013	1.74	22.46	13.98	0.24
53rd Street	450	10-yr	64	257.47	260.84	259.42	260.91	0.001607	2.16	29.69	15.54	0.27
53rd Street	450	25-yr	78	257.47	261.06	259.59	261.14	0.001763	2.35	33.13	16.24	0.29
53rd Street	450	50-yr	82	257.47	261.11	259.63	261.2	0.00181	2.41	34.06	16.46	0.29
53rd Street	450	100-yr	114	257.47	261.51	259.96	261.63	0.002141	2.78	40.95	18.01	0.33
53rd Street	450	500-yr	170	257.47	262.05	260.42	262.22	0.002694	3.31	51.37	20.79	0.37
53rd Street	365	2-yr	24	257.02	259.88	258.33	259.9	0.000595	1.13	21.15	13.85	0.16
53rd Street	365	5-yr	39	257.02	260.27	258.64	260.3	0.000847	1.45	26.92	15.97	0.2
53rd Street	365	10-yr	64	257.02	260.74	259.02	260.79	0.001167	1.83	35.04	18.72	0.24
53rd Street	365	25-yr	78	257.02	260.94	259.2	261.01	0.00131	2	39.03	19.9	0.25
53rd Street	365	50-yr	82	257.02	261	259.25	261.06	0.001347	2.04	40.11	20.19	0.26
53rd Street	365	100-yr	114	257.02	261.38	259.58	261.47	0.001596	2.37	48.19	22.13	0.28
53rd Street	365	500-yr	170	257.02	261.89	260.06	262.01	0.00196	2.83	60.05	24.55	0.32
53rd Street	216	2-yr	24	256.3	259.83	257.36	259.84	0.000276	0.75	31.84	21.62	0.11
53rd Street	216	5-yr	39	256.3	260.2	257.68	260.22	0.000384	0.97	40.39	24.38	0.13
53rd Street	216	10-yr	64	256.3	260.65	258.1	260.67	0.000517	1.23	51.84	27.12	0.16
53rd Street	216	25-yr	78	256.3	260.84	258.29	260.87	0.000583	1.36	57.27	28.33	0.17
53rd Street	216	50-yr	82	256.3	260.89	258.34	260.92	0.000601	1.4	58.73	28.65	0.17
53rd Street	216	100-yr	114	256.3	261.26	258.73	261.3	0.000734	1.64	69.53	31.02	0.19
53rd Street	216	500-yr	170	256.3	261.74	259.44	261.8	0.000942	2	85.18	34.23	0.22
53rd Street	120	2-yr	24	256.82	259.78	258.25	259.8	0.000655	1.09	22.02	16.74	0.17
53rd Street	120	5-yr	39	256.82	260.13	258.56	260.16	0.000913	1.37	28.39	19.68	0.2
53rd Street	120	10-yr	64	256.82	260.55	258.94	260.6	0.001211	1.71	37.34	23.07	0.24
53rd Street	120	25-yr	78	256.82	260.73	259.12	260.79	0.001341	1.87	41.69	24.41	0.25
53rd Street	120	50-yr	82	256.82	260.78	259.17	260.84	0.001376	1.91	42.87	24.75	0.26
53rd Street	120	100-yr	114	256.82	261.12	259.53	261.19	0.001597	2.21	51.91	33.74	0.28
53rd Street	120	500-yr	170	256.82	261.57	259.98	261.67	0.001798	2.63	71.76	50.02	0.31
53rd Street	11	2-yr	24	257.91	259.57	259.14	259.64	0.005003	2.13	11.28	15.06	0.43
53rd Street	11	5-yr	39	257.91	259.87	259.36	259.96	0.005003	2.4	16.25	18.13	0.45
53rd Street	11	10-yr	64	257.91	260.24	259.65	260.35	0.005002	2.71	23.61	21.98	0.46
53rd Street	11	25-yr	78	257.91	260.4	259.78	260.53	0.005004	2.86	27.31	23.52	0.47
53rd Street	11	50-yr	82	257.91	260.44	259.82	260.57	0.005003	2.9	28.32	23.89	0.47
53rd Street	11	100-yr	114	257.91	260.75	260.06	260.9	0.005	3.17	35.99	26.52	0.48
53rd Street	11	500-yr	170	257.91	261.16	260.42	261.36	0.005001	3.57	49.53	79.93	0.49

Proposed Conditions

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
53rd Street	1147	2-yr	24	263.59	264.39	264.24	264.52	0.012953	2.91	8.26	14.33	0.67
53rd Street	1147	5-yr	39	263.59	264.61	264.43	264.78	0.013055	3.38	11.52	15.96	0.7
