CRP693 W Cashmere Bridge Replacement Type, Size, & Location Study Report

Prepared for:
Chelan County Public Works - Engineering
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Executive Summary

This project will replace the existing bridge over the Wenatchee River that was constructed in 1929. The physical condition of this bridge has been deteriorating in recent years and requires constant maintenance and occasional closures. Replacement alignment of the existing bridge will be based on considerations of the environmental process determinations, railroad crossing requirements, WSDOT access requirements, and the CRP683 – Cashmere Area Transportation and Freight Study. The completed bridge will have a roadway section that will consist of two 12-foot lanes, two 5-foot shoulders and a separated bike and pedestrian path for a total width of 47 feet out-to-out. The project is scheduled to go to construction in 2019.

The design team developed a list of critical project criteria and improvements/impacts for the project. Criteria was developed for environmental, social and costs considerations associated with the project. The criteria used for comparison purposes included:
Environmental:

- Natural River Flow Conditions
- Natural Bank Habitat Conditions

Social:

- Temporary MOT Impacts
- Final Connections to Hay Canyon Road
- Aesthetics

Costs:

- Construction Costs (Bridge and Approaches)
- Future Maintenance and Inspection Frequency
- Right of Way Costs

The design team made careful examination of a final list of three viable structural bridge alternatives from all facets of engineering disciplines seeking an optimized bridge solution with respect to the above mentioned criterions. The three viable alternatives are:

1. Four-span pre-cast girder
2. Three-span steel girder
3. Cable-stayed

The study presented in this report leads to the conclusion that Alternative 2A (i.e., Three-span steel girder bridge built on existing alignment) best meets the criteria set forth by Chelan County. The recommended alternative will have good aesthetic value due to long spans and a superstructure that easily follows the vertical road profile.

The cost of this alternative is the lowest of all alternatives at approximately $18.8M. The team's recommendation is to advance design of the Alternative 2A on Alignment 1 through final PS&E phase.

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1. INTRODUCTION

This project will replace the existing bridge over the Wenatchee River that was constructed in 1929. The physical condition of this bridge has been deteriorating in recent years and requires constant maintenance and occasional closures. Replacement alignment of the existing bridge will be based on considerations of the environmental process determinations, railroad crossing requirements, WSDOT access requirements, and the CRP683 - Cashmere Area Transportation and Freight Study. The completed bridge will have a roadway section that will consist of two 12- foot lanes, two 5-foot shoulders and a separated bike and pedestrian path for a total width of 47 feet out-to-out. The project is scheduled to go to construction in 2019.

Road approach improvements will be on County roads and on the SR 2/SR 97 intersection. Other safety improvements include striping, guardrail, and signing. Multiple environmental design considerations include, but are not limited to SEPA/NEPA, historical structures, cultural resources, erosion control, stormwater detention and treatment, and Endangered Species Act consultation.

Design will take into consideration a railroad overpass, signalization of the SR 2/SR 97 and Hay Canyon Road intersection, and potential frontage road realignments. The work will also include project management inclusive of project scoping, multi-agency coordination, utility and railroad coordination, and preliminary design, final design, and construction phases.

The County will provide the Survey information necessary for the design and will lead the Right-of-Way and Environmental Permitting activities with support from the design team. It is also understood that at Chelan County's discretion, Construction Management services may be supplemented to the Consultant's contact.

The consultant team is composed of the following members:

TranTech – Project Management, Structural Engineering; Public Involvement
KPFF – Structural Engineering
Shannon & Wilson – Geotechnical Engineering
RH2 – Roadway, Drainage, and Utility Engineering
NHC – Hydrology Engineering
Transpo – Traffic Engineering
Ott Sakai – Constructability & Estimation

2. TYPE, SIZE & LOCATION (TS&L) STUDY

In order for this TS&L study report to be prepared, many design team members in various engineering disciplines had to provide contributions to support this investigation effort. In the following, a summary of these engineering activities is provided while detailed reports are provided in the appendices of this report.

Alternative Comparison Process:
The design team developed a list of critical project criteria and improvements/impacts for the project. Criteria was developed for environmental, social and costs considerations associated with the project. The criteria used for comparison purposes included:
Environmental:

- Natural River Flow Conditions
- Natural Bank Habitat Conditions

Social:

- Temporary MOT Impacts
- Final Connections to Hay Canyon Road
- Aesthetics

Costs:

- Construction Costs (Bridge and Approaches)
- Future Maintenance and Inspection Frequency
- Right of Way Costs

Each of the criteria was assigned a weight for comparison purposes. Further discussion on the comparison criteria and how it was used for this study is included in Section 2.8, TS&L Alternative Comparison, of this report.
In the following sections a summary of each engineering discipline with contributions to this study report is described in further detail.

2.1 SURVEYING

This activity is performed by the County's consultant and is provided to TranTech's team. Exhibit A provides a plan displaying the topo survey of the bridge site.

2.2 GEOTECHNICAL

This work element is performed by TranTech's geotechnical engineering team member Shannon & Wilson. In the following a summary of the geotechnical engineering considerations associated with each studied alternative is provided. A detailed technical memo on this topic is provided in Appendix B.

Shannon & Wilson drilled, logged, and sampled six exploratory borings along the planned alignment. Borings were located near the proposed bent locations and along approach fills.

From a geologic standpoint, the project site is in the eastern foothills of the Cascade Mountains in Central Washington. The region has a complex geologic and tectonic history.

Bridge structure foundation selection for this site depends on several factors, including required resistances due to axial and lateral loading, total and differential settlement tolerances, and construction considerations. Shannon & Wilson evaluated shallow and deep foundations including footings, driven H- and pipe piles, drilled shafts, and micropiles.

Shallow foundations (i.e., spread footings) are typically the least expensive foundation option and generally considered favorable if excavations can be completed in the dry and dense/hard bearing conditions are present.

At Bent 1 (south abutment), shallow foundations may be considered favorable given the relatively shallow, dense/hard soil conditions, provided the existing slope is capable of supporting the added load near the top. Shallow foundations may also be favorable at Bent 4 (north abutment) provided potential settlement criteria may be met. Spread footing construction at Bent 4 may require a deep overexcavation and potential replacement, potentially requiring a shored excavation.

At proposed Bent 2, within the Wenatchee River, a shallow foundation is likely feasible, although construction conditions will be more difficult than other alternatives, and project permitting may pose additional challenges. A shallow foundation at Bent 2 will require shoring and a cofferdam to construct the footing in the dry. Sheet piles may not be practical for cofferdam construction because of difficulty penetrating the anticipated very dense, coarse subsurface materials, likely including cobbles and potential boulders.

At proposed Bent 3, shallow foundation construction at this location will require a somewhat deep excavation to limit potential elastic settlements. The deep excavation will require careful planning given the proximity to SR 2 and existing bridge traffic. The excavation will likely require shoring.

On deep foundation alternatives, driven pile installations are typically performed by impacting or vibrating the pile into the ground to the prescribed bearing stratum. Pile driving vibrations and the associated damage risk should be evaluated for nearby residences, structures, businesses, etc. as warranted. Driven piles typically permit the cleanest installation and foundation construction.

It is anticipated that driven H-piles with tip protection (i.e., driving shoe) are a viable foundation alternative at each bent location permitted the associated vibration risk is permissible. The selected H-pile section should be robust to mitigate the dense driving conditions. Due to the difficult driving conditions, we estimate driven H-piles should be preferred over pipe piles.

Drilled shafts are slightly to moderately more costly than driven piles due to the equipment mobilization costs and casing requirements. Drilled shafts are a viable foundation alternative to provide adequate depth for lateral resistance considerations while protecting against potential scour provided adequate construction considerations for the dense, coarse subsurface materials with cobbles and potential boulders. Casing installation will likely require an air-rotary and/or oscillator drilling rig due to the coarse, dense/hard subsurface conditions. At Bent 2, it is anticipated that casing may be left in place to protect at the Bent 2 location. Alternatively, the concrete may stop short of the ground surface to allow casing removal and cut to desired elevation after excavation for column construction.

Micropiles are small-diameter (typically 3 to 10 inches) deep foundation elements constructed using high-strength steel casing and/or threaded bar capable of achieving relatively high capacities (in excess of 300 tons depending on micropile size and subsurface conditions). Micropile construction is feasible in very dense, coarse materials (i.e., cobbles and boulders) to bedrock using available drilling methods. The casing may extend to the full depth, or terminate above the bond zone with the reinforcing element extending the full depth. The reinforcing steel (typically an all-thread bar) is inserted through the micropile casing, and then high-strength cement grout is pumped into the casing. Micropile construction is typically cost comparable to drilled shafts but completed with relatively small drill rigs allowing construction within restricted access and/or low headroom areas. Micropiles should be considered given the dense, coarse subsurface materials.

Based on the encountered subsurface conditions, project discussions, proposed layouts, and the existing bridge structure estimated foundations, it is Shannon & Wilson's preliminarily estimate that driven H-piles and drilled shafts are viable foundation alternatives. The drivability of driven piles and constructability of drilled shafts should be further evaluated for the bridge final design. Specifically, it is recommended that sonic rotary drilling be completed at selected foundation locations.

On the need for a potential work bridge, Shannon & Wilson's preliminarily estimate is that driven H-pile and relatively small-diameter pipe piles provide feasible foundation types; although driving may require many blows to seat the foundations to the necessary depths to provide the required lateral resistance. At this time, it is recommended that additional sonic rotary drilling be completed at proposed bent locations. The sonic drilling will allow further drivability assessment of driven piles.

2.3 PERMITTING

Chelan County is providing engineering services for this work element. Following the 30% major milestone submittal from the design team, the County will start on this important critical path task.

2.4 HYDROLOGY

This work element is performed by TranTech's hydrological engineering team member Northwest Hydraulics Corp. (NHC). In the following a summary of the hydrological engineering considerations associated with each studied alternative is provided. A detailed technical memo on this topic is provided in Appendix C.

The Wenatchee Watershed drains approximately 1,188 square miles at the West Cashmere Bridge crossing (Figure 1). The watershed's maximum elevation is 9,370 feet above mean sea level with a mean basin elevation and annual precipitation of 800 feet above mean sea level and 66.7 inches, respectively.

The U.S. Geological Survey (USGS) operates two stream gages applicable to the site; Wenatchee River at Monitor Gage (USGS 12462500) located approximately 4.3 river miles downstream and Wenatchee River at Peshastin (USGS 12459000) located 9 river miles upstream of the West Cashmere Bridge crossing.

Flow data from the USGS Wenatchee River gages were evaluated utilizing the U.S. Army Corps of Engineers (USACE) Statistical Software Package (HEC-SSP), developed by the Hydrologic Engineering Center. The software allows the user to perform a statistical analysis of the hydrological data. Utilizing the HEC-SSP software, a USGS Bulletin 17B flow frequency analysis was performed and flood flow frequency curves were developed for the two USGS gages. Table 1 summarizes the calculated peak flows and the peak flows from the 2001 Flood Insurance Study (FIS) for Wenatchee River.

Table 1. Peak Flows for Wenatchee River at USGS Gages

Mean Recurrence Interval (MRI)	Wenatchee River at Monitor USGS Gage #12462500 (cfs)	Wenatchee River at Peshastin USGS Gage #12459000 (cfs)	USACE 2001 FIS at Monitor USGS Gage #12462500 (cfs)
2-Year	17,160	15,920	-
10-Year	28,135	24,635	**26,500**
20-Year	33,495	28,205	-
50-Year	41,545	33,050	**38,500**
100-Year	48,510	36,880	**48,700**
500-Year	56,360	40,875	**82,000**

In addition to peak flows, the project team requested flows that may be encountered during construction. Based on a fish window from July 1st – August 15th (assumed construction window for in-water work), NHC conducted a flow duration analysis using mean daily flows form the USGS Wenatchee River at Monitor Gage (#124625000). The gage recorded mean daily flows from 1962 through present (54 years). Table 2 shows the flow exceedance values for the 1-, 5-, 10-, 50-, and 95-percent exceedance within the assumed July 1st – August 15th construction window.

Table 2. Flow Exceedance Values for July 1st through August 15th at Wenatchee River Monitor Gage

Percent Exceedance	Mean Daily Flow (cfs)
1-Percent	11,400
5-Percent	8,640
10-Percent	7,080
50-Percent	2,300
95-Percent	615

The hydraulic analysis of the existing and proposed West Cashmere Bridge Crossings was performed utilizing the USACE HEC-RAS 5.0.1 computer program, a one-dimensional gradually varied steady flow numerical model. Four unique alternatives were provided by the TranTech team which were analyzed with the HEC-RAS model, namely: 1) existing conditions; 2) a two-span cable stay; 3) a three-span steel girder; and 4) a four-span concrete girder bridge structure.

The channel and floodplain geometry data for the model was obtained from topographic surveys of the site provided by Chelan County. Topographic data was submitted to NHC in electronic files formatted for AutoCAD Civil 3D. Cross section locations and extents for the HEC-RAS model were laid out on the topographic drawing using the standard requirements for one-dimensional hydraulic model development (e.g. cross section oriented perpendicular to flow direction, cross section extending to or beyond limits of effective flow, maximum spacing between cross sections to keep EGL changes less than one foot, cross sections located to capture all unique channel and floodplain changes, etc.). The geometry of each cross section was then obtained from AutoCAD Civil 3D; the program established ground elevations along the length of the cross section based on the location of the cross section on the digital terrain model created from the survey.

To obtain hydraulic characteristics within the project reach, the model geometry extends approximately 1,650 feet downstream and about 1,250 feet upstream of the existing West Cashmere Bridge. The upstream and downstream boundary locations are sufficiently far enough away from the project site to not influence hydraulics at the West Cashmere Bridge.

Tables 3 and 4 on the following page provide the results of the NHC's hydrology simulation results:

Table 3. Water Surface Elevations and Velocities for Existing Conditions

Mean Recurrence Interval (MRI) / Exceedance	Water Surface Elevation (Feet, NAVD88)	Channel Average Velocity (Feet/Second)
2-Year	800.7	8.4
100-Year	808.7/811.2*	13.4/13.5*
1-Percent	799.1	7.1
5-Percent	798	6.2
10-Percent	797.2	5.4
50-Percent	794.1	2.8
95-Percent	792.1	1.2

Water Surface Elevations and Velocities at Upstream Bridge Face (HEC-RAS XS 3034)
*Water Surface Elevation and Velocity from 2001 USACE Study

Table 4. 100-Year Water Surface Elevations and Velocities for Proposed Conditions

Proposed Alternative	100-Year Water Surface Elevation (Feet, NAVD88)	100-Year Channel Average Velocity (Feet/Second)
2-Span Bridge	808.2	14.0
3-Span Bridge	808.4	13.8
4-Span Bridge	808.6	13.7

Water Surface Elevations and Velocities at Upstream Bridge Face (HEC-RAS XS 3034)

Based on the preliminary hydrologic and hydraulic analysis performed, all three bridge alternatives should meet zero-rise criteria per the Washington Department of Fish and Wildlife (WDFW) 2013 Water Crossing Design Guidelines and WAC 220-660-190. Due to high flows, subsurface soils have not been collected in the area of the proposed in-water pier(s), which will assist in determining depths of scour for the proposed alternatives. NHC plans on conducting a site visit to collect pertinent data to support the team as the design progresses and support the need to collect geotechnical data at the proposed in-water pier(s) to be able to refine depth of scour calculations.

2.5 TRAFFIC

This work element is performed by TranTech's traffic engineering team member Transpo Group, Inc. (Transpo). A detailed technical memo on this topic is provided in Appendix D.

In the following a summary of the traffic engineering considerations associated with each studied alignment and viable structural concepts is provided.

Alignment A – Existing Bridge Alignment

Alignment A involves the demolition of the existing bridge and utilizing the existing alignment for new construction; therefore, this alignment requires long term detours for bridge traffic. The options for temporary traffic control will vary depending on what type of bridge is constructed.

2-Span Cable Stay Bridge: US 2 could remain open throughout the duration of the project. The bridge pier location will conflict with the access road which connects the existing bridge to US 2, and traffic will not be able to utilize the access road; however, under this alignment bridge traffic would be detoured so this concern is mitigated.

3-Span Steel Plate Girder Bridge: US 2 will require temporary closure and detour. A potential detour option involves utilizing the access road during the bridge closure. This temporary route will be constructed south of the existing US 2 alignment and would require minor grading and construction of temporary pavement. Construction of one of the piers would occur between US 2 and the access road under this option; however, the proposed detour is anticipated to still be viable.

4-Span Concrete Girder Bridge: US 2 will again require temporary closure and detour. It is anticipated the same detour option described for the 3-Span Steel Plate Girder Bridge could be utilized in this case. The proposed pier is not anticipated to have a negative impact to the temporary route for this bridge option.

Alignment B – New Alignment East of Existing Bridge

Alignment B generally maintains traffic across the existing bridge and would require no long term detours. Short duration bridge closures and detours would be required to transition the alignment to the new bridge. Once again, the options for temporary traffic control vary depending on what type of bridge is constructed.

2-Span Cable Stay Bridge: US 2 could remain open throughout the duration of the project; however, the bridge pier will again conflict with the access road. A proposed mitigation to the pier conflict involves realigning the existing bridge's north approach so the geometry would curve sharply to the south, then to the north to intersect US 2 at an angle that would allow for full or partial access to US 2. This option is anticipated to be costly due to realignment costs and the potential need for a temporary signal at the intersection.

3-Span Steel Plate Girder Bridge: US 2 will require temporary closure and detour. However, the detour cannot utilize the access road with the existing bridge still in use. The detour option will most likely only be feasible for night closures and with routes through the City of Cashmere.

4-Span Concrete Girder Bridge: The same closures and detours will be required as those mentioned for the case of a 3-Span Steel Plate Girder Bridge.

Emergency Vehicle Access:
The City of Cashmere does not have a hospital with emergency room. The nearest hospitals are Cascade Medical Center, located in Leavenworth, and Central Washington Hospital, located in Wenatchee. The project location is located almost equidistance from each hospital at 10.5 miles from Cascade Medical Center, and 14 miles from Central Washington Hospital. Given the hospitals' locations to the project, and their emergency responders' probable use of US 2 as an emergency route, it is likely they utilize the current bridge to access Goodwin Road and its immediate surrounding area. In the event the existing bridge and/or US 2 is closed to through traffic (short term or long term) at the project site, the contractor is required to halt operations and allow emergency access through the project site. If providing a route through the project site is not feasible, emergency responders for the hospitals can utilize marked detour routes.

The nearest fire department to the project location is Cashmere Fire Department, which is located at 101 Woodring Street in Cashmere. It is the only fire department in Cashmere, and is located to provide easy access to US 2 via Aplets Way or Cottage Avenue. Additionally, it has easy access to Sunset Highway via Division Street. Given the current geometry of the existing bridge and Goodwin Road, and the likely emergency routes, it is unlikely Cashmere Fire Department utilizes the current bridge. In the event US 2 is closed to through traffic (short term or long term) at the project site, the contractor is required to halt operations and allow emergency access through the project site via US 2. If providing a route through the project site is not feasible, Cashmere Fire Department can utilize marked detour routes, or Chelan County Fire District #3, located in Leavenworth, may be able to respond.

2.6 ROADWAY/ UTILITIES

This work element is performed by TranTech's roadway engineering team member RH2 Engineering, Inc. (RH2). A detailed technical memo on this topic is provided in Appendix E.

In the following a summary of the roadway engineering considerations associated with each studied alternative is provided.

Roadway Alignment

RH2 prepared two alignments for the West Cashmere Bridge Replacement project. The first alignment, Alignment A, is centered on the existing bridge. The second alignment, Alignment B, is to the east of the existing bridge with approximately 10 feet between the existing bridge and the new bridge to allow room for construction. The easterly alignment would allow traffic to use the existing bridge through most of construction, but would have the greatest impact to property owners on the south end of the bridge. Both alignments are constrained with tying into the private road on the north end. The design speed for Alignment A is 25 miles per hour (mph). Alignment B has a design speed between 15 to 20 mph because of the tighter curve required to tie into the private road. These alignments can be seen on **Figure 1 of Appendix E**.

Consideration was given to tying in further north along Hay Canyon Road to increase the design speed for Alignment B, but the County stated that it was their desire to have this alignment tie into Hay Canyon Road across from the private road.

Roadway Profile

Along previously-described Alignment A, profiles were created for a 7-foot and 10.5-foot girder depth. Profiles were controlled by the clearance needed over the Burlington Northern & Santa Fe (BNSF) railroad, US 2, and the frontage road. A 30-mile-per-hour design speed was used for vertical curves and a maximum grade of 7 percent was used. These profiles are shown on **Figures 2** and **3 of Appendix E**.

A minimum clearance of 16 feet, 10 inches was used over US 2 and the frontage road. A minimum clearance of 23.5 feet was used between the top of the railroad rail and the bottom soffit of the bridge. It was assumed that any future tracks would be at the same elevation as the existing. The envelope, where this clearance is required, extends between a 45-foot southerly offset and a 25-foot northerly offset from the centerline of the existing tracks. This clearance envelope is shown on **Figure 4 of Appendix E**.

It should be noted that the existing bridge does not provide the required clearance at the railroad, therefore any of the options will be higher than the existing bridge. This will result in impacts to south-end properties and to the intersection at Ruby Street. Slopes can be minimized using retaining walls, but access to the properties will be an issue because of the grade difference.

Roadway Section
The roadway section used for the project is two 12-foot lanes, two 5-foot shoulders, and a 10-foot sidewalk separated by a barrier. Bridge rails on either side bring the total width of the bridge to 47 feet.

The preliminary design assumes the bridge will be on a 2-percent cross slope so the bridge will drain west to east. A maximum 4-percent superelevation table was used for the preliminary design of the curve at the north end of the bridge.

Bridge Abutments
The north-end bridge abutment was set based on the location of the frontage road. Based on Exhibit 1600-2 in the WSDOT Design Manual, the clear zone is 10 feet from the edge of travelled way. For the preliminary design, the abutment was set 10 feet off the edge of the frontage road pavement. The location of the southerly abutment was set at a 45 foot offset from the centerline of the existing rail to accommodate future tracks.

2.7 STRUCTURAL

This work element is performed jointly by TranTech team's structural team members of KPFF and TranTech.

To investigate the viable structural bridge concepts that provides all of the County's desired attributes for this roadway facility, key staff from KPFF and TranTech participated in a workshop. The structural concepts had to meet important design constraints like satisfying clearance over BNSF rail and US 2; minimizing the number of piers with the Wenatchee River's 100-year flood zone, and being cost effective.

The team focused its attention to five viable structural concepts, namely:

1. Standard steel or concrete girder
2. Arch
3. Precast segmental concrete
4. Cable-styed
5. Truss

Through careful examination of each viable alternative, the workshop panel narrowed the list of these concepts to a final list of three for further investigation. The alternatives chosen for further investigation included the following:

1. Four-span pre-cast girder
2. Three-span steel girder
3. Cable-stayed

Pier locations were chosen to provide required clearance to the railroad (including a potential future track), required clearance to the frontage road at the north end, and to prevent conflicts with existing bridge foundations. Moreover, for the steel alternative, the design team is envisioning utilization of weathering steel which is not only a low maintenance material but also does not require a paint coating.
In the following sections, further details regarding the final viable alternatives are presented:

SUPERSTRUCTURE
The steel plate girder alternative is laid out to have spans applicable to that type of superstructure. This layout has three spans and two intermediate piers, one of which is located in the middle of the river. To keep bridge length to a minimum, all piers are skewed at 35 degrees to align with river flow, railroad, and US 2 orientation. A cast-in-place concrete deck is used. The recommended layout and cross-section for the steel girder alternative is shown in Appendix F.

The layout for the concrete girder bridge type has the same overall configuration as does the steel alternative except that the number of spans was increased to four thus reducing span lengths to be compatible with this type of superstructure.

In order to cross highway US 2 with required clearances the spliced post-tensioned (PT) concrete girder is required. Span length required over the highway is too long for a conventional pre-stressed girder. Layout resulted in two piers being within the Wenatchee River 100-year flood limits. The deepest available concrete girder section is chosen. Constant depth girders with cast–in-place deck are developed for this study. Layout for the concrete girder alternative and its cross section is shown in Appendix F.

The cable stay bridge was selected as a feasible bridge type for this study due to its ability to span the river without any piers within the 100-year flood limits. In fact, the cable stayed bridge is more advantageous than the other bridge types except for cost, which is about 43% higher than the steel or concrete girder alternatives. The Alternatives Comparison Matrix on page 24 highlights these advantages. The bridge envisioned is a two span, single tower structure with 64 paired stays supporting the bridge superstructure at 24'-0" in Span 1 and 19'-7" in Span 2. The tower pier rises 190' above the deck and is founded on four drilled shafts installed above the north river bank. Because the only intermediate pier is located on shore, the bridge is squared up (no skew) to facilitate use of rectangular panels throughout. The bridge deck (superstructure) consists of 12'-0" wide lightweight concrete panels in Span 1 and 9'-9½" wide standard weight concrete panels in Span 2. All are match cast to the previous panel and lifted to final position by crane located on a temporary work bridge. Temporary attachment is provided by temporary post tensioning between the last two or three panels erected. Plan and elevation are shown in Appendix F. Two feasible alternate panel configurations are also shown in Appendix F.

SUBSTRUCTURE

For each of the three structural alternatives considered (4-span precast concrete girder bridge, 3-span steel plate girder bridge, and 2-span cable stayed bridge), preliminary design for above ground and below ground substructure design was completed. Substructure members included abutment walls, columns, cross beams, pile/shaft caps, piles, and shafts. The purpose of the preliminary substructure design was to identify viable systems at abutments and intermediate piers and to size the substructure members for constructability and cost estimation.

Based on the roadway section described previously, the bridge will be approximately 47'-0" (55'-6" for cable stayed bridge) in total width. Given that this width is between 40' and 60', two cast-in-place (CIP) concrete columns have been assumed (WSDOT BDM 2.3.1) to connect the bridge superstructure to the underground foundations. It has been assumed that the columns will be connected and support the girders using a CIP concrete cap beam located either integrally with or directly below the girders.

Based on discussions with the project geotechnical engineer and constructability reviewer, both CIP concrete drilled shaft and driven steel pile foundations were considered. CIP concrete spread footings were not considered due to scour issues in the water and concerns for the costs and constructability of deep foundations at all locations. The bedrock is located approximately 53 to 74 feet below the ground line. In addition, settlement is expected at the north abutment, which is mitigated by drilled shaft or driven pile foundations.

Intermediate Piers: In order to reduce work within the water (Pier 2 and 3 for the Concrete Girder Alternative and Pier 2 for the Steel Girder Alternative) and to reduce excavation and shoring for the other piers, it was assumed that each column would be supported by its own single, larger sized drilled shaft. It was assumed that the top of shafts would be located 2 feet below ground line. The one exception is the land side intermediate pier (Pier 3 for steel girder alternative and Pier 4 for concrete girder alternative) which is 12 feet below ground. A deeper shaft with a larger silo has been proposed at this location in order to better balance out the heights (and therefore stiffnesses) of adjacent piers. See below for further discussion on the silos.

Only drilled shafts were looked at in detail during preliminary design for the in-water pier because a pile foundation would require a large pile cap constructed under the elevation of the water. It was determined that construction of single shafts would be beneficial due to lower costs, shorter construction durations, and potential impacts/risks to working in the water (e.g. high flows, shoring/cofferdam difficulties, etc.).

For the two-span cable stayed alternative, one CIP concrete "A" frame tower was used at the intermediate pier to which cables are attached. Four drilled shafts were used to support the tower. Each tower leg was supported by a pair of shafts which were connected together by a grade beam. The top of the grade beams was located 2 feet below ground. The location of this Pier 2 is above the river bank.

Abutments: Conventional L shape abutments were assumed for all three considered alternatives. Both drilled shafts and driven piles were considered at the abutments.

Substructure Analysis: After the survey data and the preliminary bridge layouts were defined, a global structural analysis model (SAP 2000 18) was developed for each alternative. The following assumptions were used in creation of the model:

- Column gross moment of inertial was reduced by half to take into account the section loss due to cracking (BDM 4.2.2).
- Static analysis for dead and live loads and response spectrum analysis for seismic loads was conducted.
- 10-feet of scour for 100-year return period scour event was assumed for the in-water piers. Scour was considered with the Extreme Event I limit state. According to BDM 7.2.6.2, it was assumed that the soil in the upper 25 percent of the 100-year scour depth, which is 2.5 feet, has been removed when identifying the soil resistance to drilled shafts.
- Abutments were conservatively excluded from the earthquake resistant system except that the translational movement in transverse direction was assumed to be restrained for the cable stayed option.
- The depths to fixity of drilled shafts were assumed to be 2.5 times the shaft diameter at the intermediate piers with cantilever condition and 4 times the shaft diameter and 8 times pile section depth at the abutments with fix-fix condition.
- Load Combinations for Strength-I, Service-I and Extreme-I limit states were considered.
- Live load factor of 0.5 was used for the Extreme-I Limit State load combination.

- Force based design methodology was used for column seismic design for this study.
- The bridge Operational Category was assumed to be “Essential”, which designates an “R” factor of 3.5 per AASHTO LRFD Article 3.10.7.
- In addition to the loading criteria, the columns were sized and reinforced to satisfy slenderness criteria (kL/r <100), minimum lateral strength (AASHTO Seismic 8.7.1), P-delta effects (AASHTO Seismic 4.11.5), and 1% minimum and 4% max reinforcement ratio (AASHTO Seismic 8.8).
- The balanced stiffness requirement from AASHTO Seismic 4.1.2 was checked but not satisfied. Although this requirement is only mandatory for SDC D, the design team considered the length/stiffness of the water pier and the land pier immediately north of the river (Pier 3 for steel girder alternative and Pier 4 for concrete girder alternative) to be an extremely unbalanced condition. As a result, a 12-ft deep silo around column was specified at the landside intermediate pier to mitigate this unbalanced condition.
- The drilled shafts were capacity designed to resist the overstrength column plastic forces.
- There is no redundancy in the substructure of the cable stayed option. Therefore, no plasticity was assumed to occur in the substructure, and all substructure elements were designed to keep elastic.
- The abutment wall and foundation were sized to resist dead and live load, static and dynamic earth pressure, inertial loads from the superstructure (20% reaction) and substructure.

Substructure Analysis Results: For the intermediate piers and the Concrete Girder and Steel Girder Alternatives, 5-ft diameter columns connected by a crossbeam at the top were found to be required. Each column need to be supported by a 9-ft oversized drilled shaft.

For the two-span cable stayed alternative, one "A" frame tower was used at the intermediate pier to which cables are attached. Four 10-ft drilled shafts are required to support the tower. Each tower leg is supported by a pair of shafts which are connected by a grade beam. The top of the grade beam was assumed to be 2 feet below ground.

Two types of foundations were sized for the abutment wall and foundations for all three alternatives; shafts and piles with a large CIP concrete shaft/pile cap connecting all the foundation elements. Eight (2 x 4) 5-ft diameter drilled shafts or Eighteen (3 x 6) HP16x162 driven piles were found to be necessary.

2.8 TS&L ALTERNATIVES COMPARISON

The design team developed a list of critical project criteria and improvements/impacts for the project. Importance factors were developed for each issue in the three different categories: environmental, social and costs. The categories and issues used for comparison purposes included:

Environmental Issues:

- Natural River Flow Conditions
- Natural Bank Habitat Conditions

Social Issues:

- Temporary MOT Impacts
- Final Connections to Hay Canyon Road
- Aesthetics

Project Costs:

- Construction Costs (Bridge and Approaches)
- Future Maintenance and Inspection Frequency
- Right of Way Costs

Importance Factors (IF) values were assigned a number (value out of 100). The values were set to be in conformance with the overall project goals.
For each of the six alternatives compared (i.e., 3 alternatives per alignment), a metric was defined and a scoring state value was assigned. In general, the scoring states used were:

State 1 – Substantial Benefit
State 2 – Moderate Benefit
State 3 – Minor Benefit
State 4 – No Benefit or Worst Condition

Each of the scoring state values were multiplied by the associated importance factor. After each of the issues were scored, the alternatives were ranked. The smallest weighted score indicates the alternative with the most benefits.

Comparison Criteria:

Environmental - Natural River Flow Conditions (IF = 14 of 100):

Each of the alternatives was compared for impacts to the natural river flow (e.g. how many obstructions are being placed within the 100-year flood flows).

Currently there are two existing piers located within the 100-year flood flow. The existing piers consist of approximately 5 feet wide and 25 feet long pier walls. Proposed in-water piers will consist of (2) 5-ft diameter columns, spaced 36 feet apart (center to center). Preliminary hydraulic analysis checked conditions for 1, 2 and zero new piers in the water. Based on the hydraulic analysis all 3 alternatives should meet zero-rise criteria.

Scoring State:

- State 1 – No piers in the 100-year flood flow
- State 2 – One smaller pier in the 100-year flood flow
- State 3 – Two smaller piers in the 100-year flood flow
- State 4 – More numbers or larger piers in the 100-year flood flow (not used)

In addition, the north (left) bank appears to have been filled/constricted when the existing bridge was built in order to shorten the truss span length and to make room for the on/off ramp connecting the bridge to US 2. The potential for the constriction to be removed was not included in this comparison, however, all alternatives would allow for the constriction to be removed, but will place an additional pier in the water.

In addition to the hydraulic flow conditions, this location of the Wenatchee River is popular among rafters and kayakers. The design team solicited anecdotal feedback from an experienced kayaker and the Osprey Rafting Company that have been down this stretch of river many times. We were told that this is not a particularly challenging part of the river to navigate because there is good sight distance as you approach the bridge. One entity preferred one pier and one preferred two but neither indicated a strong preference. It was suggested to keep the piers out of the deepest part of the river where the strongest current is and most rafts are regardless of configuration. Both discussed the issue of debris getting hung up in the pier more than the location of the pier itself. It was noted that if 2 columns are used for the pier that debris can get hung up between them and that a solid pier is more desirable. It should be noted that the River has almost a level bottom throughout majority of its cross section at the project site.

Environmental - Natural Bank Habitat Conditions (IF = 14 of 100):

Each of the alternatives were compared for impacts to the riverbanks and whether or not they were impacted by intermediate piers close to the bank and/or the need for scour protection on the bank.

Currently, the south (right) riverbank is covered with large rock and concrete pieces. The concrete is presumably from a failed concrete drainage trench (located on bridge as-built). The north (left) riverbank is currently covered in smaller rounded rock.

Because the south (right) riverbank supports the railroad grade, it is recommended that the existing scour protection remain in place on this slope to reduce liability/future maintenance to the County.

Scoring State:

- State 1 – No piers in the river; Potential to remove riprap on the north bank
- State 2 – In-water piers located away from the riverbanks, Potential to remove riprap on the north bank
- State 3 – In-water piers located away from the riverbanks, No potential to remove riprap on the north bank (not used)
- State 4 – New piers located near the river banks, therefore requiring additional riprap for scour protection.

Social - Temporary MOT Impacts (IF = 10 of 100):

Scoring State:

- State 1 – Existing bridge traffic and direct US 2 connections maintained for a duration of construction; no falsework required within US 2 limits (short term lane closures will be needed for girder placement)
- State 2 – Existing bridge traffic maintained for a duration of construction (indirect connections or connection through construction area to US 2); falsework required within US 2 footprint therefore longer term lane closures and temporary restriping necessary, in addition to short term road closures with longer detour around site for girder placement
- State 3 – Existing bridge traffic closed throughout the duration of construction; no falsework required within US 2 limits (short term lane closures will be needed for girder placement)
- State 4 – Existing bridge traffic closed throughout the duration of construction; falsework required within US 2 footprint; therefore, longer term lane closures or and temporary striping necessary, in addition to short term road closures with longer detour around site OR long term road closures with simple detour through site for girder placement

Social - Final Connections to Hay Canyon Road (IF = 10 of 100):

Access from the existing bridge to the intersection with US 2 is via a steep and sharply curved road under the north bridge approach spans. This road has substandard curves and sight distance. Each of the proposed alternatives carry bridge traffic over the top of US 2 and tie into Hay Canyon Road. There will be a stop-controlled intersection at Goodwin Road and Hay Canyon Road. Traffic to and from US 2 will then access US 2 via a signalized intersection at Hay Canyon Road.

Scoring State:

- State 1 – The north end of Goodwin Road ties into Hay Canyon Road immediately across from the existing private residential road; County standards for design speed and curve radius are met or exceeded

- State 2 – The north end of Goodwin Road ties into Hay Canyon Road immediately across from the existing private residential road; County standards for design speed and curve radius are not met, but are reasonable for the expected use at this site
- State 3 – (not used)
- State 4 – (not used)

<u>Social – Aesthetics (IF = 6 of 100):</u>

The existing bridge is a historical structure and is prominently located where it is viewed by traffic on US 2 and is considered by some a gateway to the City of Cashmere. The existing truss spans over the river allow for a person viewing the bridge to see "through" the structure to the river and views beyond (apparent structure depth than cannot be seen through is approximately 2 feet).

The new structures for Alternatives 1 and 2 consist of concrete or steel girders that are shallower in depth than the existing truss (approximately 9.5-ft and 10.7-ft, respectively), but could be viewed as very "tall" compared to the existing truss because one cannot see through them. Comparing the aesthetic impact between steel versus concrete girders is very subjective. While the steel girders are slightly deeper than the concrete girders, they also require less piers, which create visual obstructions to the view, and are more adaptable to the vertical and horizontal profile curvatures. However, some people view the color of a concrete girder more aesthetically pleasing than a weathered steel finish.

The cable stayed alternative will allow for a much shallower span (approximately 6-ft), with the use of the tower and cables. In addition, some may view the "signature style" of the bridge as appealing in this location.

Scoring State:

- State 1 – Shallower depth than existing truss (not used)
- State 2 – Shallowest viable depth and signature style bridge
- State 3 – Standard (Concrete or Steel) girder depth with less piers
- State 4 – Standard (Concrete or Steel) girder depth with more piers

<u>Project Costs - Construction Costs (Bridge and Approaches) (IF = 25 of 100):</u>

Each of the three alternatives were analyzed for construction costs. Details on the cost estimates are provided in Appendix G of this report, Bridge Alternatives Opinion of Cost. Please note that the costs are developed for Alignment A only; for Alignment B, all of the cost items from Alignment A are increased by 15%, except for the work trestle item that is increased by 100%. The reasoning behind the former factor is due to the fact that the contractor needs to be working adjacent to the traveling public and will also be faced with a tight working environment at the Cashmere end of the project site. The reasoning behind the latter increase is due to the fact that we envision two work trestles are needed for Alignment B option, namely; one for the construction of the new bridge and another for demolition of the old bridge, following diversion of traffic to the new bridge.

Scoring State:

- State 1 – Estimated construction cost is the lowest of all alternatives.
- State 2 – Estimated construction cost is the second lowest of all alternatives
- State 3 – Estimated construction cost is the highest of all alternatives

- State 4 – Estimated construction cost is the highest of all alternatives and a higher risk factor for increase.

<u>Project Costs – Future Maintenance and Inspection Frequency (IF = 15 of 100):</u>
Each of the proposed alternatives will require some level of future inspection and/or maintenance, therefore the alternatives have been compared using an approximately 30 year duration.

Inspection requirements are assumed to be as follows:

- 24-month NBIS routine inspections: All three alternatives
- UBIT Inspections: 48 months for Non-Fracture Critical Steel Bridges, 72 months for Concrete Bridges with Fixed Bearings or no bearings at the interior spans
- 60-month NBIS underwater inspections: Concrete and Steel Girder Alternatives only
- 10-year (120 month) in-depth cable/tower inspections: Cable Stayed Alternative

Please note that Fracture Critical Inspections are not included in this list, because each of the proposed alternatives are redundant systems.

In addition to inspection frequency, future scheduled maintenance has also been considered. No future schedule maintenance has been assumed for the concrete girder alternative. The steel girder alternative is proposed to be weathering steel. The weathering steel patina does not require painting or sealing. However, it is recommended that the ends of the girders, particularly under the expansion joints, be painted or sealed. This area will need to be repainted approximately every 30 years.

As part of the in-depth cable inspection, many new cable stay bridges are constructed with extra strands provided in the cables. This allows for removal of strands to determine the presence or absence of corrosion within the cable weather protection (e.g. sheathing). Removal of an extra strand is recommended approximately every 10 years.

Summary:

Concrete Girders over 30 years:

- (15) routine inspections
- (5) UBIT inspections
- (6) Underwater Inspections
- No scheduled paint/sealer requirements after construction

Steel Girders over 30 years:

- (15) routine inspections
- (8) UBIT inspections
- (6) Underwater Inspections
- (1) scheduled repainting/resealing 30 years after construction (limited to approx 20-ft at the expansion joints/abutments only)

Cable-Stay Bridge over 30 years:

- (15) routine inspections
- (5) (concrete superstructure) to (8) (steel superstructure) UBIT inspections
- (0) Underwater Inspections
- (3) in-depth cable/tower inspections
- No scheduled paint/sealer requirements after construction if concrete superstructure, (1) scheduled repainting/resealing 30 years after construction if steel superstructure.

We have used our past experience to estimate the cost associated with above mentioned routine inspection and maintenance activities and have found out that for a 30-yr window, the present value cost is approximately $170K, $285K, and $375K for concrete, steel, and cable-stay bridge alternatives respectively. Hence, the cost associated with routine inspection and maintenance activities for a 75-yr design life of a new bridge will be $425K, $712.5K, and $937K for concrete, steel, and cable-stay bridge alternatives respectively.

In order to incorporate the effect of this attribute in our Comparison Matrix, we have used the least ordinal number for the concrete alternative, followed by the steel and cable-stayed alternatives.

Scoring State:

- State 1 – Routine Inspections only, no Cable, Tower or Underwater Inspections; No specific routine future maintenance anticipated, beyond normal roadway upkeep (not used)
- State 2 – Routine Inspections only, no Cable or Tower Inspections. Maximum UBIT frequency (72 months). Will require Underwater Inspections; No specific routine future maintenance anticipated, beyond normal roadway upkeep
- State 3 – Routine Inspections only, no Cable or Tower Inspections. Middle UBIT frequency (48 months). Will require Underwater Inspections; Future repainting/resealing required at steel girder ends.
- State 4 - Routine Inspections and Cable or Tower Inspections on a 10-year interval. Does not require Underwater Inspections; No specific routine future maintenance anticipated, beyond normal roadway upkeep

Project Costs - Right of Way Costs (IF = 6 of 100):
Each of the alignments will require acquisition of Right of Way (ROW).

At the south bridge approach, the proposed roadway will need to be higher in elevation than the existing roadway in order to provide required clearance over the BNSF ROW. The deeper the superstructure, the largest footprint impact and therefore the largest assumed cost. In addition, there may be impacts to the roadway elevation at the intersection of Goodwin Road and Ruby Street with the deepest girder alternatives. These property impacts could be permanent acquisition, permanent easements or possibly temporary easements.

At the north bridge approach, the County will need to permanently acquire property in order to construct the north bridge approach roadway. It has been assumed that permanent easements will not be preferred for the County or existing land owner. The ROW requirements for Alignment B are more than for Alignment A, because the roadway grades and design speeds require that the intersection on Hay Canyon Road are further north.

Scoring State:

- State 1 – No permanent ROW
- State 2 – Minor permanent ROW
- State 3 – Moderate permanent ROW
- State 4 – Highest Permanent ROW

West Cashmere Bridge Replacement – Alternatives Comparison Matrix

		Importance Factors (out of 100)	**Alignment A** Existing Bridge Alignment, ties into Hay Canyon Road at the existing private road.			**Alignment B** Bridge Alignment to the east of existing, ties into Hay Canyon Road approx. 150 ft north of the existing private road.		
			Alt. 1A concrete girders 4 spans	**Alt. 2A** steel plate girders 3 spans	**Alt. 3A** cable-stay 2 spans	**Alt. 1B** concrete girders 4 spans	**Alt. 2B** steel plate girders 3 spans	**Alt. 3B** cable-stay 2 spans
Environmental:	Natural River Flow Conditions	14	3	2	1	3	2	1
	Natural Bank Habitat Conditions	14	4	2	1	4	2	1
Social:	Temporary MOT Impacts	10	4	3	3	3	2	2
	Final Connections to Hay Canyon Road	10	1	1	1	2	2	2
	Aesthetics	6	3	3	2	3	3	2
Costs:	Construction Costs (Bridge and Approaches)	25	2	1	4	3	2	4
	Future Maintenance and Inspection Frequency	15	2	3	4	2	3	4
	Right of Way Requirements	6	2	3	1	3	4	2
Total Score: Sum (Importance Factor x State)			**258**	**202**	**246**	**289**	**233**	**252**

3. CONCLUDING REMARKS AND RECOMMENDATIONS

The study presented in this report leads to the conclusion that Alternative 2A best meets the criteria set forth by Chelan County. Although this alternative has a pier in the river, it is less obstructive than the existing bridge which has two piers in the water and will satisfy zero-rise criteria.

Because the one pier in the river is located away from the river banks, the natural bank habitat conditions will not change from existing conditions.

The recommended alternative will have good aesthetic value due to long spans and a superstructure that easily follows the vertical road profile.