53rd Street	1147	10-yr	64	263.59	264.89	264.69	265.13	0.013008	3.91	16.35	18.12	0.73
53rd Street	1147	25-yr	78	263.59	265.02	264.81	265.29	0.012933	4.14	18.84	19.09	0.73
53rd Street	1147	50-yr	82	263.59	265.06	264.84	265.33	0.012863	4.19	19.56	19.36	0.74
53rd Street	1147	100-yr	114	263.59	265.32	265.08	265.65	0.012319	4.59	24.84	20.71	0.74
53rd Street	1147	500-yr	170	263.59	265.71	265.41	266.12	0.011753	5.14	33.07	22.3	0.74
53rd Street	1070	2-yr	24	263.01	263.71	263.46	263.79	0.007167	2.25	10.66	17.36	0.51
53rd Street	1070	5-yr	39	263.01	263.93	263.63	264.04	0.007247	2.67	14.6	18.47	0.53
53rd Street	1070	10-yr	64	263.01	264.23	263.85	264.38	0.007164	3.14	20.39	23.13	0.55
53rd Street	1070	25-yr	78	263.01	264.37	263.96	264.54	0.00718	3.35	23.29	24.77	0.56
53rd Street	1070	50-yr	82	263.01	264.41	263.99	264.59	0.007184	3.4	24.09	25.21	0.56
53rd Street	1070	100-yr	114	263.01	264.69	264.21	264.91	0.007209	3.78	30.14	28.29	0.57
53rd Street	1070	500-yr	170	263.01	265.1	264.55	265.38	0.007217	4.28	39.74	32.4	0.59
53rd Street	987	2-yr	24	262.41	263.1	262.87	263.18	0.007462	2.28	10.52	17.29	0.52
53rd Street	987	5-yr	39	262.41	263.32	263.02	263.43	0.007388	2.69	14.5	18.41	0.53
53rd Street	987	10-yr	64	262.41	263.62	263.25	263.78	0.007337	3.17	20.21	19.9	0.55
53rd Street	987	25-yr	78	262.41	263.76	263.36	263.94	0.007332	3.38	23.1	20.61	0.56
53rd Street	987	50-yr	82	262.41	263.8	263.39	263.98	0.007329	3.43	23.9	20.81	0.56
53rd Street	987	100-yr	114	262.41	264.08	263.61	264.31	0.007296	3.8	29.98	22.22	0.58
53rd Street	987	500-yr	170	262.41	264.5	263.94	264.79	0.00721	4.28	39.7	24.31	0.59
53rd Street	913	2-yr	24	261.86	262.56	262.32	262.64	0.007008	2.24	10.74	17.36	0.5
53rd Street	913	5-yr	39	261.86	262.78	262.48	262.89	0.007104	2.66	14.69	18.47	0.52
53rd Street	913	10-yr	64	261.86	263.08	262.7	263.23	0.007094	3.13	20.44	19.97	0.55
53rd Street	913	25-yr	78	261.86	263.23	262.81	263.4	0.007042	3.33	23.42	20.7	0.55
53rd Street	913	50-yr	82	261.86	263.27	262.84	263.45	0.007026	3.38	24.25	20.9	0.55
53rd Street	913	100-yr	114	261.86	263.56	263.06	263.78	0.006901	3.73	30.57	22.36	0.56
53rd Street	913	500-yr	170	261.86	263.99	263.39	264.26	0.00671	4.17	40.72	24.53	0.57
53rd Street	851	2-yr	24	261.41	262.08	261.87	262.17	0.008241	2.38	10.09	16.72	0.54
53rd Street	851	5-yr	39	261.41	262.31	262.03	262.43	0.008028	2.8	13.93	17.67	0.56
53rd Street	851	10-yr	64	261.41	262.62	262.26	262.78	0.007637	3.26	19.64	18.99	0.56
53rd Street	851	25-yr	78	261.41	262.77	262.37	262.96	0.007434	3.45	22.62	19.65	0.57
53rd Street	851	50-yr	82	261.41	262.81	262.4	263	0.007408	3.5	23.42	19.82	0.57
53rd Street	851	100-yr	114	261.41	263.11	262.62	263.34	0.007291	3.87	29.44	21.06	0.58
53rd Street	851	500-yr	170	261.41	263.54	262.96	263.84	0.007173	4.36	38.97	22.9	0.59
53rd Street	842		Bridge									
53rd Street	834	2-yr	24	261.29	261.98	261.69	262.03	0.004853	1.86	12.87	20.77	0.42
53rd Street	834	5-yr	39	261.29	262.22	261.83	262.29	0.004471	2.16	18.07	21.97	0.42