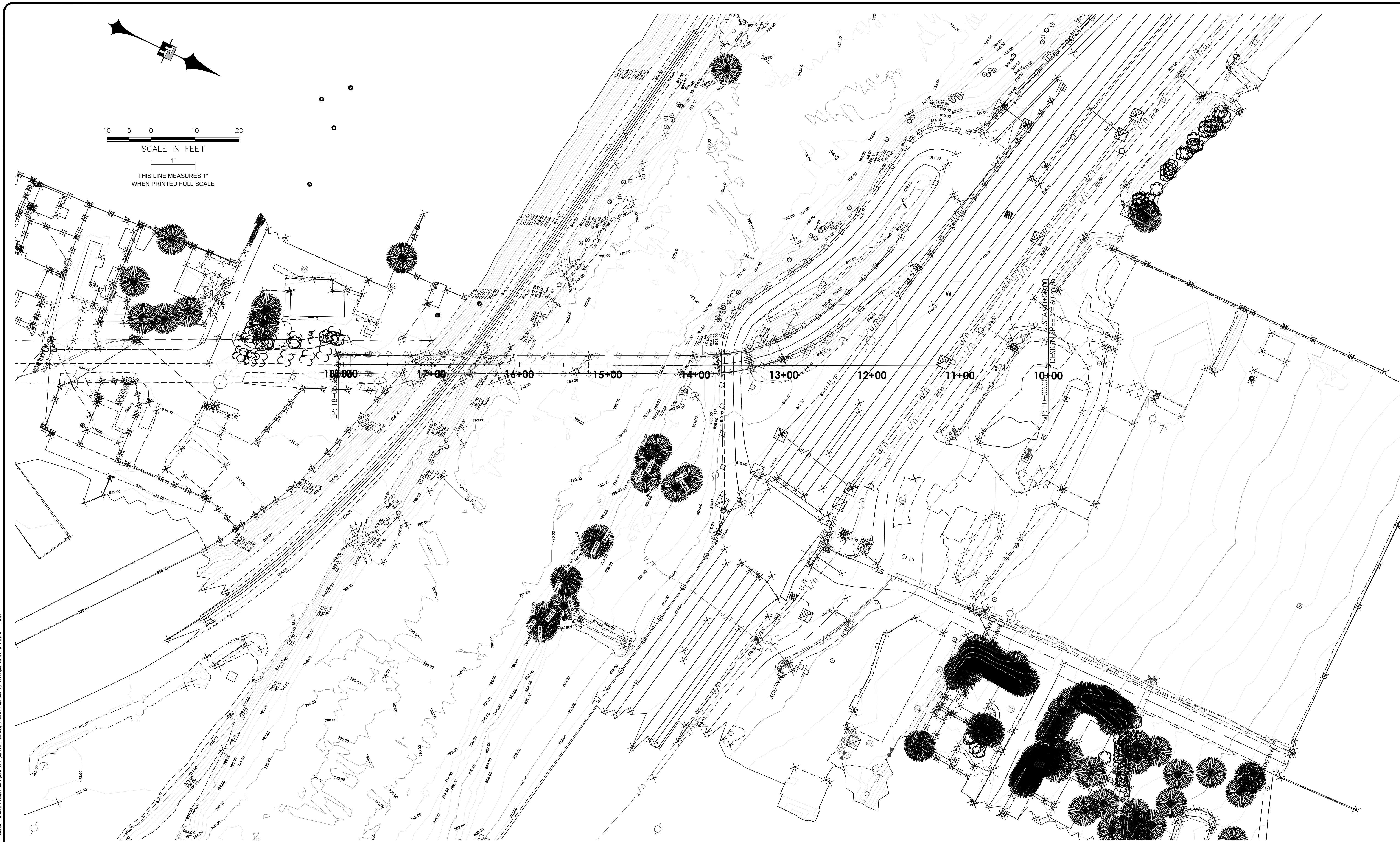
The cost of this alternative is the lowest of all alternatives at approximately $18.8M. Future maintenance should be low with a two year inspection cycle. The weathering steel girders will be coated under both expansion joints to detour damage should leakage occur. Typically, the steel should not need attention except for limited removal of debris in confined areas, if any, for 30 years.

Locating the new structure on the same centerline as the existing bridge (Alignment A) will result in the lower cost bridge than offsetting it to the east (Alignment B). Required right-of-way purchase will also be less. However, the team recognizes that additional maintenance of traffic (MOT) effort will be required.

Also, locating on the existing centerline allows a larger radius curve at the north end of the project, and thus, a greater design speed (i.e., 20 mph) in that location.

The team's recommendation is to advance the design of Alternative 2A on Alignment 1 through final PS&E phase.

Appendix A - Surveying Map



DATE:	ISSUED / REVISED	BY:
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Chelan County
Public Works Department
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CRP693 W CASHMERE BRIDGE REPLACEMENT

SITE SURVEY

PROJECT NUMBER
2016001

SHEET
#

PAGE
1 OF 1

Appendix B - Geotechnical Investigations Technical Memo

July 6, 2016

Mr. Kash Nikzad, PhD, PE
Principal
TRANTECH Engineering, LLC
12011 NE First Street
Suite 305
Bellevue, Washington 98005

RE: PRELIMINARY GEOTECHNICAL ENGINEERING ASSESSMENT; CRP693 - WEST CASHMERE (GOODWIN ROAD/WENATCHEE RIVER) BRIDGE REPLACEMENT, CHELAN COUNTY, WASHINGTON

Dear Mr. Nikzad:

Shannon & Wilson, Inc. (Shannon & Wilson) prepared this Preliminary Geotechnical Engineering Assessment letter report for the West Cashmere (Wenatchee River) Bridge Replacement project in Chelan County, Washington. This letter report summarizes the encountered subsurface conditions and provides a brief discussion of foundation alternatives for the proposed replacement structure. We also provide preliminary foundation recommendations based on encountered subsurface conditions and project discussions.

BACKGROUND

Chelan County (County) plans to replace the existing West Cashmere (Wenatchee River) Bridge, a.k.a., Goodwin Road Bridge, in Chelan County, Washington. The existing bridge extends north across the Wenatchee River from the west end of the Cashmere community and terminates along the south side of State Route (SR) 2. Hay Canyon Road intersects SR 2 just east of the bridge alignment on the north side of SR 2. The SR 2/Highway (Hwy) 97 interchange is approximately 3⅓ miles (straight line distance) northwest of the project site, and the community of Wenatchee is approximately 8⅛ miles southeast. We show the project site location in Figure 1, Vicinity Map.

Near the south approach, the project alignment consists of a relatively tall, steep cut bank extending down to the Wenatchee River south side. The south slope consists of two segments, the first extending from the approach area to the Burlington Northern Santa Fe (BNSF) railroad line, which extends approximately east-west through the project alignment. The second segment of the steep south cut slope extends below the BNSF rail line to the Wenatchee River. The south slope is mostly absent of substantial vegetation, exhibiting bare soils consisting of sandy gravels with cobbles, some of which are loose.

The Wenatchee River north cut bank is approximately 10 to 15 feet tall and nearly vertical at the river edge. The bank is vegetated with dense grasses and heavy shrubs.

The topography beyond the Wenatchee River north bank generally slopes down to the south at approximately 3 to 10 percent. The existing Goodwin Road alignment includes a slip ramp exit from SR 2 and a loop ramp onto and off of the existing bridge. The loop ramp intersects SR 2 immediately across from Hay Canyon Road at a stop light.

SR 2 extends approximately east-west through the project alignment north of the Wenatchee River. SR 2 is approximately 95-foot-wide (shoulder to shoulder) through the project alignment area with two travel lanes in each direction and a center turn lane. Sunburst Lane extends north of SR 2 and parallel. Private property with a vacant building lies just above Sunburst Lane elevation and to the north. The private property also contains loading docks west of the building and exhibits generally tall grasses/weeds elsewhere with an occasional brush pile. We show the project alignment, topography, and nearby site features in Figure 2, Site and Exploration Plan.

EXISTING AND PROPOSED IMPROVEMENTS

The existing bridge consists of an approximately 503-foot-long, 25¼-foot-wide, eight-span structure of multiple types. The approximately 124-foot-long main span consists of steel truss with a cast-in-place concrete deck. The south abutment is located at the top of the approximately 35- to 50-foot-tall cut bank above the BNSF railroad and Wenatchee River. Three relatively short spans extend from the south abutment to Pier 4, then the two longer main spans extending across the Wenatchee River, culminating in three relatively short spans at the Wenatchee River north cut bank. Pier 5 is located within the middle of the Wenatchee River. The existing bridge crossing has been rated Structurally Deficient with a 25.2 sufficiency rating. The bridge is posted for (restricted) loads. As-built drawings indicate the existing structure is founded upon driven piles, although the exact material type (i.e, timber or steel) is not listed.

The design team and County are considering two-, three-, and four-span replacement structures. The conceptual two-span structure includes a cable-stayed central pier at the Wenatchee River north cut bank, a longer span extending from the south approach and a slightly shorter span across SR 2. A three-span structure will utilize a middle pier within the Wenatchee River to break the longest span. We understand the four-span structure may utilize a pier near each bank of the Wenatchee River. The design team preliminary engineering includes steel and concrete girders for the three- and four-span structure alternatives.

The design team provides replacement bridge structure preliminary loads for the indicated pier locations as shown in Tables 1 through 3 below corresponding to the indicated structure alternative.

TABLE 1
TWO-SPAN, CABLE-STAYED STRUCTURE LOADS

Location	Foundation Type	Preliminary Loads (kips)		
		Strength	**Service**	**Extreme**
Piers 1 & 3 (Abutments)	4-foot-diameter Drilled Shaft	Vertical loads are variable depending on the final design, therefore, the design team estimates the Concrete Girder loads in Table 3 below provide a reasonable estimate of potential load ranges.		
	HP 16 Pile			
Pier 2	8-foot-diameter Drilled Shaft	10871	8320	13365

TABLE 2
THREE-SPAN, STEEL GIRDER STRUCTURE LOADS

Location	Foundation Type	Preliminary Loads (kips)		
		Strength	**Service**	**Extreme**
Piers 1 & 4 (Abutments)	4-foot-diameter Drilled Shaft	1330	926	1214
	HP 16 Pile	461	323	454
Pier 2	8-foot-diameter Drilled Shaft	3050	2251	2675
Pier 3	8-foot-diameter Drilled Shaft	2985	2199	2624

TABLE 3
FOUR-SPAN, CONCRETE GIRDER STRUCTURE LOADS

Location	Foundation Type	Preliminary Loads (kips)		
		Strength	Service	Extreme
Piers 1 & 5 (Abutments)	4-foot-diameter Drilled Shaft	1502	1064	1352
	HP 16 Pile	537	384	515
Pier 2	8-foot-diameter Drilled Shaft	3349	2532	2989
Pier 3	8-foot-diameter Drilled Shaft	3279	2481	2946
Pier 4	8-foot-diameter Drilled Shaft	3693	2790	3232

FIELDWORK AND LABORATORY TESTING

Shannon & Wilson drilled, logged, and sampled six exploratory borings along the planned alignment. We planned our borings near the proposed bent locations and along approach fills, and adjusted the locations based on traffic and utility constraints (Figure 2). Our geologist recorded boring locations with a handheld global positioning system and measured off nearby structures. We present the boring logs in Appendix A.

Drilling

Under subcontract to Shannon & Wilson, HazTech Drilling, Inc. (HazTech) of Meridian, Idaho completed drilling from April 13 through 18, 2016. HazTech drilled the borings with a CME 85 truck-mounted drill rig. All borings began using approximately 4¼-inch inside diameter, 8¼-inch outside diameter (O.D.), hollow-stem augers. While drilling boring B-1, the augers deflected off vertical at 15 bgs due to likely boulders, causing refusal at 20 feet below ground surface (bgs). While drilling boring B-2, the drillers switched to 4¾-inch O.D. casing advance drilling methods after the auger reached refusal at approximately 7½ feet bgs.

When we encountered suspected bedrock, the drilling subcontractor switched to 3¾-inch O.D. HQ3 wire-line rock coring methods. HQ3 rock coring was used at borings B-2, B-4, B-5, and B-6 starting at 64.7, 50, 50, and 28 feet bgs, respectively. The HQ3 wire-line coring method uses a core bit embedded with diamond chips to obtain approximately 2.4-inch-diameter core samples.

SHANNON & WILSON, INC.

We attempted a seventh boring (B-3) through the bridge deck in the river channel. However, high water and fast velocity flow deflected the drill casing downstream, wedging the casing in the bridge deck before the casing could be set on the river bottom. We did not drill boring B-3 due to extreme Wenatchee River currents which prevented setting the drill casing into the river bottom.

Our subcontractor cut 10-inch diameter holes in the bridge deck for borings B-2 and B-3. HazTech repaired the bridge deck with quick setting concrete and rebar upon completion of drilling activities.

Sampling

We obtained disturbed soil samples at approximately 2½-foot intervals for the first 25 feet below ground surface (bgs) using a 2-inch O.D. Standard Penetration Test (SPT). We conducted SPTs in accordance with ASTM International (ASTM) Designation: D1586, Test Method for Penetration Test and Split-Barrel Sampling of Soil. We drove the SPT samples 18 inches (three 6-inch increments) below the augers with an automatic hammer weighing 140 pounds and free-falling 30 inches. We recorded the number of blows required to advance the split-spoon through each 6-inch increment. The SPT resistance, or N-value, is defined as the number of blows required to drive the sampler from 6 to 18 inches below the drill casing. The N-value is reported as the number of blows per 1 foot of penetration. When 50 blows are required to achieve penetration of 6 inches, or less, we halted testing and recorded the number of blows with the corresponding penetration. The N-value provides an indication of the relative density, or consistency, of the soil and is plotted on the boring logs. Our field representative placed disturbed soil samples into labeled, sealed plastic bags.

We collected approximately 2.4-inch-diameter rock core samples in borings B-2, B-4, B-5, and B-6. To collect rock core samples using the HQ3 wire-line coring methods, the drilling subcontractor lowers a 3¾-inch O.D. core barrel to the hole bottom and attached to the casing. The core and casing rotate in unison as the rock core is cut. Rock samples slide into a steel liner loaded into an inner core barrel. Once the sample bottom depth is achieved, a wire line retrieves the core barrel and rock core sample. We placed rock samples in labeled, partitioned, cardboard boxes.

Laboratory Testing

We completed the following laboratory tests on selected soil and rock samples obtained from the exploratory borings.

- Water Content (ASTM D2216).
- Particle Size Analysis (ASTM D422 and ASTM D1140)
- Atterberg Limits (ASTM D4318)
- Unconfined Compressive Strength (ASTM D7012)

We graphically display the water content, fines content (particle sizes less than 0.075 millimeter), and Atterberg limits (plasticity) test results on the boring logs (Appendix A) and incorporated the results into the log soil descriptions. Laboratory test results are presented in Appendix B.

GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geologic Setting

The project site is in the eastern foothills of the Cascade Mountains in central Washington. The region has a complex geologic and tectonic history. During the Middle Eocene, central Washington experienced tensional stresses and vertical movement along faults, which opened several large scale, northwest by southeast trending depressions or grabens (Orr and Orr, 2002). As the Cascade Mountains formed, rivers and glaciers sculpted terraces, canyons, and valleys.

Local Geologic Setting

The project site lies within the 12-mile-wide Chiwaukum graben which developed during the Middle Eocene. The graben is bounded by the Entiat fault zone approximately 5 miles northeast and the Leavenworth fault zone approximately 7 miles southwest of the project site (Orr and Orr, 2002). This graben accumulated up to 18,000 feet of sandstone and conglomerate with interbedded shale known as the Chumstick Formation. The fluvial and lacustrine deposits of the Wenatchee Formation unconformably overly the Chumstick Formation (Tabor et al., 1987).

The 1:100,000 geologic map of the area (Tabor et al., 1987) indicates a northwest by southeast trending, unnamed normal fault is approximately ½-mile to the northeast of the project site. The map indicates Swakane Biotite Gneiss is exposed on the uplifted block to the northeast and Chumstick Formation is exposed on the downthrown block to the southwest toward the project site.

The south abutment lies on top of a river terrace sitting approximately 40-foot above the modern flood plain (Tabor et al., 1987). About ¼-mile north of the proposed abutment, Tabor et al. (1987) mapped an alluvial fan extending from Hay Canyon to the Wenatchee River. An orchard previously grew on the alluvial fan, north of Sunburst Lane. After the orchard trees were removed, the surface was graded.

Geologic Unit Descriptions and Distribution

This section describes the geologic units encountered in borings completed along the project alignment. The geologic unit descriptions and general distribution are described below and shown in the boring logs presented in Appendix A, and a generalized subsurface profile is shown in Figures 3.

The geologic units encountered in our explorations include fill (Hf) and alluvium (Qa) overlying Chumstick Formation (Tc). Unit descriptions were interpreted based on the soil and rock encountered in our borings and from the 1:24,000 (Whetten and Waitt, 1978) and 1:100,000 (Tabor et al., 1987) scale geologic maps. Unit descriptions and distribution are as follows:

- **Holocene Fill (Hf)** – At the north end of the site borings B-4, B-5, B-6, and B-7 encountered loose surface material that we interpret as fill placed during orchard and industrial development. Fill is composed of silty sand with gravel, and is likely locally-sourced Boring B-1 had a 1-foot pavement section on top the terrace deposits.
- **Alluvium (Qal)** – We collectively refer to deposits from an alluvial terrace along the south end of the project, an alluvial fan at the north end of the project, and Wenatchee River deposits as Qal. All three alluvial deposits are composed of medium dense to dense, *Poorly Graded Gravel with Silt, Sand, Cobbles, and Boulders.* The split spoon sampler used with hollow stem auger drilling techniques is 2 inches in diameter, therefore we infer the presence of cobbles and boulders based on drilling action. In addition, we observed boulders up to 5-feet in diameter along the river banks. All borings encountered Qal.
- **Chumstick Formation (Tc)** – Borings B-2, B-4, and B-5 encountered Tc at approximately 73.7 feet, 53 feet, and 63 feet bgs, respectively. The Tc encountered in the borings is predominantly medium strong to very strong, slightly weathered to fresh *SANDSTONE* with interbeds of very weak to medium strong, slightly to moderately weathered *SILTSTONE*. Sandstone layers have highly to completely weathered layers. The cores had smooth to rough, very close to moderately spaced, low to high angle joints. (neeed to check against other borings). Contacts between sandstone and siltstone are typically dipping at about 70 degrees.

Groundwater Conditions

During our April 2016 drilling, borings B-2, B-4, B-5, and B-6 encountered groundwater at 38.5, 11.5, 20.5, and 31.5-feet bgs, respectively. We did not encounter free groundwater within the maximum exploration depths at borings B-1 and B-7. Groundwater levels vary with the time of year at the site, and depend on the Wenatchee River levels, irrigation, and precipitation.

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PRELIMINARY ENGINEERING

Seismic Ground Motions

In accordance with the AASHTO LRFD BDS, the U.S. Geological Survey (USGS) indicates the peak horizontal ground acceleration (PGA) for rock (Site Class B) along the project alignment is 0.16g for an approximately 1,000-year (7 percent probability of exceedance in 75 years) return period. Short- and long-period spectral response accelerations (S_s and S_1) are 0.367 and 0.130, respectively.

The AASHTO LRFD BDS indicates PGA, short- and long-period spectral response accelerations are scaled by site amplification factors that are functions of the site subsurface conditions or site class. The subsurface conditions at the site are consistent with Site Class D (stiff soil profile) based on the 2014 AASHTO manual, using the weighted average SPT N-values. We present the corresponding Seismic Design Parameters in Table 4.

TABLE 4
AASHTO SEISMIC DESIGN PARAMETERS

Site Class	Site Factors, F_{pga} / F_a / F_v	Design Spectral Response Accelerations (g), A_s / S_{Ds} / S_{D1}
D	1.48 / 1.51 / 2.28	0.24 / 0.55 / 0.30

Notes:
Site Class B = rock; Site Class C = very dense soil and soft rock; Site Class D = stiff soil profile; Site Class E = soft clay soil; F_{pga} = site factor at zero-period; F_a = site factor at short-period range (0.2sec); F_v = site factor at long-period range (1.0sec); A_S = modified zero-period spectral acceleration; S_{Ds} = modified short-period spectral response acceleration; S_{D1} = modified long-period spectral response acceleration.
AASHTO = American Association of State Highway and Transportation Officials

Based on the AASHTO BDS, bridge design shall be in accordance with the requirements of Seismic Zone 2 ($0.15g < S_{D1} \leq 0.30g$).

Foundation Alternatives

Bridge structure foundation selection depends on several factors, including required resistances due to axial and lateral loading, total and differential settlement tolerances, and construction considerations. Shannon & Wilson evaluated shallow and deep foundations, including footings, driven H- and pipe piles, drilled shafts, and micropiles.

Shallow Foundations

Shallow foundations (i.e., spread footings) are typically the least expensive foundation option and generally considered favorable if excavations can be completed in the dry and dense/hard bearing conditions are present.

At Bent 1 (south abutment), shallow foundations may be considered favorable given the relatively shallow, dense/hard soil conditions, provided the existing slope is capable of supporting the added load near the top. Shallow foundations may also be favorable at Bent 4 (north abutment) provided potential settlement criteria may be met. Spread footing construction at Bent 4 may require a deep overexcavation and potential replacement, potentially requiring a shored excavation.

At proposed Bent 2, within the Wenatchee River, we anticipate a shallow foundation is likely feasible, although construction conditions will be more difficult than other alternatives, and project permitting may pose additional challenges. A shallow foundation at Bent 2 will require shoring and a cofferdam to construct the footing in the dry. Sheet piles may not be practical for cofferdam construction because of difficulty penetrating the anticipated very dense, coarse subsurface materials, likely including cobbles and potential boulders (if conditions within the river are similar to borings B-1, B-2, and B-4). The footing(s) may require anchors to mitigate overturning forces due to flood conditions.

At proposed Bent 3, somewhat less competent (loose to medium dense) soils extend to approximately 18 feet bgs. Shallow foundation construction at this location will require a somewhat deep excavation to limit potential elastic settlements. The deep excavation will require careful planning given the proximity to SR 2 and existing bridge traffic. The excavation will likely require shoring.

Deep Foundations

Deep foundations develop axial resistance through frictional interaction between the pile surface and the soil profile, and end-bearing resistance at the pile tip.

Driven Piles

Driven pile installations are typically performed by impacting or vibrating the pile into the ground to the prescribed bearing stratum. Pile driving vibrations and the associated damage risk should be evaluated for nearby residences, structures, businesses, etc. as warranted. Driven piles typically permit the cleanest installation and foundation construction.

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We anticipate driven H-piles with tip protection (i.e., driving shoe) are a viable foundation alternative at each bent location permitted the associated vibration risk is permissible. The selected H-pile section should be robust to mitigate the dense driving conditions. Due to the difficult driving conditions, we estimate driven H-piles should be preferred over pipe piles.

Drilled Shafts

Drilled shafts are slightly to moderately more costly than driven piles due to the equipment mobilization costs and casing requirements. Drilled shafts are a viable foundation alternative to provide adequate depth for lateral resistance considerations while protecting against potential scour provided adequate construction considerations for the dense, coarse subsurface materials with cobbles and potential boulders. Casing installation will likely require an air-rotary and/or oscillator drilling rig due to the coarse, dense/hard subsurface conditions. At Bent 2, we anticipate the casing may be left in place to separate the construction from the active river channel. Alternatively, the concrete may stop short of the ground surface to allow casing removal and cut to desired elevation after excavation for column construction.

Micropiles

Micropiles are small-diameter (typically 3 to 10 inches) deep foundation elements constructed using high-strength steel casing and/or threaded bar capable of achieving relatively high capacities (in excess of 300 tons depending on micropile size and subsurface conditions). Micropile construction is feasible in very dense, coarse materials (i.e., cobbles and boulders) to bedrock using available drilling methods. The casing may extend the full depth, or terminate above the bond zone with the reinforcing element extending the full depth. The reinforcing steel (typically an all-thread bar) is inserted through the micropile casing, and then high-strength cement grout pumped into the casing. Micropiles resist compressive, uplift/tension, and lateral loads, and are typically load tested in accordance with ASTM D 1143 (compressive), ASTM D 3689 (uplift/tension) and ASTM D 3966 (lateral). Micropile construction is typically cost comparable to drilled shafts but completed with relatively small drill rigs allowing construction within restricted access and/or low headroom areas. Micropiles should be considered given the dense, coarse subsurface materials.

Preliminary Foundations

Based on the encountered subsurface conditions, project discussions, proposed layouts, and the existing bridge structure estimated foundations, we preliminarily estimate driven H-piles and drilled shafts are viable foundation alternatives. The drivability of driven piles and constructability of drilled shafts should be further evaluated for the bridge final design. Specifically, we recommend sonic rotary drilling be completed at selected foundation locations. We provide further discription within the Recommended Additional Geotechnical Explorations section below.

We understand the design team will preliminarily consider driven H-piles at the abutments and drilled shafts at all bent locations. We estimate shallow footings may be considered at the south abutment in conjunction with a slope stability review above the railroad and Wenatchee River.

In our pile analyses, we assume all required fill will be placed prior to pile driving and/or shaft construction. Fill placement following pile installation which induces settlement greater than 0.4-inch may induce downdrag loads over the entire pile length of compressible soils. Fill placement and construction sequencing should be considered for final foundation design.

In the following sections, we provide preliminary axial capacities for driven H-piles and drilled shafts, as described above, estimated settlement, and lateral LPILE® parameters. For the purpose of preliminary foundation recommendations, we focus on the three-span alternative with a separate drilled shaft foundation recommendation for the two-span (cable-stayed) alternative.

Driven H-Piles

We evaluated 16x162 steel H-pile axial resistance for a single pile at the abutments. The pile wall thickness selection is typically based on a preliminary drivability evaluation performed using GRLWEAP, or other similar programs. We did not perform this evaluation; we selected the typical pile section based on the encountered subsurface conditions, anticipated hard driving conditions, and the preliminary design loads.

Driven pile design and construction should meet the requirements of WSDOT Standard Specifications, Section 6-05. The H-pile tips should be reinforced with prefabricated cast steel points to protect the pile structural integrity from the hard driving conditions.

The total nominal axial pile resistance is the sum of the frictional and base resistance, with frictional resistance cumulating with embedment depth and base resistance determined considering the subsurface conditions for about two pile diameters below the pile tip.

We used an in-house spreadsheet to perform axial pile resistance analyses in accordance with AASHTO LRFD BDS (2014) guidelines based on the encountered subsurface conditions and experience with similar soil and project conditions. The AASHTO LRFD BDS recommends pile group efficiency (reduction) factors equal to 1.0 for pile groups with a minimum 2.5 pile diameter center-to-center spacing. We performed our axial resistance analyses for strength and extreme event limit states for a single pile or pile groups with a minimum 2.5 pile diameter center-to-center spacing.

We present the preliminary axial resistance plot for driven 16x162 steel H-piles at the abutments in Figures 4 (Bent 1) and 5 (Bent 4). We assume the piles will be installed after settlement, if any, from new approach fills, if required. Actual pile penetrations should be determined during construction with pile dynamic testing using a pile-driving analyzer and observations of pile driving resistance.

The plots present unfactored (nominal) side and tip resistance for Strength and Extreme Limit states. Factored total compressive resistance for the Strength Limit state is shown using typical AASHTO (2014) resistance factors, as listed below the plot. The figures include generalized subsurface conditions along the left side, as encountered in the respective boring. Differing subsurface conditions are possible at specific pile locations.

Drilled Shafts

For the three-span layout alternative, we understand 4-foot-diameter drilled shafts with a pile cap are being considered at the abutments and 8-foot-diameter shafts with no pile cap at the interior piers. We understand 10-foot-diameter shafts are being considered for the two-span, cable-stayed layout alternative center pier. Shannon & Wilson completed preliminary axial capacity analyses of the proposed drilled shafts.

Drilled shafts are constructed by excavating a cylindrical bore to the prescribed bearing stratum using any number of drilling, tool, and excavation support methods. Shaft performance is sensitive to construction techniques; selection of drilling procedure, tooling, and excavation support method should consider impacts on the integrity of the bearing materials and the shaft structural integrity. The overall shaft construction quality will have a significant impact on shaft performance.

Our engineering evaluation assumes all shafts are constructed using temporary casing socketed into dense to very dense coarse gravels with cobbles and potential boulders.

We evaluated the axial resistance of 4-foot-diameter drilled shafts at the abutments and 8-foot-diameter shafts at the interior piers of the three-span layout alternative, and 10-foot-diameter shafts for the cable-stayed layout alternative center pier. We performed axial resistance analyses in general accordance with AASHTO LRFD, Section 10.8. We evaluated the nominal axial resistance and recommend appropriate resistance factors for the strength and extreme event limit states. We estimated unit side and base resistance values based on the average SPT N-values within soil and weathered rock units, laboratory test results, unconfined compressive strength and point load index test results of rock, and our experience. We assume the temporary casing will be removed after shaft installation. If the casing is left in place, we should be notified so that we can re-evaluate the unit side resistance values assumed in design.

The total nominal axial resistance is a summation of nominal side resistance and nominal end bearing. The nominal and factored axial resistance versus depth for strength and extreme event limit states are provided in Figures 6 through 8 for Bents 1, Bent 3, and Bent 4, respectively. We also provide Figure 9 for a potential 10-foot-diameter drilled shaft at Bent 2 (center pier) of the cable-stayed layout alternative. Recommended resistance factors for each limit state are provided on each figure in the notes below the plot.

The estimated nominal axial resistance assumes that the piles are spaced at least four times the shaft diameter, measured center to center. Based on the assumption, no group action was considered.

The pile geotechnical resistances under strength, service, and extreme event limit state should not be greater than the pile structural resistance. The structural engineer should evaluate the pile structural resistance in accordance with AASHTO LRFD, Section 6.5.4.

Estimated Settlement

Assuming pile design and installation in accordance with our recommendations contained herein, we estimate total settlements will be less than about 1-inch, with differential settlement across the individual bent approximately one-half of estimated total settlement. The foundation soils at the site generally consist of granular, cohesionless (non-plastic) fine to coarse silty sand to coarse gravels with cobbles and potential boulders. We anticipate settlement within cohesionless, granular soils will be elastic, or generally occur as the load is applied during construction.

The AASHTO LRFD BDS indicates a minimum 0.4-inch of ground settlement around driven piles is typically required to induce downdrag loading conditions following installation. We expect less than 0.4-inch compression of the foundation soils under static and live loading conditions if piles are driven following fill placement. However, this assumption may be revised for final design as the design team considers construction sequencing.

Lateral Pile Resistance

The pile foundations will be subjected to lateral loads resulting from live loads, wind, and potential flood and earthquake loading. We understand that the laterally loaded pile analyses will be performed with the aid of the LPILE® Plus 5.0 computer program developed by Ensoft, Inc. Geotechnical input parameters for the LPILE® computer program are provided in Tables 5 through 7 for the three-span layout alternative Bents 1 through 4, respectively, excluding Bent 2.

TABLE 5
LPILE® GEOTECHNICAL INPUT
PARAMETERS FOR DRILLED SHAFT FOUNDATIONS – BENT 1

Exploration	Elevation (feet msl)		Soil Type (p-y Curve)	Effective Unit Weight[1] (pci)[2]	Friction Angle (degrees)	Modulus (pci)[2]	Unconfined Compressive Strength (psi)[2]
	Layer Top	Layer Bottom					
B-1 & B-2	836.5	826.5	Sand (Reese)	0.075	42	175	--
	826.5	798	Sand (Reese)	0.078	45	225	
	798	762.8	Sand (Reese)	0.042	43	225	--
	762.8	743	Strong Rock	0.087	--	--	>2,000

Notes:
1 Effective unit weight = Total unit weight – Unit weight of water (62.4 pounds per cubic foot = 0.036 pci)
2 pci = pounds per cubic inch and psi = pounds per square inch.

TABLE 6
LPILE® GEOTECHNICAL INPUT
PARAMETERS FOR DRILLED SHAFT FOUNDATIONS – BENT 3

Exploration	Elevation (feet msl)		Soil Type (p-y Curve)	Effective Unit Weight[1] (pci)[2]	Friction Angle (degrees)	Modulus (pci)[2]	Unconfined Compressive Strength (psi)[2]
	Layer Top	Layer Bottom					
B-4	809.6	800.6	Sand (Reese)	0.067	31	65	--
	800.6	797.1	Sand (Reese)	0.025	26	5	
	797.1	789.6	Sand (Reese)	0.039	40	95	--
	789.6	779.6	Sand (Reese)	0.033	33	60	
	779.6	756.6	Sand (Reese)	0.042	43	125	
	756.6	729.6	Strong Rock	0.087	--	--	>2,000

Notes:
[1] Effective unit weight = Total unit weight – Unit weight of water (62.4 pounds per cubic foot = 0.036 pci)
[2] pci = pounds per cubic inch and psi = pounds per square inch.

TABLE 7
LPILE® GEOTECHNICAL INPUT
PARAMETERS FOR DRILLED SHAFT FOUNDATIONS – BENT 4

Exploration	Elevation (feet msl)		Soil Type (p-y Curve)	Effective Unit Weight[1] (pci)[2]	Friction Angle (degrees)	Modulus (pci)[2]	Unconfined Compressive Strength (psi)[2]
	Layer Top	Layer Bottom					
B-5	820.3	813	Sand (Reese)	0.064	29	20	--
	813	803	Sand (Reese)	0.068	33	85	
	803	760.3	Sand (Reese)	0.042	43	125	
	760.3	730.3	Strong Rock	0.087	--	--	>2,000

Notes:
[1] Effective unit weight = Total unit weight – Unit weight of water (62.4 pounds per cubic foot = 0.036 pci)
[2] pci = pounds per cubic inch and psi = pounds per square inch.

Temporary Work Bridge

We preliminarily estimate driven H- and relatively small-diameter pipe piles are a feasible foundation type for a potential work bridge, although driving may require many blows to seat the foundations to the necessary depths to provide the required lateral resistance. At this time, we recommend additional sonic rotary drilling be completed at proposed bent locations. The sonic drilling will allow further drivability assessment of driven piles.

RECOMMENDED ADDITIONAL GEOTECHNICAL EXPLORATIONS

This letter report presents our preliminary assessment of foundation alternatives for the West Cashmere (Wenatchee River) Bridge based on the boring explorations described above. The borings encountered considerable oversized materials, including coarse gravels, cobbles, and potential boulders, as interpreted primarily from the drill rig action and occasionally from recovered HQ3 coring samples. The HQ3 sample sizes are limited by the core barrel ID (approximately 2.4 inches). Therefore, Shannon & Wilson recommends the following explorations be completed for final design of the replacement bridge structure.

- One boring at Pier 2 (of the four pier option, if utilized) through the existing bridge deck for foundation design and construction recommendations.
- One boring within the south river bank slope (near railroad right-of-way) to review global slope stability. Shannon & Wilson provides this recommendation considering the observed raveling of the existing slope materials and placement of future foundations within (walls) and above.
- Sonic rotary explorations should be considered at bent locations to better characterize the coarse gravels, cobbles, and boulders for foundation design and construction considerations. Characterization of the coarse gravel, cobbles, and potential boulders is important to consider for driven pile design (i.e., axial and lateral capacities) and to assess the potential for piles to encounter premature refusal (i.e., termination prior to the design tip elevation). For drilled shafts, characterizing the oversize material is important to assess drilling methods and equipment. Larger pieces may require different auger buckets and/or coring equipment. Larger pieces may also require specialized drill rigs with oscillator and/or rotator casing installation capabilities.

LIMITATIONS

The analyses, conclusions, and recommendations presented in this report are based on the site conditions as observed during our reconnaissance and explorations. We assume that the soil and rock conditions observed in the explorations are representative of the subsurface conditions in all areas of the site; i.e., the subsurface conditions everywhere are not significantly different from those observed in the borings. If, during construction or additional explorations, subsurface conditions different from those described in our letter report are observed or appear to be present, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and the start of construction, or if conditions have changed due to natural events or construction operations at or near the site, we recommend that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

This report was prepared for the exclusive use of the TranTech design team and the County in the preliminary design of the West Cashmere (Wenatchee River) Bridge Replacement. It should be made available to prospective contractors and/or the Contractor for information on factual data only and not as a warranty of subsurface conditions. Unanticipated conditions are commonly encountered and cannot be fully determined by reconnaissance and subsurface explorations. Such unexpected conditions frequently require that additional expenditures be made to achieve a properly constructed project. Some contingency fund is recommended to accommodate such potential extra costs.

We recommend we be retained to provide the final design recommendations and review the geotechnical-related portions of the plans and specifications to evaluate if they are in accordance with our recommendations.

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To assist you and others in understanding the use and limitations of our report, Shannon & Wilson, Inc. has prepared Appendix C, "Important Information About Your Geotechnical Report." Please review the attached information and contact us at your convenience to discuss the foundation options.

We appreciate the opportunity to assist TranTech on this project and look forward to further foundation discussions. Please contact me if you have questions, comments, or concerns.

SHANNON & WILSON, INC.

DRAFT

Clinton A. Wilson, P.E.
Senior Geotechnical Engineer

JWT:JKP:CAW:WJP/caw

Enclosures: Figure 1 – Vicinity Map
Figure 2 – Site and Exploration Plan
Figure 3 – Generalized Subsurface Profile A-A'
Figure 4 – Estimated Axial Pile Resistance: HP16 x 162, Bent 1
Figure 5 – Estimated Axial Pile Resistance: HP16 x 162, Bent 4
Figure 6 – Estimated Axial Pile Resistance: 4-Foot-Diameter Drilled Shaft, Bent 1
Figure 7 – Estimated Axial Pile Resistance: 8-Foot-Diameter Drilled Shaft, Bent 3
Figure 8 – Estimated Axial Pile Resistance: 10-Foot-Diameter Drilled Shaft, Bent 2 (Cable-Stayed)
Figure 9 – Estimated Axial Pile Resistance: 4-Foot-Diameter Drilled Shaft, Bent 4
Appendix A – Exploratory Boring Logs
Appendix B – Laboratory Test Results
Appendix C – Important Information about Your Geotechnical/Environmental Report

N

0 2,000 4,000

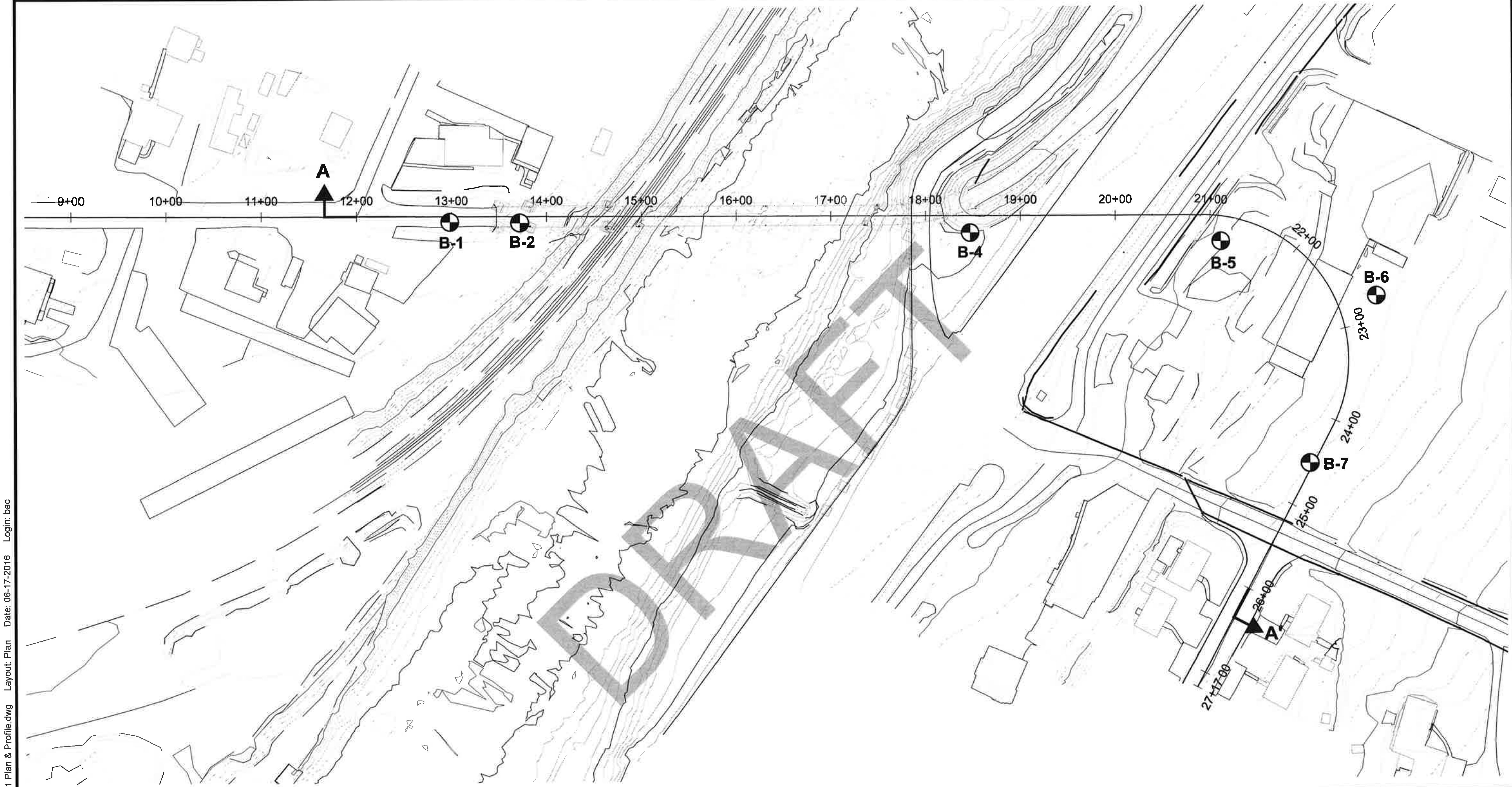
Approximate Scale in Feet

NOTE

Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth ™ Mapping Service.

CRP693 - West Cashmere Bridge (Wenatchee River) Replacement Chelan County, Washington	
VICINITY MAP	
July 2016	22-1-03144-001
SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS	**FIG. 1**

Filename: J:\221\03144\001\22-1-03144-001 Vicinity Map.dwg Date: 06-21-2016 Login: bac



LEGEND

B-1 Boring Designation and Approximate Location

A Generalized Subsurface Profile

N

0 100 200

Scale in Feet

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

SITE AND EXPLORATION PLAN

July 2016 22-1-03144-001

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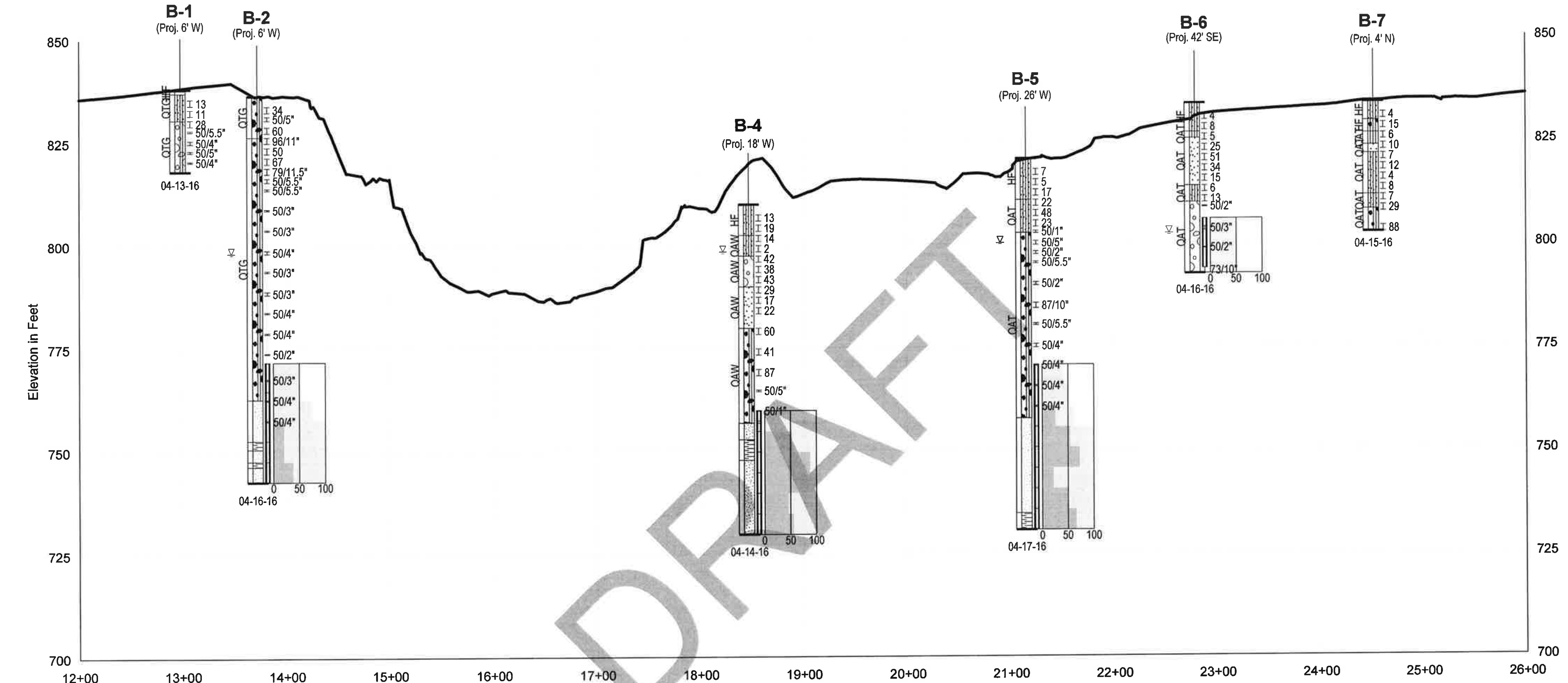
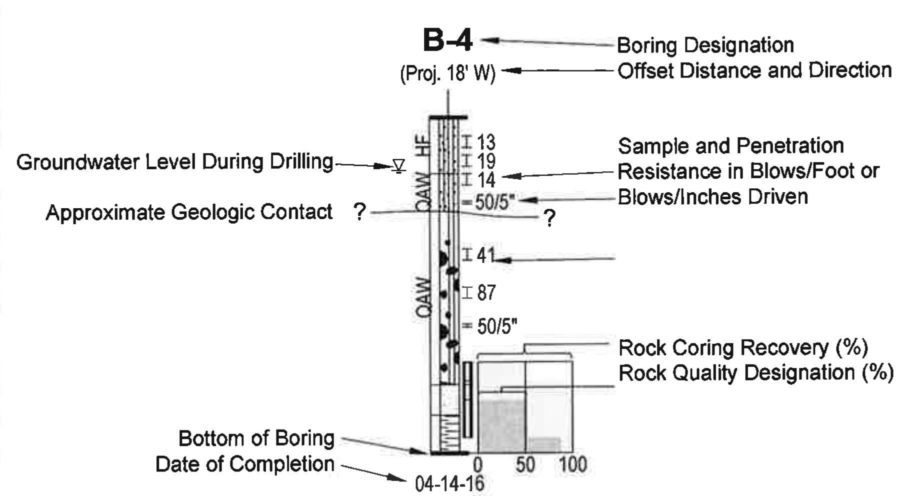
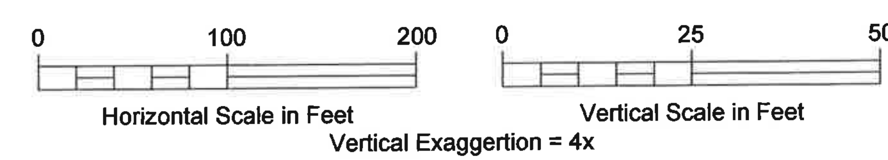
FIG. 2