53rd Street	834	10-yr	64	261.29	262.56	262.03	262.65	0.004095	2.48	25.76	23.62	0.42

Proposed Conditions

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
53rd Street	834	25-yr	78	261.29	262.72	262.13	262.83	0.003968	2.63	29.7	24.42	0.42
53rd Street	834	50-yr	82	261.29	262.77	262.15	262.88	0.003954	2.67	30.75	24.63	0.42
53rd Street	834	100-yr	114	261.29	263.07	262.35	263.21	0.003901	2.96	38.58	26.14	0.43
53rd Street	834	500-yr	170	261.29	263.53	262.64	263.7	0.003857	3.34	50.91	28.36	0.44
53rd Street	807	2-yr	24	261.09	261.8	261.55	261.88	0.006826	2.24	10.73	16.93	0.5
53rd Street	807	5-yr	39	261.09	262.04	261.71	262.15	0.006474	2.61	14.97	17.97	0.5
53rd Street	807	10-yr	64	261.09	262.38	261.94	262.52	0.006107	3.02	21.19	19.41	0.51
53rd Street	807	25-yr	78	261.09	262.54	262.05	262.7	0.005996	3.2	24.36	20.1	0.51
53rd Street	807	50-yr	82	261.09	262.58	262.07	262.74	0.006011	3.26	25.17	20.27	0.52
53rd Street	807	100-yr	114	261.09	262.87	262.3	263.07	0.006168	3.65	31.22	21.52	0.53
53rd Street	807	500-yr	170	261.09	263.29	262.64	263.56	0.006367	4.18	40.69	23.33	0.56
53rd Street	767	2-yr	24	260.79	261.6	261.25	261.65	0.004326	1.92	12.52	17.69	0.4
53rd Street	767	5-yr	39	260.79	261.84	261.41	261.92	0.004503	2.29	17.01	18.86	0.43
53rd Street	767	10-yr	64	260.79	262.17	261.63	262.29	0.004902	2.71	23.64	21.75	0.46
53rd Street	767	25-yr	78	260.79	262.34	261.74	262.46	0.005	2.85	27.36	23.63	0.47
53rd Street	767	50-yr	82	260.79	262.38	261.78	262.51	0.004969	2.89	28.36	23.87	0.47
53rd Street	767	100-yr	114	260.79	262.68	261.99	262.84	0.004868	3.19	35.76	25.53	0.47
53rd Street	767	500-yr	170	260.79	263.12	262.35	263.32	0.00478	3.59	47.42	27.91	0.48
53rd Street	755		Bridge									
53rd Street	742	2-yr	24	260.62	261.31	261.08	261.39	0.007469	2.26	10.63	17.81	0.51
53rd Street	742	5-yr	39	260.62	261.53	261.24	261.64	0.007305	2.64	14.75	19.12	0.53
53rd Street	742	10-yr	64	260.62	261.84	261.45	261.98	0.007091	3.06	20.89	21.15	0.54
53rd Street	742	25-yr	78	260.62	261.99	261.57	262.15	0.006926	3.23	24.17	22.2	0.55
53rd Street	742	50-yr	82	260.62	262.03	261.59	262.2	0.006875	3.27	25.09	22.49	0.55
53rd Street	742	100-yr	114	260.62	262.33	261.82	262.53	0.006358	3.55	32.13	23.92	0.54
53rd Street	742	500-yr	170	260.62	262.78	262.13	263.02	0.005838	3.92	43.39	25.98	0.53
53rd Street	728	2-yr	24	260.51	261.21	260.97	261.28	0.00722	2.25	10.65	17.4	0.51
53rd Street	728	5-yr	39	260.51	261.43	261.13	261.54	0.007229	2.67	14.63	18.53	0.53
53rd Street	728	10-yr	64	260.51	261.73	261.35	261.88	0.007092	3.12	20.49	20.08	0.55
53rd Street	728	25-yr	78	260.51	261.88	261.45	262.05	0.006961	3.31	23.57	20.84	0.55
53rd Street	728	50-yr	82	260.51	261.92	261.49	262.1	0.006922	3.36	24.43	21.05	0.55
53rd Street	728	100-yr	114	260.51	262.23	261.71	262.43	0.006639	3.67	31.05	22.6	0.55
53rd Street	728	500-yr	170	260.51	262.68	262.04	262.93	0.006274	4.07	41.8	24.91	0.55
53rd Street	713	2-yr	24	260.4	261.09	260.85	261.17	0.007393	2.27	10.58	17.45	0.51