Filename: J:\221\03144\001\22-1-03144-001 Plan & Profile.dwg Layout: Plan Date: 06-17-2016 Login: bac







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Chelan County, Washington

GENERALIZED SUBSURFACE PROFILE A-A'

July 2016 | 22-1-03144-001

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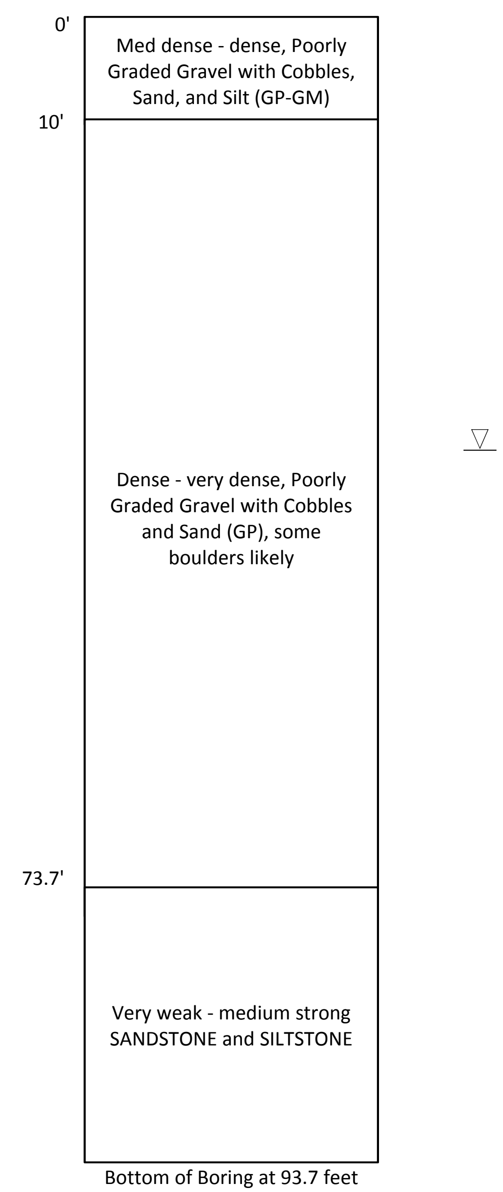
FIG. 3

Filename: J:\221\03144\001\22-1-03144-001 Plan & Profile.dwg Layout: Profile Date: 06-21-2016 Login: bac

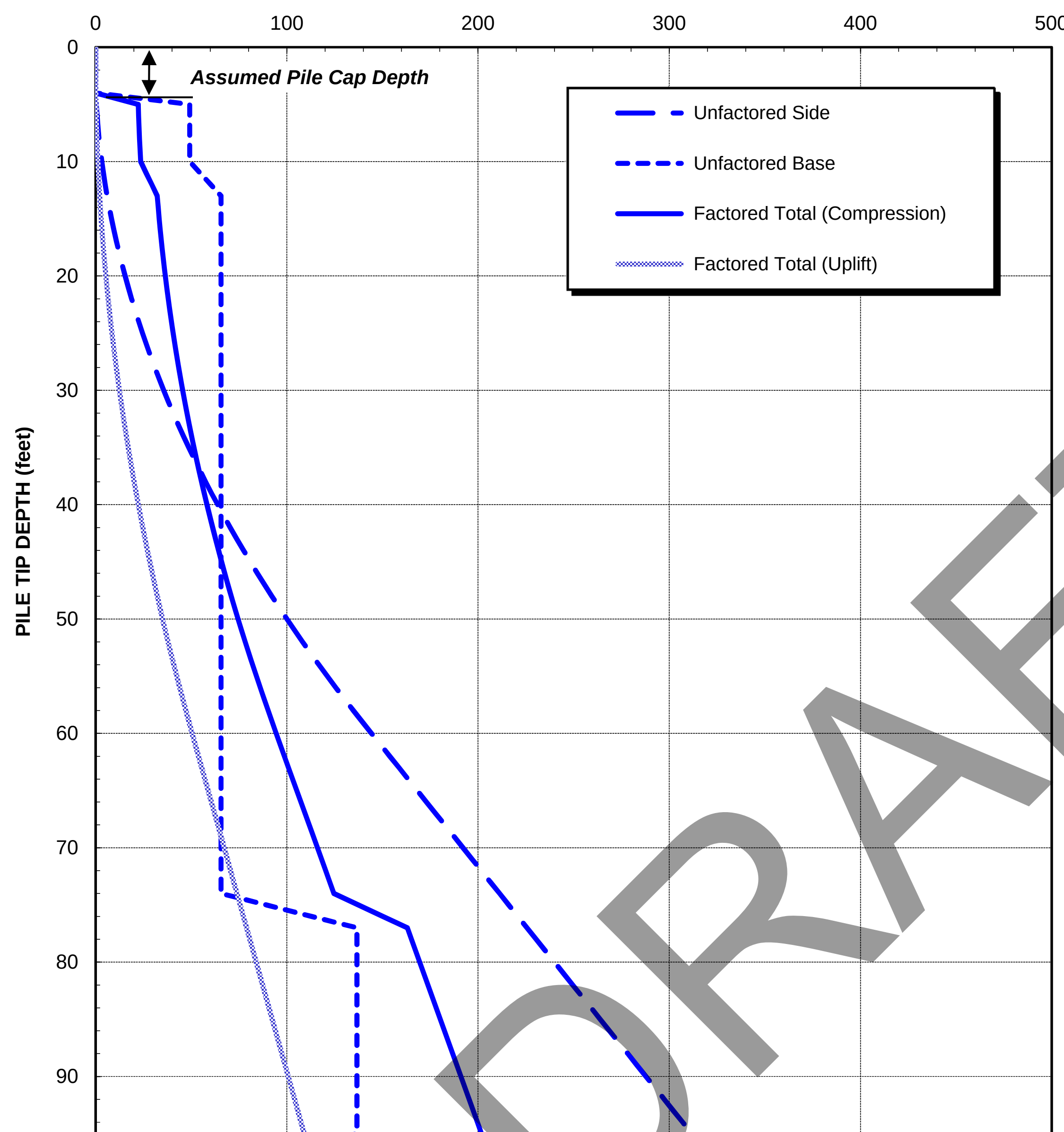
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-1 & B-2



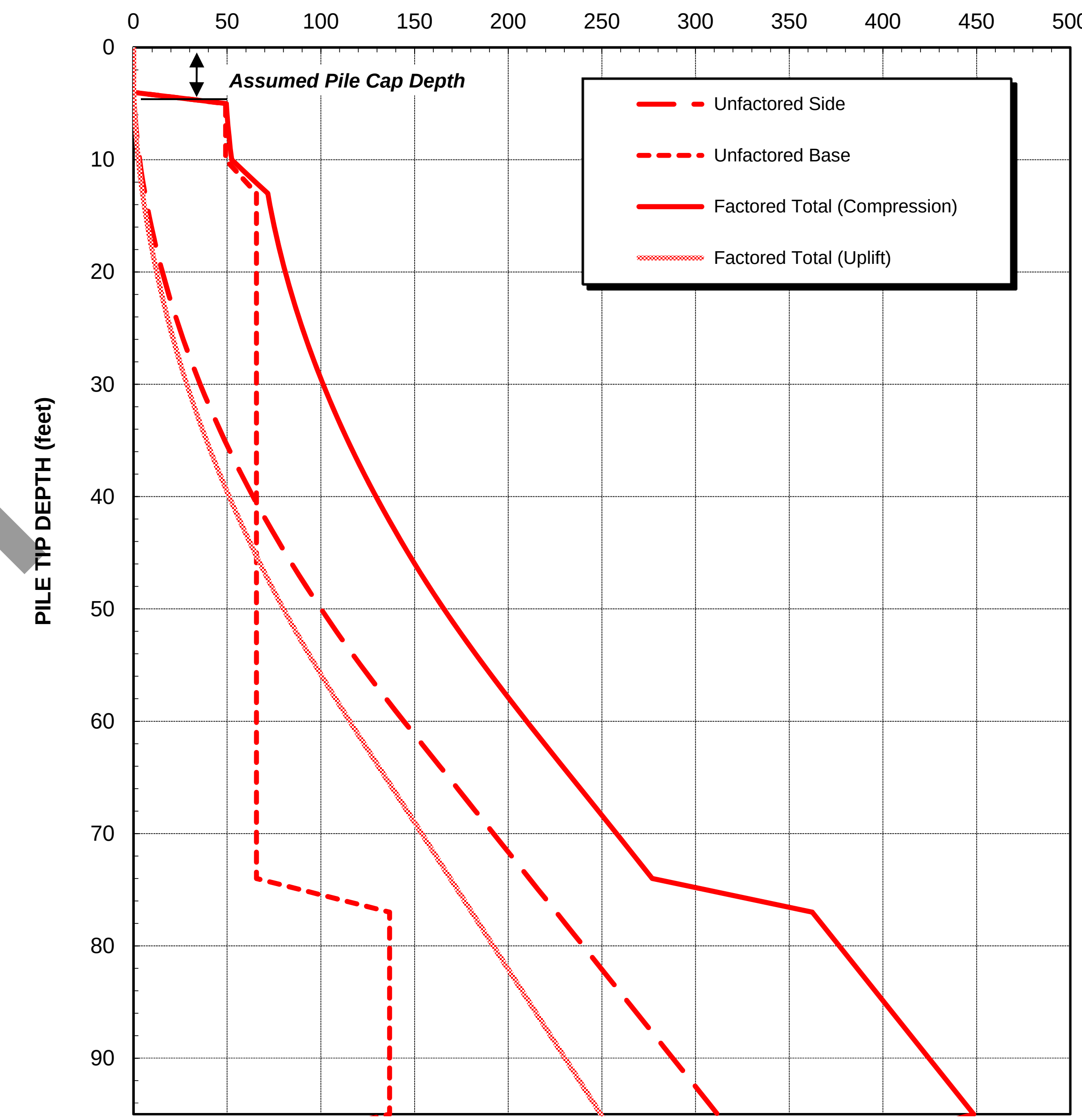
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



STRENGTH LIMIT NOTES:

1. Recommended resistance factors are 0.45 and 0.45 for side and base resistance, respectively. See general note 3 below.
2. Pile uplift capacity can be estimated by using the unfactored side resistance shown above and a recommended resistance factor of 0.35 .

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors are 0.45 and 0.45 for side and base resistance, respectively.
2. Unfactored downdrag force is estimated to be 0 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. Factored total pile resistance shown on plots is determined by adding its unfactored side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Recommended resistance factors for the strength limit state are intended to be used with the Shannon & Wilson pile design method. These resistance factors are based upon substantial successful application of the Shannon & Wilson pile design method in the Pacific Northwest. They are not calibrated to a specific reliability index.
4. Estimated capacities assume that the driven piles will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations.

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ESTIMATED AXIAL PILE RESISTANCE
HP16 x 162
BENT 1

July 2016 | 22-1-03144-001

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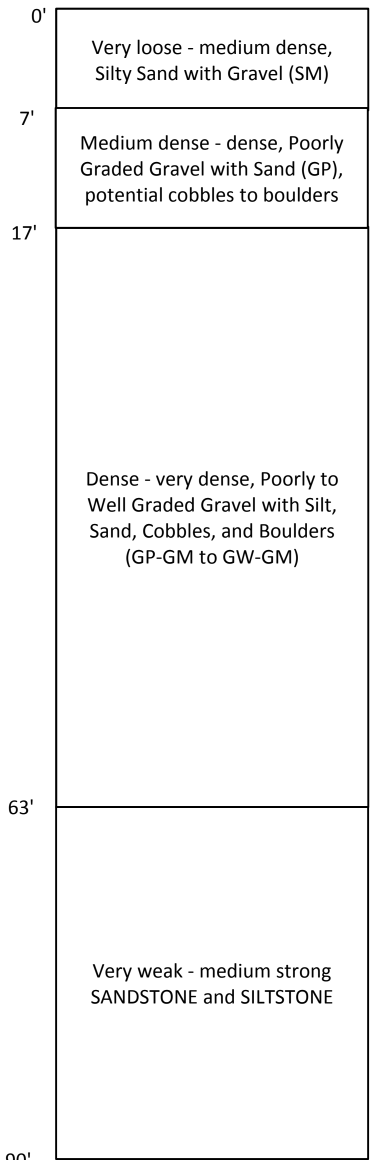
FIG. 4

ASSUMED SUBSURFACE PROFILE

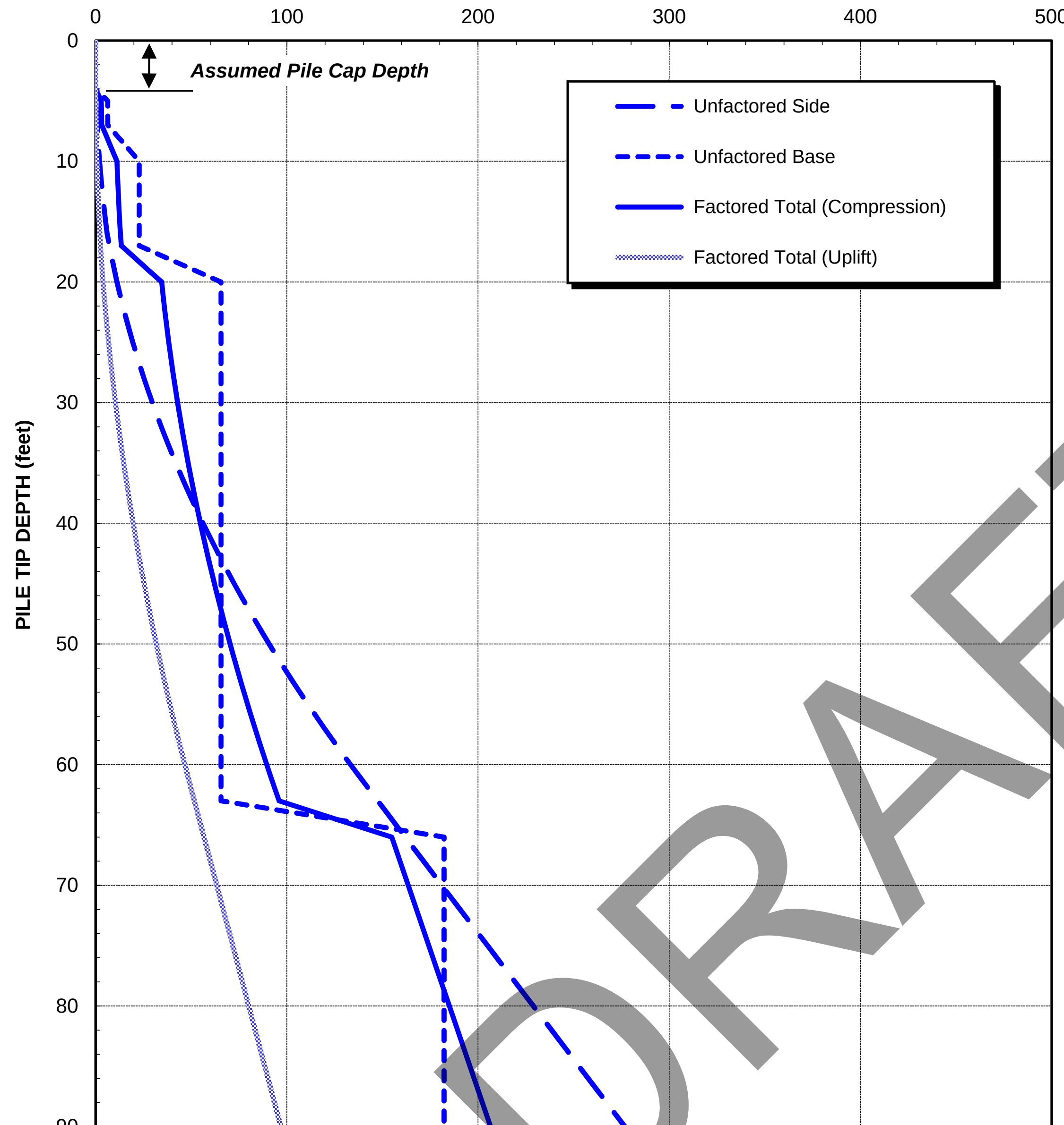
Based on Nearby Explorations:
B-5



Bottom of Boring at 90.0 feet.

STRENGTH LIMIT

NOMINAL RESISTANCE (tons)

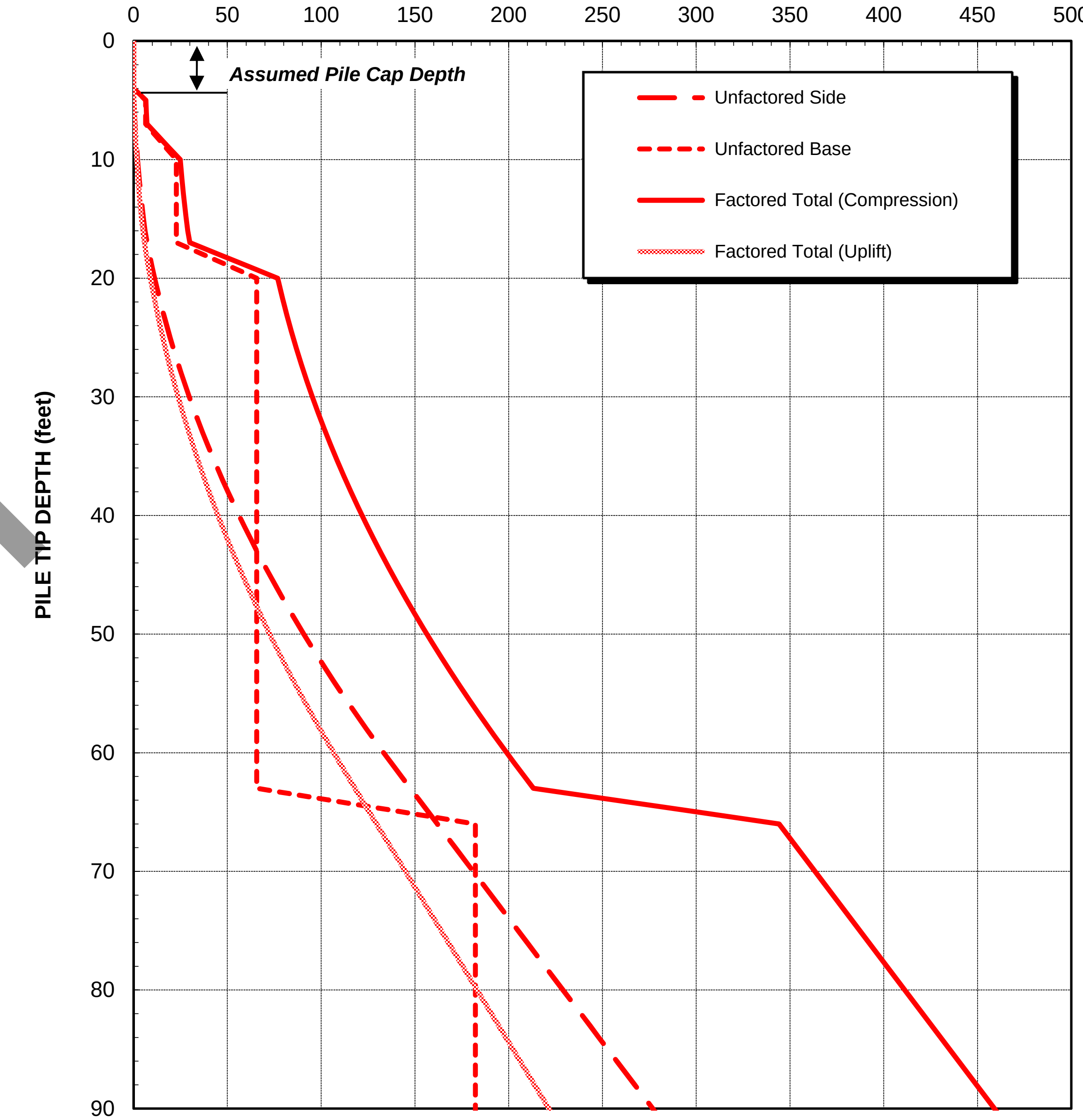


STRENGTH LIMIT NOTES:

1. Recommended resistance factors are 0.45 and 0.45 for side and base resistance, respectively. See general note 3 below.
2. Pile uplift capacity can be estimated by using the unfactored side resistance shown above and a recommended resistance factor of 0.35 .

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors are 0.45 and 0.45 for side and base resistance, respectively.
2. Unfactored downdrag force is estimated to be 0 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. Factored total pile resistance shown on plots is determined by adding its unfactored side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Recommended resistance factors for the strength limit state are intended to be used with the Shannon & Wilson pile design method. These resistance factors are based upon substantial successful application of the Shannon & Wilson pile design method in the Pacific Northwest. They are not calibrated to a specific reliability index.
4. Estimated capacities assume that the driven piles will be installed after construction of the approach embankments. Downdrag loads due to potential fill embankment settlement have not been included.
5. Per the WSDOT GDM, potential liquefaction below a depth of 80 feet was not considered in the calculations.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

ESTIMATED AXIAL PILE RESISTANCE
HP16 x 162
BENT 4

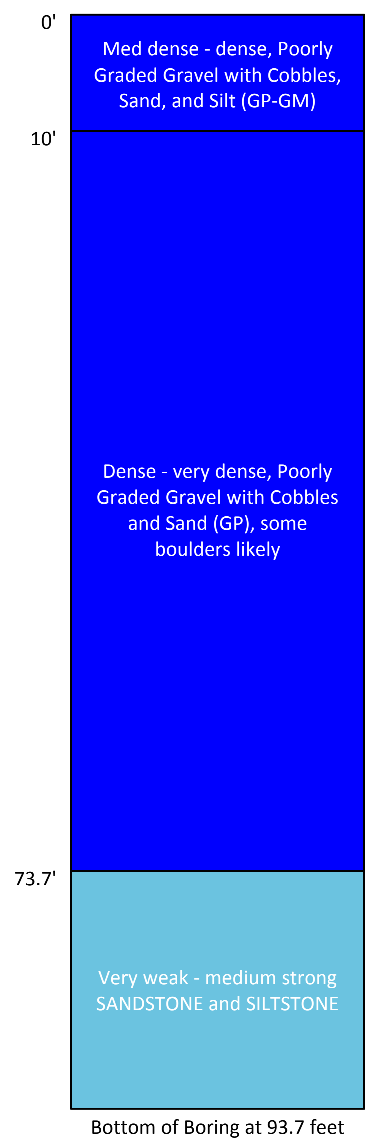
July 2016 | 22-1-03144-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 5

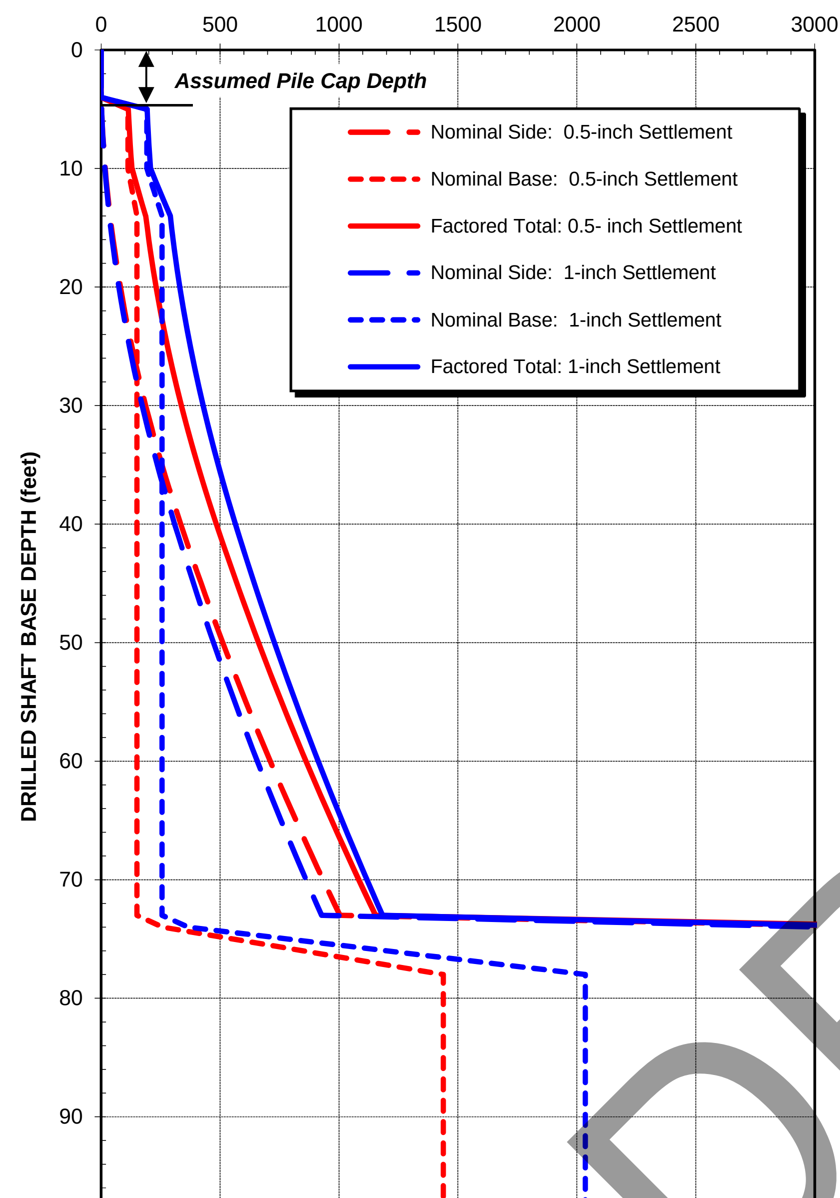
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-1 & B-2



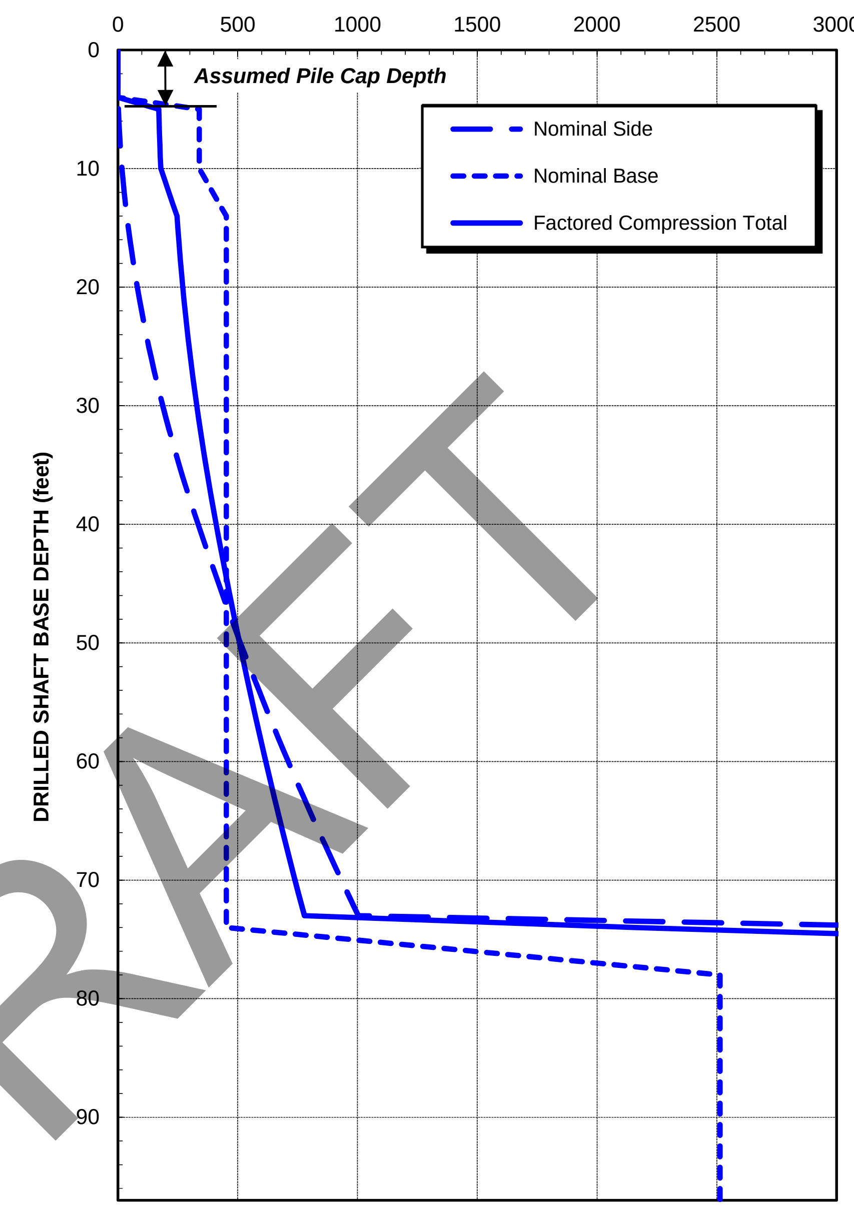
SERVICE LIMIT

NOMINAL RESISTANCE (tons)



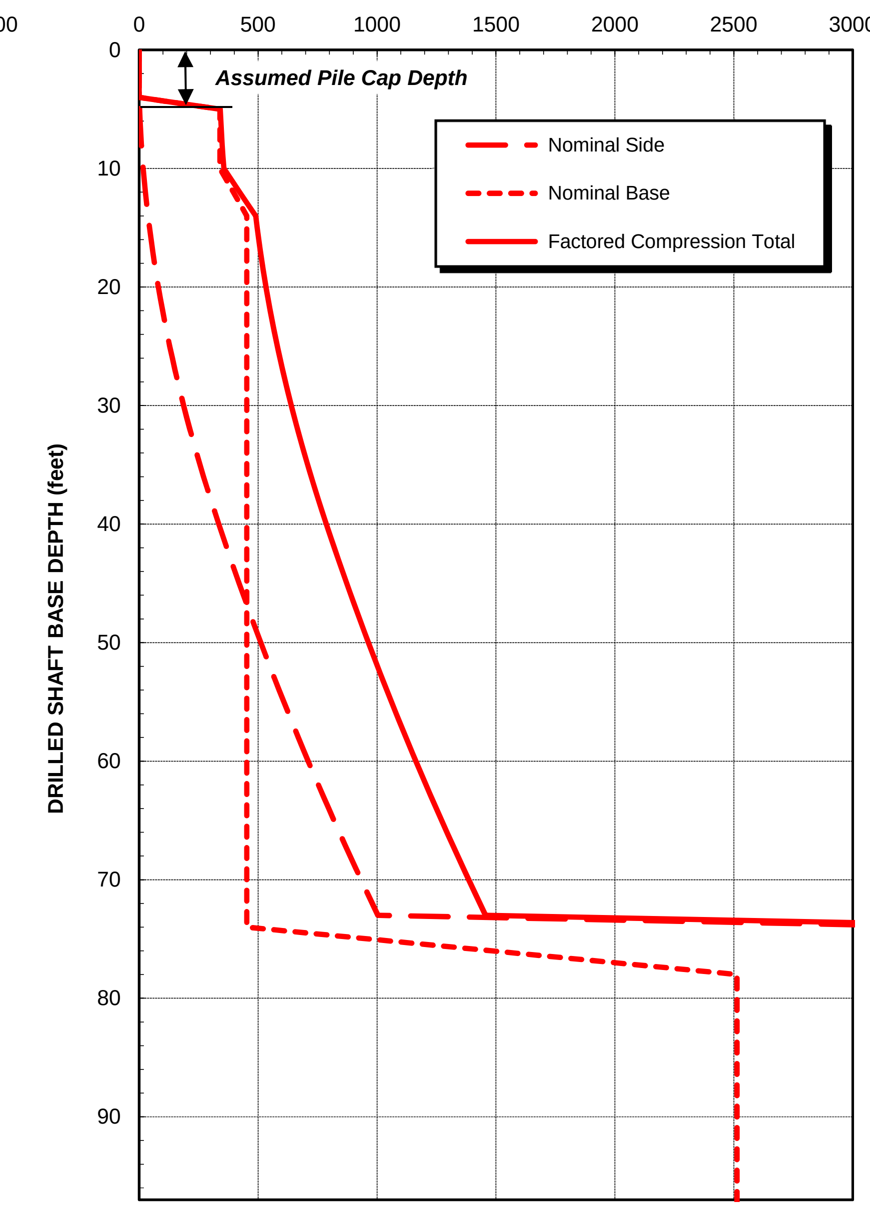
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



SERVICE LIMIT NOTES:

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

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per WSDOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 0 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

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

CRP693 - West Cashmere Bridge
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Chelan County, Washington

ESTIMATED AXIAL SHAFT RESISTANCE
4-FOOT-DIAM. DRILLED SHAFT
BENT 1

July 2016 | 22-1-03144-001

SHANNON & WILSON, INC.
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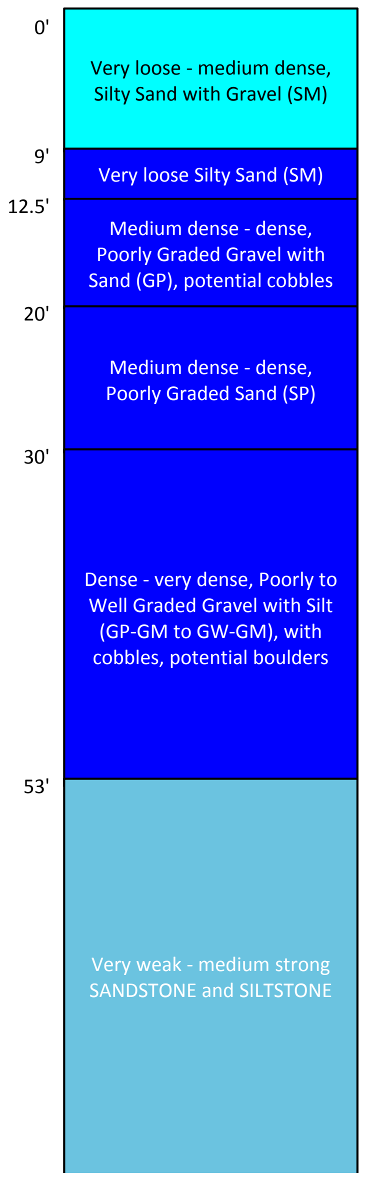
FIG. 6

ASSUMED SUBSURFACE PROFILE

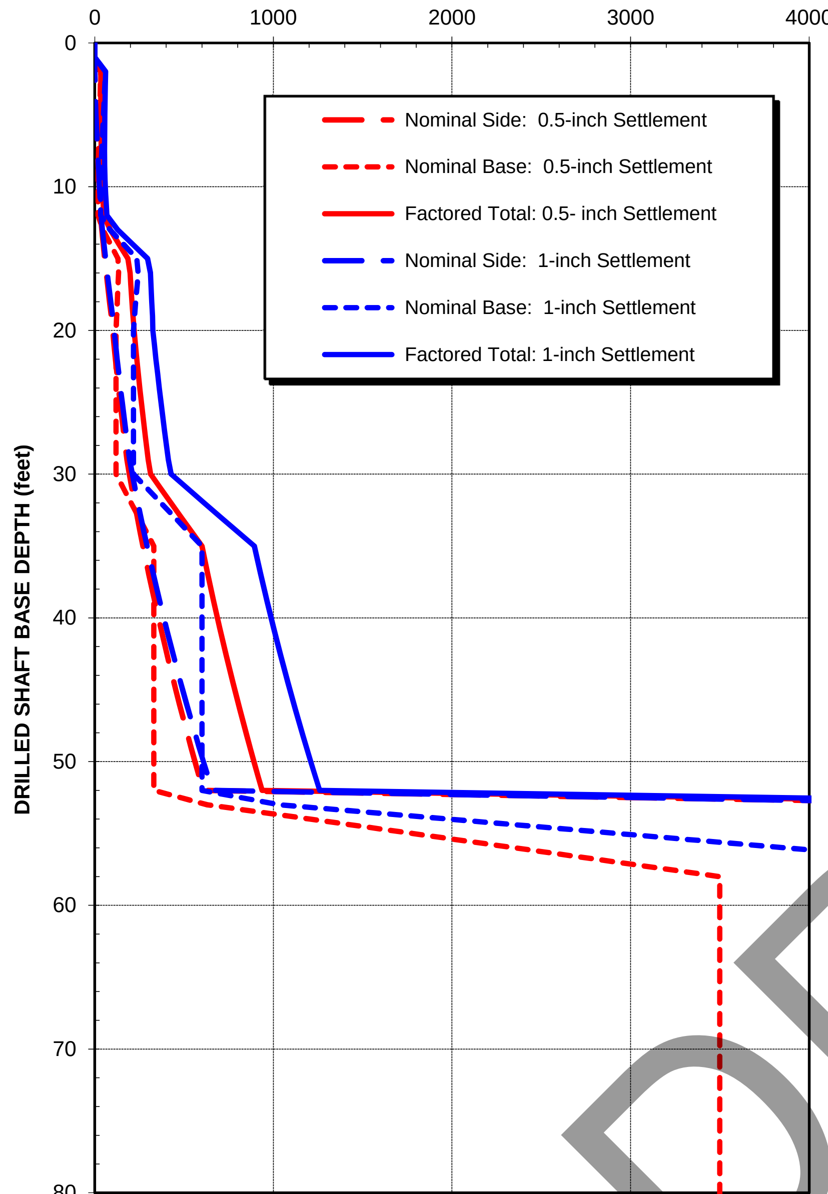
Based on Nearby Explorations: ***B-4***



Bottom of Boring at 80.0 feet.

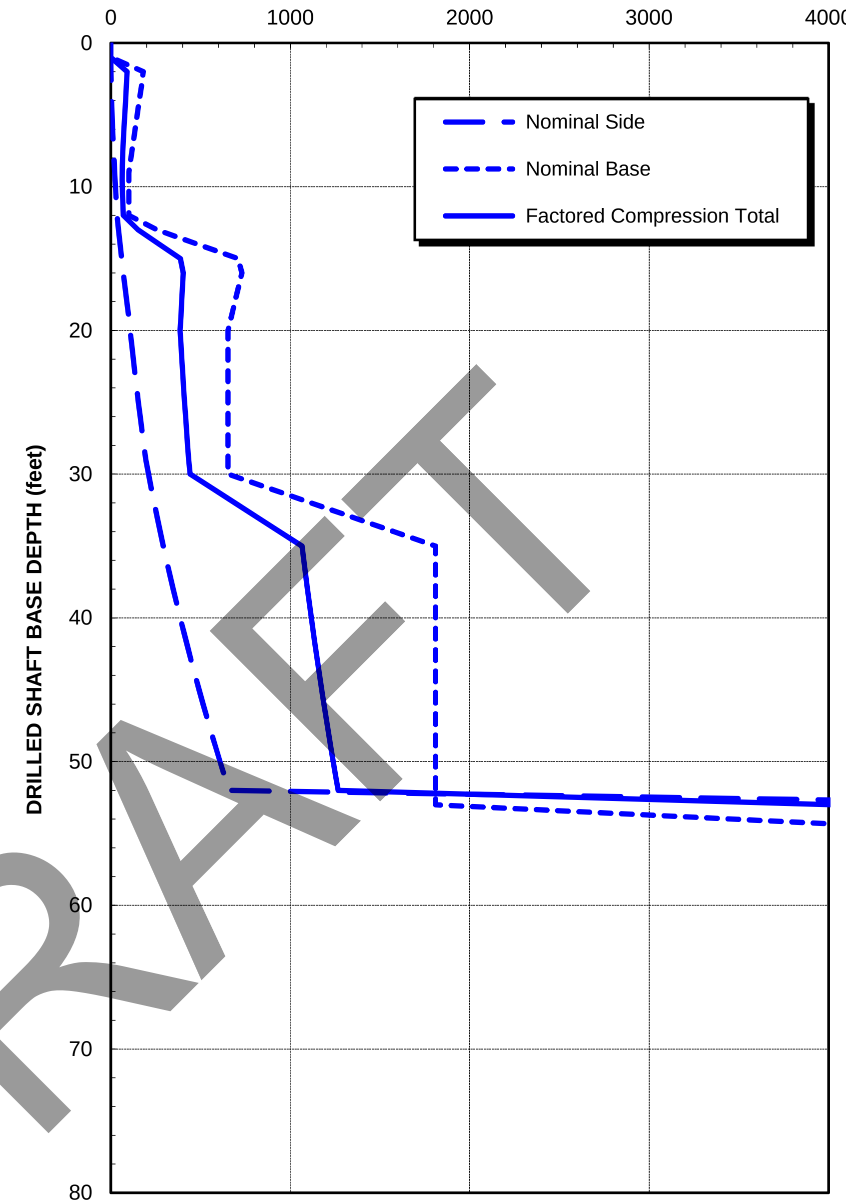
SERVICE LIMIT

NOMINAL RESISTANCE (tons)



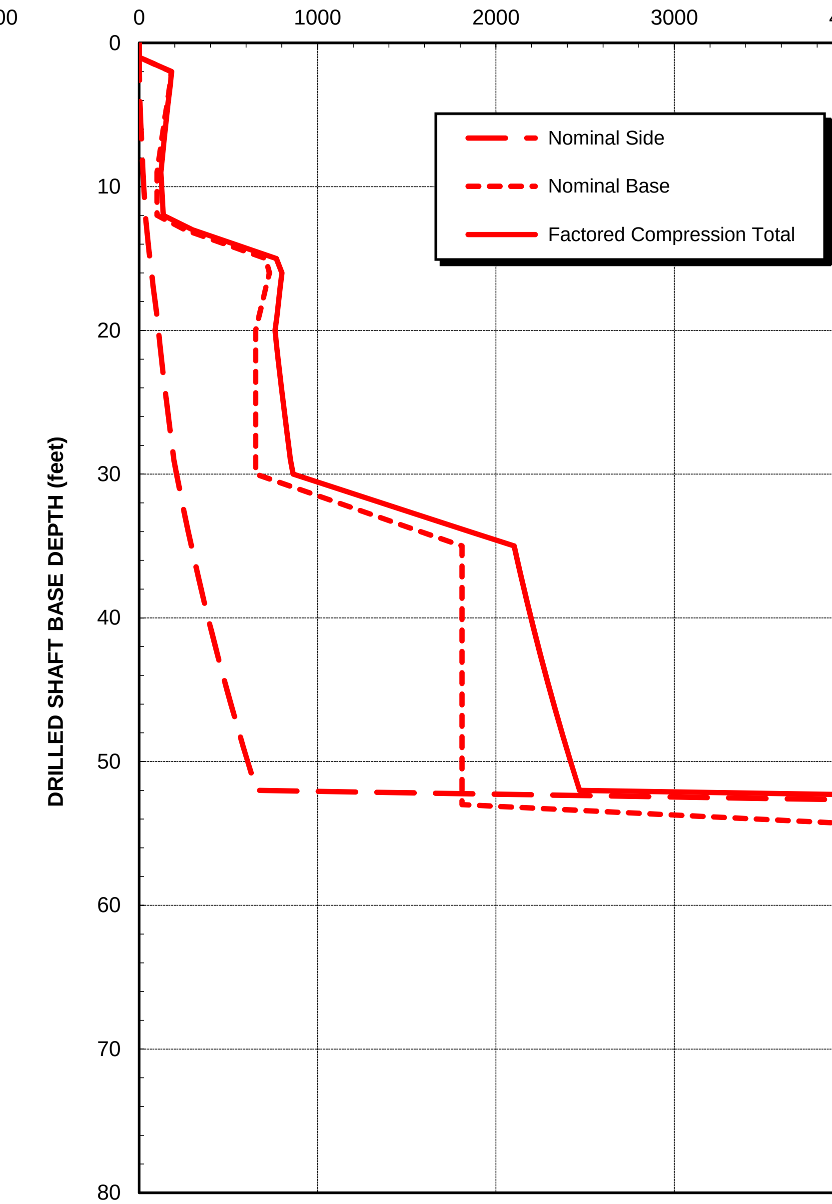
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



SERVICE LIMIT NOTES:

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

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per WSDOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 0 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