53rd Street	713	5-yr	39	260.4	261.31	261.02	261.42	0.00731	2.67	14.6	18.61	0.53
53rd Street	713	10-yr	64	260.4	261.62	261.24	261.77	0.007046	3.11	20.58	20.22	0.54
53rd Street	713	25-yr	78	260.4	261.77	261.35	261.94	0.006854	3.28	23.75	21.02	0.54
53rd Street	713	50-yr	82	260.4	261.82	261.38	261.99	0.006801	3.33	24.64	21.24	0.54
53rd Street	713	100-yr	114	260.4	262.13	261.6	262.33	0.006433	3.62	31.48	22.86	0.54

Proposed Conditions

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
53rd Street	713	500-yr	170	260.4	262.59	261.92	262.83	0.006013	3.99	42.56	25.26	0.54
53rd Street	698	2-yr	24	260.29	260.99	260.74	261.07	0.006947	2.21	10.84	17.7	0.5
53rd Street	698	5-yr	39	260.29	261.22	260.9	261.32	0.0069	2.61	14.96	18.94	0.52
53rd Street	698	10-yr	64	260.29	261.53	261.12	261.67	0.006594	3.02	21.17	20.67	0.53
53rd Street	698	25-yr	78	260.29	261.69	261.24	261.84	0.006375	3.18	24.49	21.54	0.53
53rd Street	698	50-yr	82	260.29	261.73	261.26	261.89	0.006316	3.22	25.43	21.77	0.53
53rd Street	698	100-yr	114	260.29	262.05	261.49	262.24	0.0059	3.49	32.66	23.53	0.52
53rd Street	698	500-yr	170	260.29	262.52	261.81	262.75	0.005473	3.83	44.33	26.13	0.52
53rd Street	660	2-yr	24	260.02	260.72	260.47	260.8	0.007086	2.25	10.69	17.31	0.5
53rd Street	660	5-yr	39	260.02	260.95	260.63	261.06	0.006856	2.63	14.85	18.46	0.52
53rd Street	660	10-yr	64	260.02	261.28	260.86	261.42	0.006294	3.01	21.26	20.11	0.52
53rd Street	660	25-yr	78	260.02	261.45	260.97	261.61	0.006002	3.16	24.7	20.94	0.51
53rd Street	660	50-yr	82	260.02	261.5	261	261.66	0.005924	3.2	25.66	21.16	0.51
53rd Street	660	100-yr	114	260.02	261.84	261.22	262.02	0.005456	3.44	33.1	22.84	0.5
53rd Street	660	500-yr	170	260.02	262.32	261.55	262.54	0.005105	3.8	44.74	25.23	0.5
53rd Street	619	2-yr	24	259.72	260.48	260.18	260.55	0.005295	2.04	11.75	17.61	0.44
53rd Street	619	5-yr	39	259.72	260.71	260.34	260.81	0.005492	2.44	15.98	18.75	0.47
53rd Street	619	10-yr	64	259.72	261.07	260.56	261.19	0.004963	2.78	23.03	20.53	0.46
53rd Street	619	25-yr	78	259.72	261.25	260.67	261.38	0.00471	2.91	26.82	21.42	0.46
53rd Street	619	50-yr	82	259.72	261.3	260.7	261.44	0.004647	2.94	27.88	21.66	0.46
53rd Street	619	100-yr	114	259.72	261.66	260.92	261.81	0.004306	3.17	35.91	23.42	0.45
53rd Street	619	500-yr	170	259.72	262.15	261.25	262.35	0.004138	3.53	48.15	25.87	0.46
53rd Street	576		Bridge									
53rd Street	536	2-yr	24	259.11	259.95	259.56	260	0.003694	1.81	13.24	18.1	0.37
53rd Street	536	5-yr	39	259.11	260.32	259.73	260.38	0.002739	1.93	20.19	19.95	0.34
53rd Street	536	10-yr	64	259.11	260.77	259.95	260.84	0.002352	2.15	29.79	22.25	0.33
53rd Street	536	25-yr	78	259.11	260.98	260.06	261.06	0.002295	2.27	34.43	23.28	0.33
53rd Street	536	50-yr	82	259.11	261.03	260.09	261.11	0.002285	2.3	35.7	23.55	0.33
53rd Street	536	100-yr	114	259.11	261.41	260.31	261.51	0.002274	2.53	45.02	25.47	0.34
53rd Street	536	500-yr	170	259.11	261.91	260.64	262.04	0.002415	2.91	58.45	28.01	0.35