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

CRP693 - West Cashmere Bridge
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ESTIMATED AXIAL SHAFT RESISTANCE
8-FOOT-DIAM. DRILLED SHAFT
BENT 3

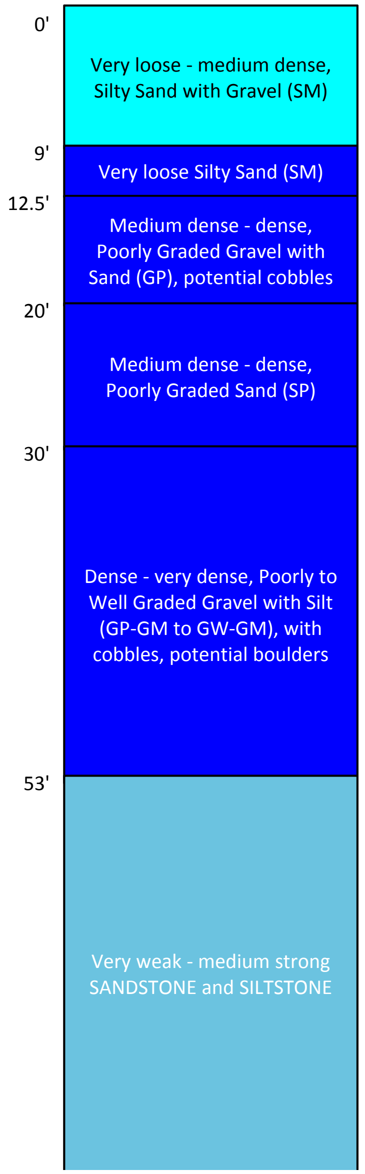
July 2016 22-1-03144-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 7

ASSUMED SUBSURFACE PROFILE

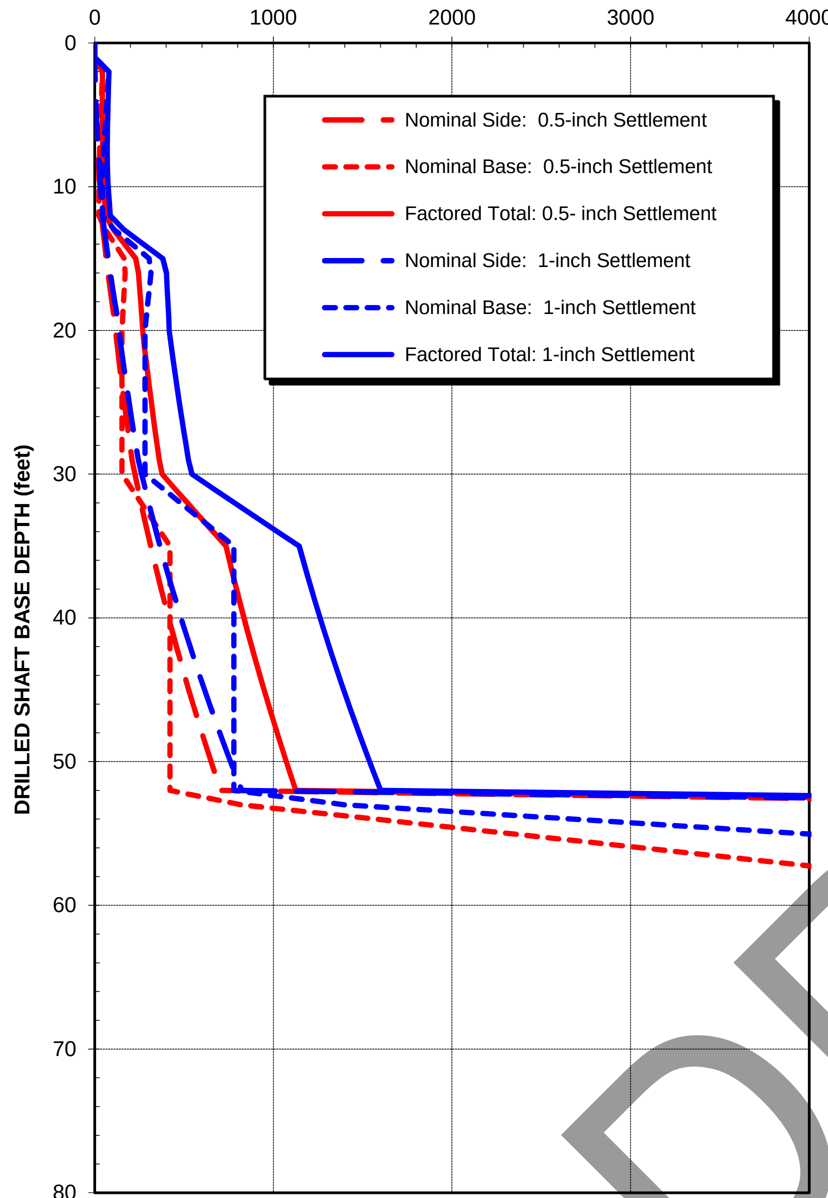
Based on Nearby Explorations:
B-4



Bottom of Boring at 80.0 feet.

SERVICE LIMIT

NOMINAL RESISTANCE (tons)

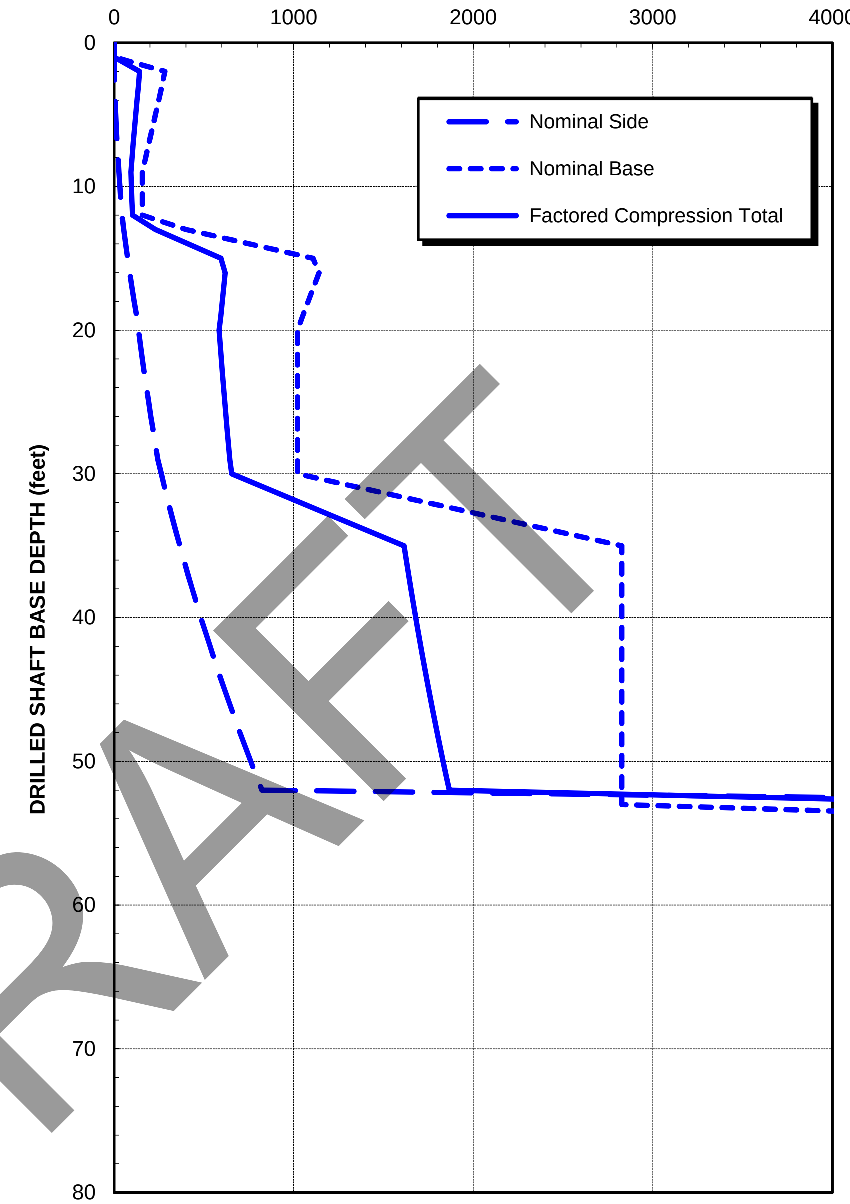


SERVICE LIMIT NOTES:

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

STRENGTH LIMIT

NOMINAL RESISTANCE (tons)

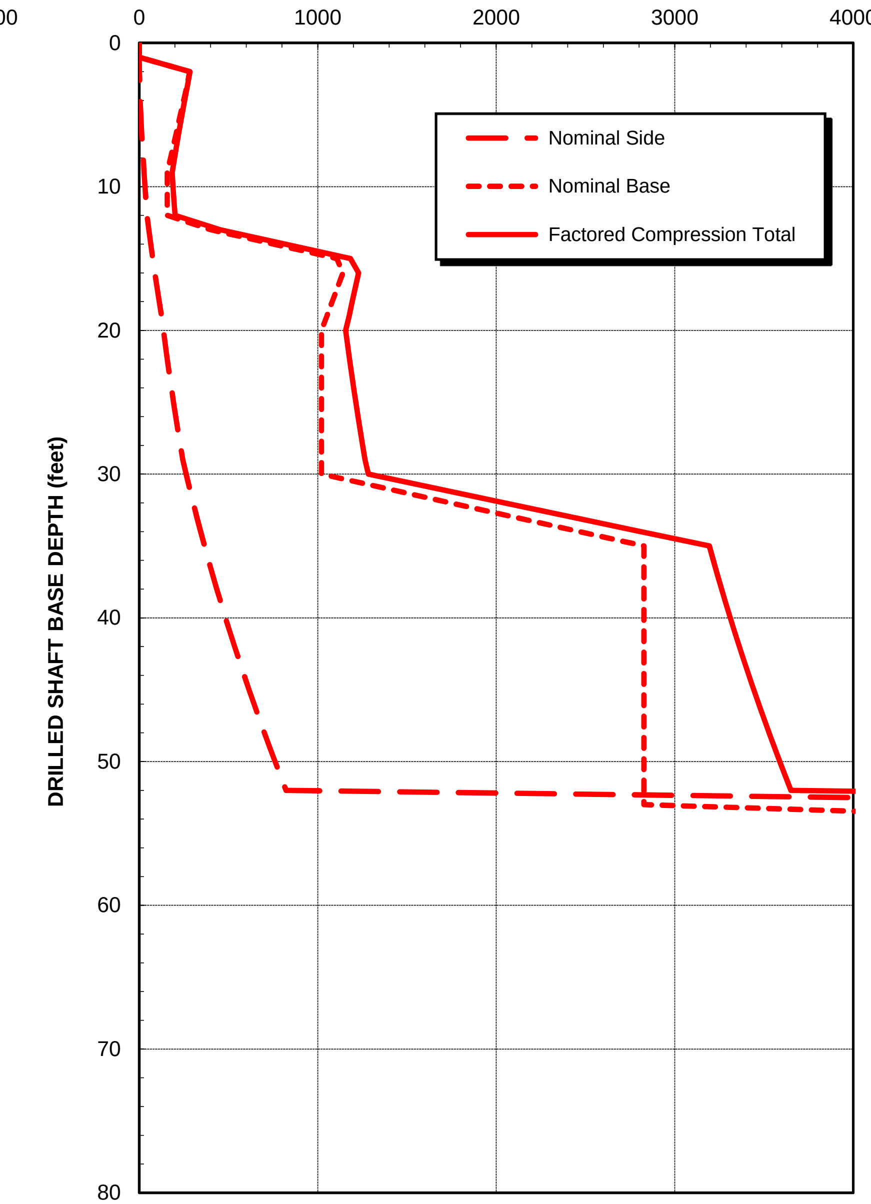


STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per WSDOT GDM).

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 0 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

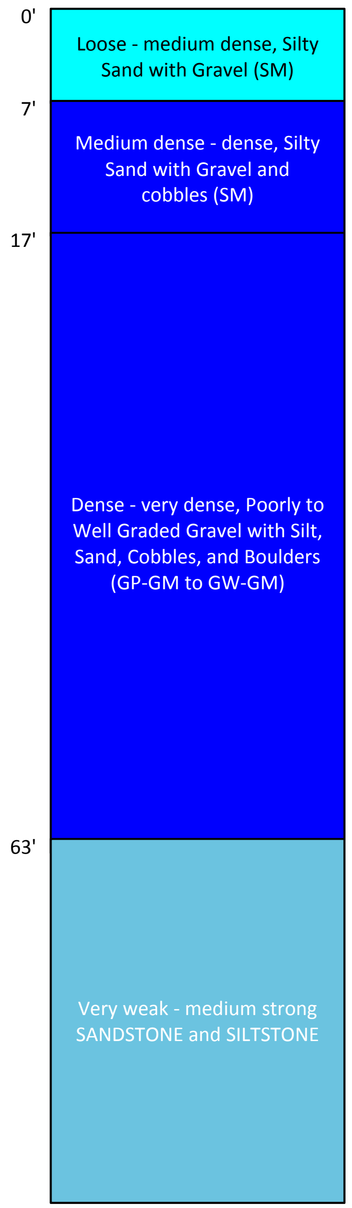
GENERAL NOTES

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

CRP693 - West Cashmere Bridge (Wenatchee River) Replacement Chelan County, Washington	
ESTIMATED AXIAL SHAFT RESISTANCE 10-FOOT-DIAM. DRILLED SHAFT BENT 2 - CABLE-STAYED	
July 2016	22-1-03144-001
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 8**

ASSUMED SUBSURFACE PROFILE

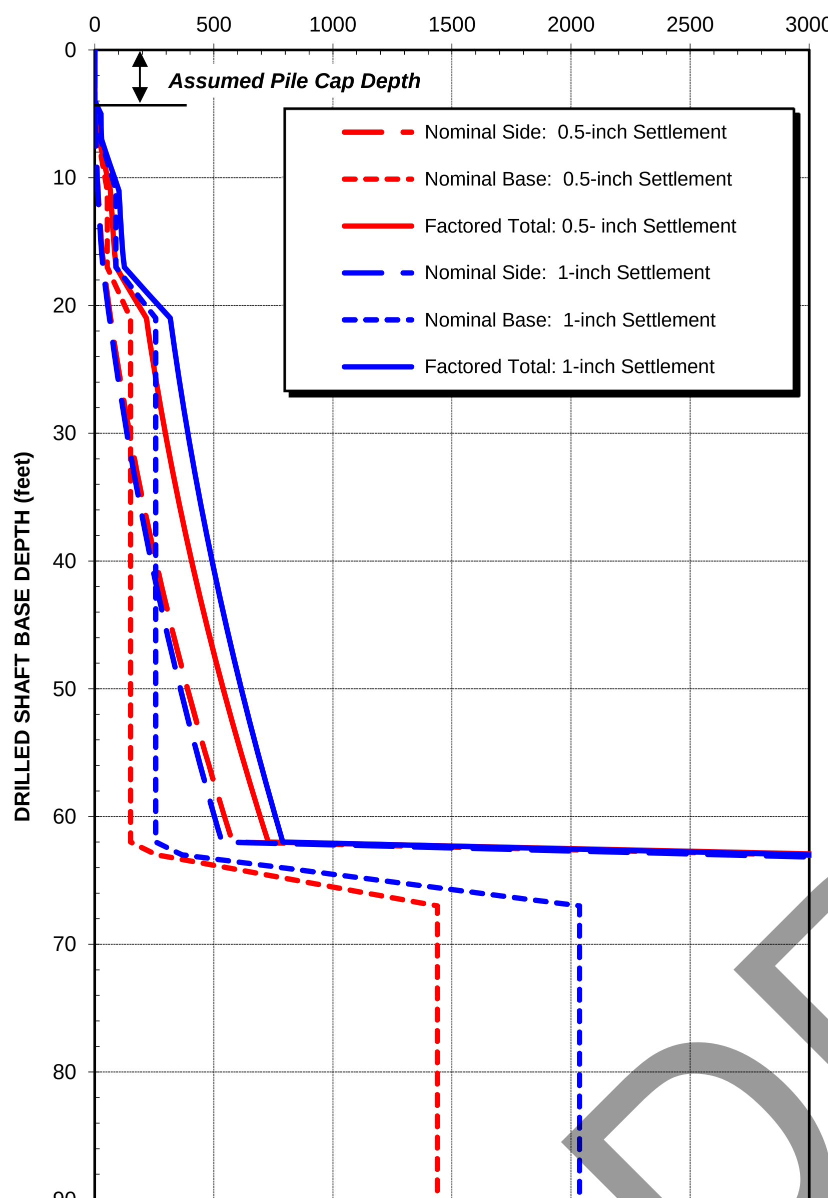
Based on Nearby Explorations:
B-5



Bottom of Boring at 90.0 feet

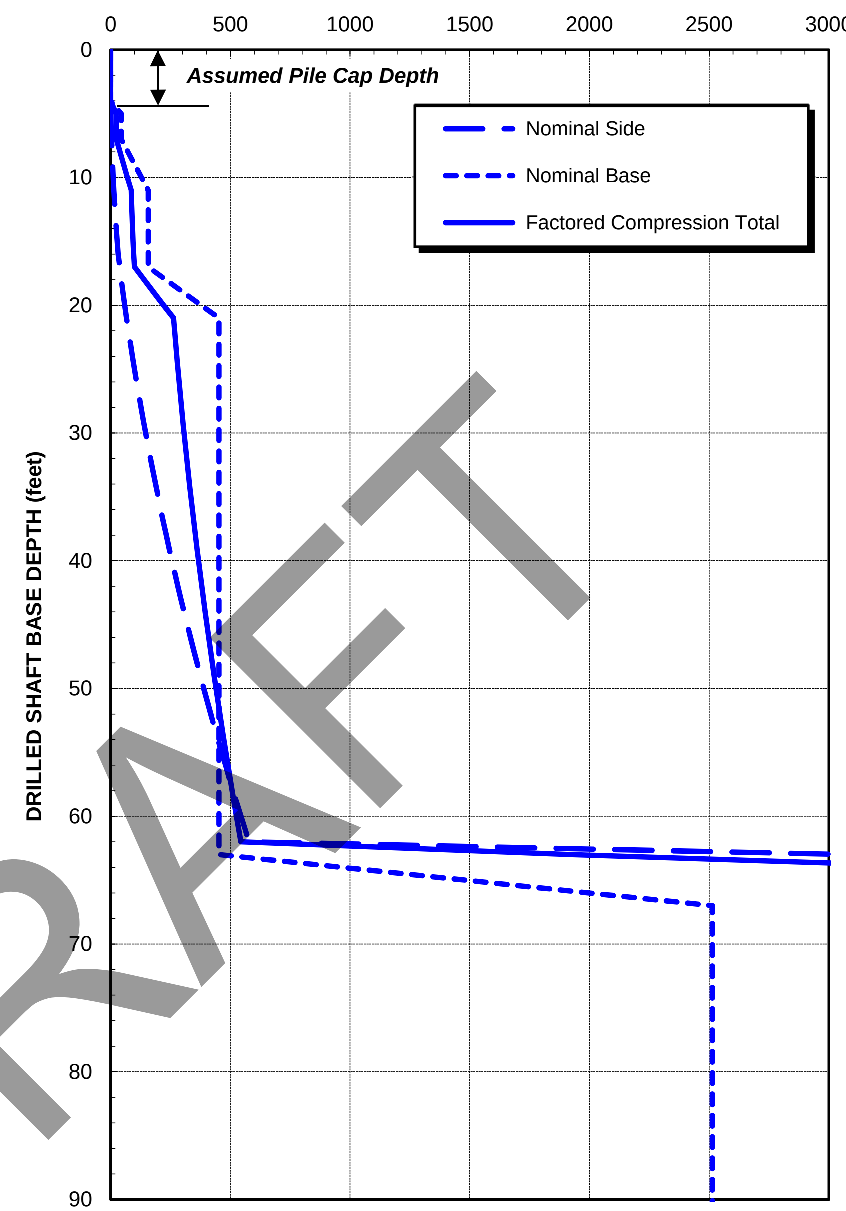
SERVICE LIMIT

NOMINAL RESISTANCE (tons)



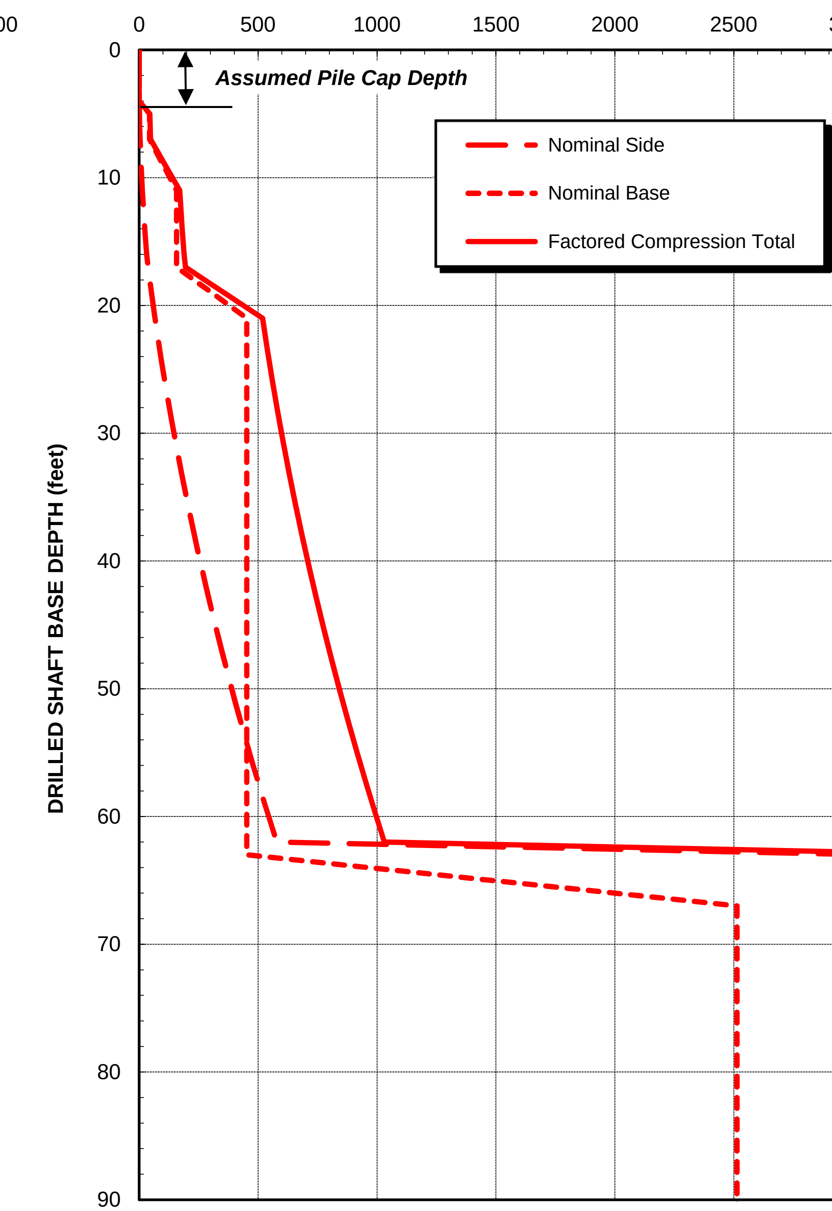
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



SERVICE LIMIT NOTES:

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

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per WSDOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored downdrag force is estimated to be 0 tons. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

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

CRP693 - West Cashmere Bridge
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ESTIMATED AXIAL SHAFT RESISTANCE
4-FOOT-DIAM. DRILLED SHAFT
BENT 4

July 2016 | 22-1-03144-001

SHANNON & WILSON, INC.
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FIG. 9

APPENDIX A

EXPLORATORY BORING LOGS

APPENDIX A

EXPLORATORY BORING LOGS

TABLE OF CONTENTS

FIGURES

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[2]	FINE-GRAINED SOILS (50% or more fines)[1]	COARSE-GRAINED SOILS (less than 50% fines)[1]
Major	***Silt, Lean Clay, Elastic Silt,*** or ***Fat Clay***[3]	***Sand*** or ***Gravel***[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: ***Sandy*** or ***Gravelly***[4]	More than 12% fine-grained: ***Silty*** or ***Clayey***[3]
Minor Follows major constituent	15% to 30% coarse-grained: ***with Sand*** or ***with Gravel***[4]	5% to 12% fine-grained: ***with Silt*** or ***with Clay***[3]
	30% or more total coarse-grained *and* lesser coarse-grained constituent is 15% or more: ***with Sand*** or ***with Gravel***[5]	15% or more of a second coarse-grained constituent: ***with Sand*** or ***with Gravel***[5]

[1]All percentages are by weight of total specimen passing a 3-inch sieve.
[2]The order of terms is: *Modifying Major with Minor*.
[3]Determined based on behavior.
[4]Determined based on which constituent comprises a larger percentage.
[5]Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm NOTE: If automatic hammers are used, blow counts shown on boring logs should be adjusted to account for efficiency of hammer.
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND	
Fine	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.)
Medium	#40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.)
Coarse	#10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL	
Fine	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.)
Coarse	3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

Bentonite Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Inclinometer or Non-perforated Casing
Perforated or Screened Casing	Vibrating Wire Piezometer

PERCENTAGES TERMS [1, 2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1]Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

[2]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

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(Wenatchee River) Replacement
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SOIL DESCRIPTION AND LOG KEY

July 2016 22-1-03144

SHANNON & WILSON, INC.
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FIG. A-1
Sheet 1 of 3

SOIL_CLASS_KEY_PG1 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW	Well-Graded Gravel; Well-Graded Gravel with Sand
			GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM	Silty Gravel; Silty Gravel with Sand
			GC	Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW	Well-Graded Sand; Well-Graded Sand with Gravel
			SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM	Silty Sand; Silty Sand with Gravel
			SC	Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat or other highly organic soils (see ASTM D4427)

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart. Graphics shown on the logs for these soil types are a combination of the two graphic symbols (e.g., SP and SM).