53rd Street	478	2-yr	24	258.68	259.86	259.13	259.89	0.001127	1.23	19.57	19.61	0.22
53rd Street	478	5-yr	39	258.68	260.24	259.3	260.27	0.001112	1.43	27.3	21.44	0.22
53rd Street	478	10-yr	64	258.68	260.7	259.52	260.74	0.001181	1.7	37.61	23.67	0.24
53rd Street	478	25-yr	78	258.68	260.9	259.63	260.95	0.001234	1.83	42.53	24.65	0.25
53rd Street	478	50-yr	82	258.68	260.95	259.66	261.01	0.001248	1.87	43.86	24.92	0.25
53rd Street	478	100-yr	114	258.68	261.33	259.88	261.4	0.001366	2.13	53.58	26.75	0.26
53rd Street	478	500-yr	170	258.68	261.82	260.21	261.92	0.001598	2.52	67.34	29.14	0.29
53rd Street	390	2-yr	24	258.04	259.83	258.5	259.84	0.000254	0.74	32.53	22.7	0.11

Proposed Conditions

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
53rd Street	390	5-yr	39	258.04	260.2	258.66	260.22	0.000338	0.94	41.28	24.53	0.13
53rd Street	390	10-yr	64	258.04	260.65	258.88	260.67	0.000452	1.21	52.8	26.75	0.15
53rd Street	390	25-yr	78	258.04	260.85	258.99	260.88	0.000509	1.34	58.22	27.73	0.16
53rd Street	390	50-yr	82	258.04	260.9	259.02	260.93	0.000525	1.37	59.69	27.99	0.17
53rd Street	390	100-yr	114	258.04	261.27	259.24	261.31	0.00064	1.62	70.33	29.81	0.19
53rd Street	390	500-yr	170	258.04	261.75	259.57	261.81	0.000836	2	85.14	32.17	0.22
53rd Street	228	2-yr	24	256.86	259.82	257.31	259.83	0.000039	0.38	62.82	28.69	0.05
53rd Street	228	5-yr	39	256.86	260.19	257.48	260.19	0.000066	0.53	73.61	30.41	0.06
53rd Street	228	10-yr	64	256.86	260.63	257.7	260.64	0.000112	0.73	87.62	33.18	0.08
53rd Street	228	25-yr	78	256.86	260.82	257.81	260.83	0.000138	0.83	94.21	34.59	0.09
53rd Street	228	50-yr	82	256.86	260.88	257.84	260.89	0.000146	0.85	96.01	34.98	0.09
53rd Street	228	100-yr	114	256.86	261.23	258.06	261.25	0.000205	1.05	109.07	37.93	0.11
53rd Street	228	500-yr	170	256.86	261.7	258.39	261.72	0.000308	1.33	127.81	47.56	0.13
53rd Street	120	2-yr	24	256.8	259.8	258.23	259.81	0.000614	1.06	22.6	17.03	0.16
53rd Street	120	5-yr	39	256.8	260.14	258.54	260.17	0.000862	1.34	29.08	20.04	0.2
53rd Street	120	10-yr	64	256.8	260.56	258.93	260.61	0.00115	1.68	38.12	23.29	0.23
53rd Street	120	25-yr	78	256.8	260.74	259.09	260.8	0.001278	1.84	42.48	24.59	0.25
53rd Street	120	50-yr	82	256.8	260.79	259.14	260.85	0.001311	1.88	43.67	24.93	0.25
53rd Street	120	100-yr	114	256.8	261.12	259.5	261.2	0.001485	2.19	52.84	35.98	0.27
53rd Street	120	500-yr	170	256.8	261.54	259.95	261.65	0.001787	2.65	71.6	49.81	0.31
53rd Street	11	2-yr	24	257.9	259.59	259.16	259.66	0.005009	2.13	11.29	15.09	0.43
53rd Street	11	5-yr	39	257.9	259.89	259.4	259.98	0.005002	2.4	16.28	18.2	0.45
53rd Street	11	10-yr	64	257.9	260.26	259.68	260.37	0.005003	2.7	23.69	22.15	0.46
53rd Street	11	25-yr	78	257.9	260.42	259.81	260.54	0.005008	2.85	27.38	23.65	0.47
53rd Street	11	50-yr	82	257.9	260.46	259.84	260.59	0.005006	2.89	28.39	24.02	0.47
53rd Street	11	100-yr	114	257.9	260.76	260.09	260.92	0.005003	3.16	36.11	26.72	0.48
53rd Street	11	500-yr	170	257.9	261.15	260.44	261.34	0.005002	3.53	62.36	130.71	0.49