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.

CRP693 - West Cashmere Bridge (Wenatchee River) Replacement Chelan County, Washington	
SOIL DESCRIPTION AND LOG KEY	
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SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. A-1** Sheet 2 of 3

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASITICITY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20
High	It takes considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

[1]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

[2]Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

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Chelan County, Washington

SOIL DESCRIPTION AND LOG KEY

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FIG. A-1
Sheet 3 of 3

Total Depth:	*20 ft.*	Northing:		Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	*8 in.*
Top Elevation:	*~ 838.14 ft.*	Easting:		Drilling Company:	*HazTech Drilling*	Rod Diam.:	*NWJ (2 5/8" OD)*
Vert. Datum:		Station:		Drill Rig Equipment:	*CME 85*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples
Asphalt.	0.3	
Base Course.	1.0	
Medium dense, gray-brown, *Silty Sand with Gravel and Cobbles (SM)*; moist; subangular to subrounded, fine to coarse gravel and cobbles up to 4-inches; fine to coarse sand; nonplastic to low plasticity fines. Alluvium (Qal)	7.5	S-1, S-2
Medium dense to dense, gray-brown, *Poorly Graded Gravel with Silt, Sand, Cobbles, and Boulders (GP-GM)*; moist; subangular to subrounded gravel, cobbles, and boulders; fine to coarse sand; nonplastic to low plasticity fines. Alluvium (Qal) - Blow counts in this layer may be artificially high due to the presence of gravel, cobbles, and boulders.	20.0	S-3, S-4, S-5, S-6, S-7

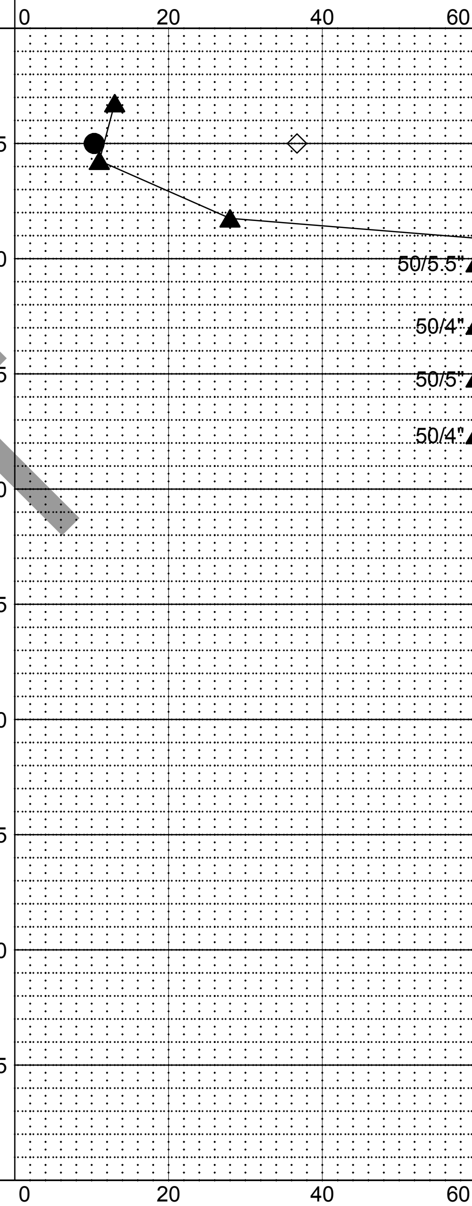
Screen Design: No groundwater encountered

Bottom of Boring
Completed 04/13/2016
Latitude: 47.52660
Longitude: -120.48876

Note: The presence of cobbles and boulders within layers described above is based on drill action and cuttings observation at the ground surface.

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

* Sample Not Recovered
⌶ 2.0" O.D. Split Spoon Sample

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge (Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-1

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2

REV 3 - Approved for Submittal

Log: JWT Rev: CAW Typ: JWT

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16

Total Depth: *93.7 ft.*	Northing:	Drilling Method:	*HSA, Mud Rotary & Core*	Hole Diam.:	*8 in.*
Top Elevation: *~ 836.48 ft.*	Easting:	Drilling Company:	*HazTech Drilling*	Rod Diam.:	*NWJ (2 5/8" OD)*
Vert. Datum:	Station:	Drill Rig Equipment:	*CME 85*	Hammer Type:	*Automatic*
Horiz. Datum:	Offset:	Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Screen Design | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

Medium dense to dense, brown, *Well Graded Gravel with Silt, Sand, Cobbles, and Boulders (GW-GM)* to *Poorly Graded Gravel with Silt, Sand, Cobbles, and Boulders (GP-GM)*; moist; subangular to subrounded, fine to coarse gravel, cobbles, and boulders up to 2 feet; fine to coarse sand; nonplastic to low plasticity fines.
Alluvium (Qal)
- Blow counts in this layer may be artificially high due to the presence of gravel, cobbles, and boulders.
- Switched from hollow stem auger to casing advance drilling methods at 7.5 feet.
- Sample wet at 10 feet due to change in drill method.

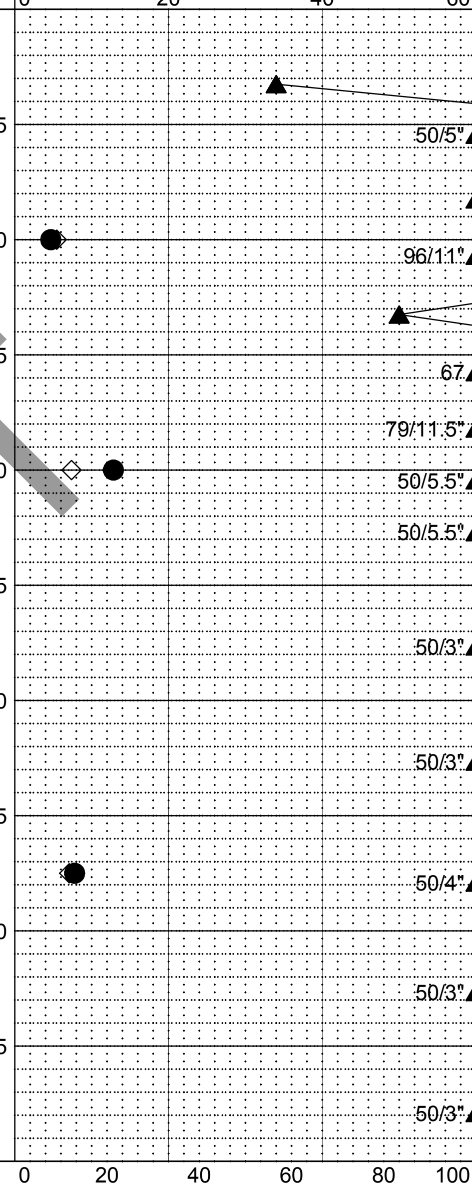
10.0

Medium dense to dense, gray-brown, *Well Graded Gravel with Silt, Sand, Cobbles, and Boulders (GW-GM), Well Graded Sand with Silt and Gravel (SW-SM)* and *Poorly Graded Gravel with Silt, Sand, Cobbles, and Boulders (GP-GM)*; wet; subangular to subrounded, fine to coarse gravel, cobbles, and boulders up to 2 feet; fine to coarse sand.
Alluvium (Qal)

- Blow counts in this layer may be artificially high due to the presence of gravel, cobbles, and boulders.

S-1, S-2, S-3, S-4, S-5, S-6, S-7, S-8, S-9, S-10, S-11, S-12, S-13, S-14

During Drilling



DRAFT

CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered
2.0" O.D. Split Spoon Sample
Rock Core
Ground Water Level ATD

RQD (%) Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content *(use scale at top)*

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-2

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-3
Sheet 1 of 3

REV 3 - Approved for Submittal

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16 Log: JWT Rev: CAW Typ: JWT

Total Depth:	93.7 ft.	Northing:		Drilling Method:	HSA, Mud Rotary & Core
Top Elevation:	~ 836.48 ft.	Easting:		Drilling Company:	HazTech Drilling
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85
Horiz. Datum:		Offset:		Other Comments:	

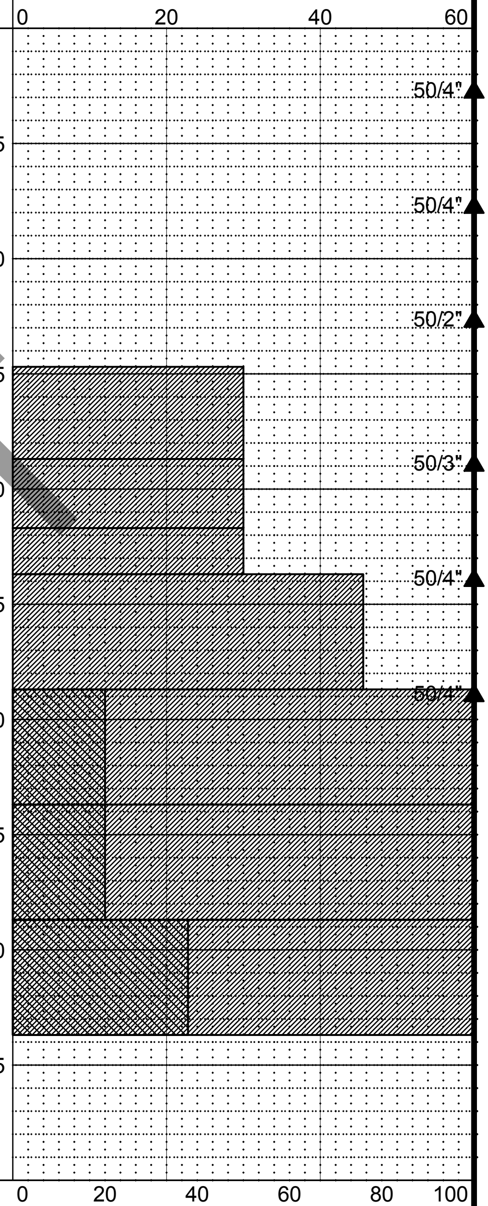
Hole Diam.: 8 in.
Rod Diam.: NWJ (2 5/8" OD)
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.	Samples
		S-15, S-16, S-17*
- Switched from casing advance to HQ core drilling at 64.7 feet.		R-1, S-18, R-2, R-3, S-19
SANDSTONE: weak to medium strong, gray to gray-brown, fine to medium grained; smooth to rough, very close to close spaced, low to high angle joints, with clay infilling and iron-oxide staining; slightly to highly weathered; high angle bedding. Chumstick Formation - Few siltstone interbeds from 82 to 83.7 feet.	73.7	R-4, S-20, R-5
SILTSTONE: very weak to weak, gray, fine grained, few sandy siltstone interbeds; smooth to rough, very close to close spaced, low to high angle joints, with clay and mineral infilling, iron-oxide staining; slightly to moderately weathered. Chumstick Formation (Tc)	83.7 / 85.8 / 88.7 / 90.0	R-6, R-7
SANDSTONE: weak to medium strong, light gray, fine to medium grained; smooth, very close to close spaced, low to high angle joints, with clay infilling and iron-oxide staining, trace	93.7	

PENETRATION RESISTANCE (blows/foot)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Rock Core
- Ground Water Level ATD

RQD (%) Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content (use scale at top)

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-2

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-3
Sheet 2 of 3

REV 3 - Approved for Submittal

DRAFT

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16 Log: JWT Rev: CAW Typ: JWT

Total Depth:	93.7 ft.	Northing:		Drilling Method:	HSA, Mud Rotary & Core
Top Elevation:	~ 836.48 ft.	Easting:		Drilling Company:	HazTech Drilling
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 8 in.
Rod Diam.: NWJ (2 5/8" OD)
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Screen Design | Depth, ft.

PENETRATION RESISTANCE (blows/foot)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches

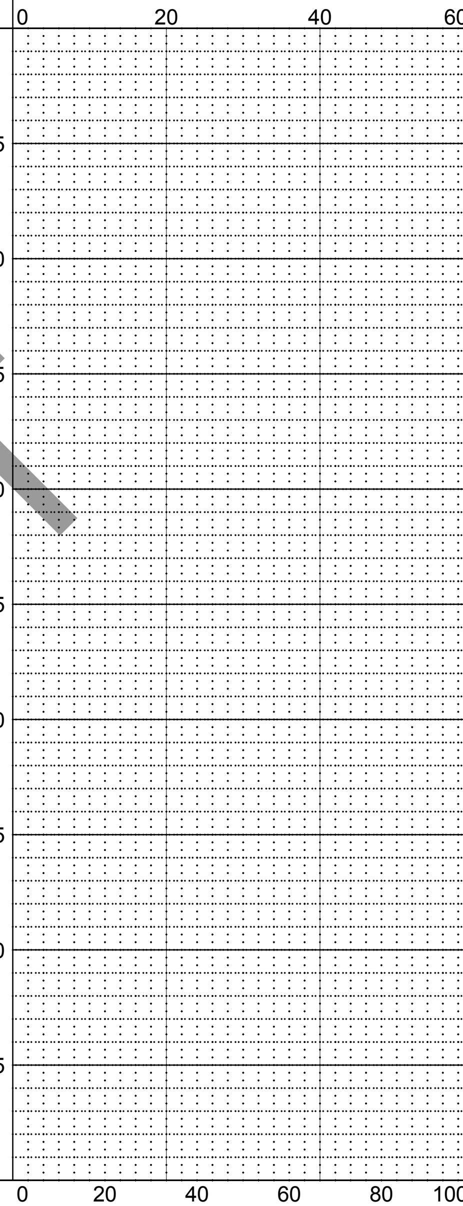
slickensides; slightly weathered.
Chumstick Formation (Tc)

SANDY SILTSTONE: weak, gray, fine grained; smooth, close spaced, low to high angle joints, with mineral infilling; slightly weathered.
Chumstick Formation (Tc)

SANDSTONE: strong, gray, fine to medium grained; smooth, widely spaced joints; fresh to slightly weathered; high angle bedding.
Chumstick Formation (Tc)
- Highly to completely weathered claystone interbed from 92.9 to 93.5 feet.

Bottom of Boring
Completed 04/16/2016
Latitude: 47.52678
Longitude: -120.48888

Note: The presence of cobbles and boulders within layers described above is based on drill action and cuttings observation at the ground surface.



◻ RQD (%) ▨ Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content (use scale at top)

LEGEND

* Sample Not Recovered
⌶ 2.0" O.D. Split Spoon Sample
▮ Rock Core
⊻ Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-2

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-3
Sheet 3 of 3

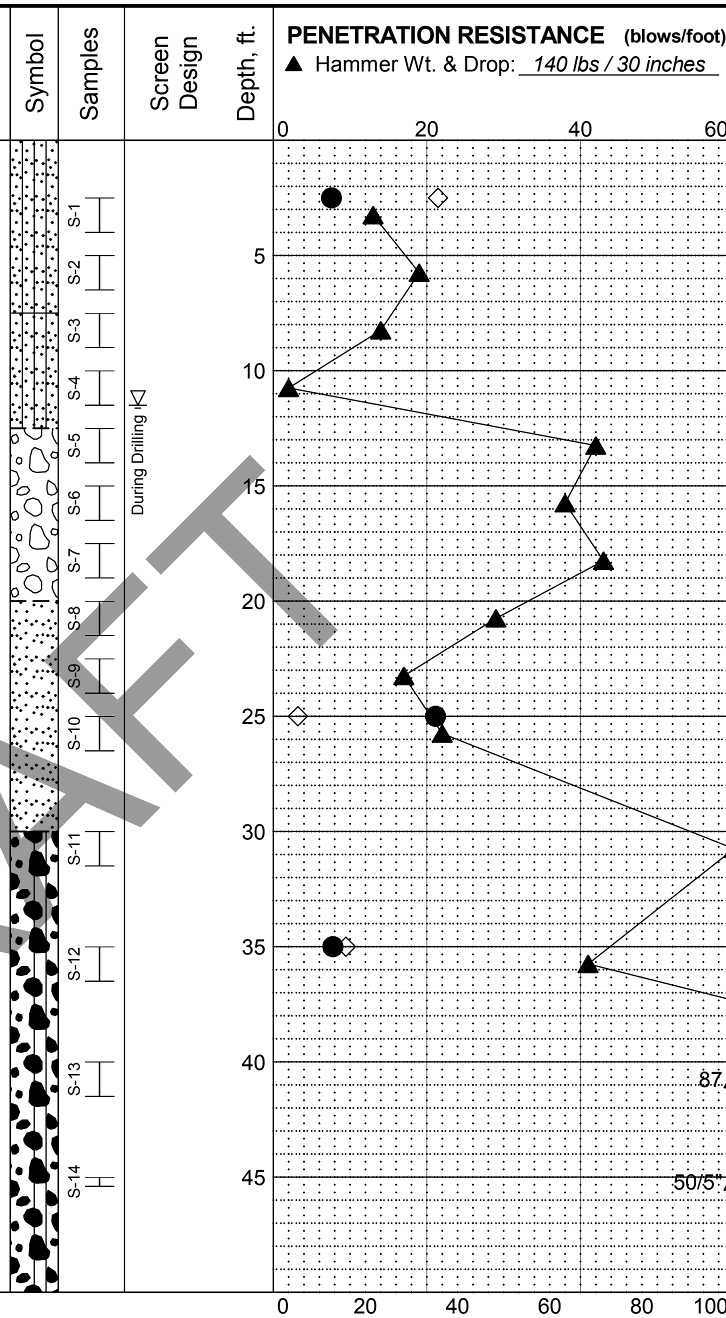
REV 3 - Approved for Submittal

Total Depth:	80 ft.	Northing:		Drilling Method:	HSA and Rock Core	Hole Diam.:	8 in.
Top Elevation:	~ 809.71 ft.	Easting:		Drilling Company:	HazTech Drilling	Rod Diam.:	NWJ (2 5/8" OD)
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.
Loose to medium dense, brown, *Silty Sand with Gravel (SM)*; moist to wet; fine to coarse gravel up to 3-inches; fine to coarse sand; nonplastic fines. Fill (Hf) - Blow counts may be artificially high due to the presence of gravel	7.5
Very loose, brown, *Silty Sand (SM)*; wet; trace fine gravel; fine to medium sand; nonplastic fines. Alluvium (Qal)	12.5
Medium dense to dense, gray-brown, *Poorly Graded Gravel with Sand and Cobbles (GP)*; moist to wet; fine to coarse gravel and cobbles up to 4-inches; fine to coarse sand; nonplastic fines. Alluvium (Qal) - Blow counts may be artificially high due to the presence of coarse gravel.	20.0
Medium dense to dense, gray-brown, *Poorly Graded Sand (SP)*; wet; trace to few fine gravel; fine to coarse sand. Alluvium (Qal)	30.0
Dense to very dense, gray-brown, *Well Graded Gravel with Silt, Sand, and Cobbles (GW-GM)*; wet; subangular to subrounded, fine to coarse gravel and cobbles up to 4-inches; fine to coarse sand; nonplastic to low plasticity fines. Alluvium (Qal) - Blow counts may be artificially high due to the presence of gravel and cobbles.	



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Rock Core

Ground Water Level ATD

RQD (%) Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content (use scale at top)

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-4

July 2016 22-1-03144

SHANNON & WILSON, INC.
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FIG. A-4
Sheet 1 of 2

DRAFT

REV 3 - Approved for Submittal

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16

Log: JWT Rev: CAW Typ: JWT

Total Depth:	80 ft.	Northing:		Drilling Method:	HSA and Rock Core	Hole Diam.:	8 in.
Top Elevation:	~ 809.71 ft.	Easting:		Drilling Company:	HazTech Drilling	Rod Diam.:	NWJ (2 5/8" OD)
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

- Switch from hollow stem auger to HQ core drilling at 50 feet.

53.0

SANDSTONE: medium strong to very strong, gray, fine to medium grained; smooth to rough, very close to closely spaced, joints, with mineral infilling and iron-oxide staining, trace slickensides; slightly weathered; high angle bedding.
Chumstick Formation (Tc)

57.0 – 62.0

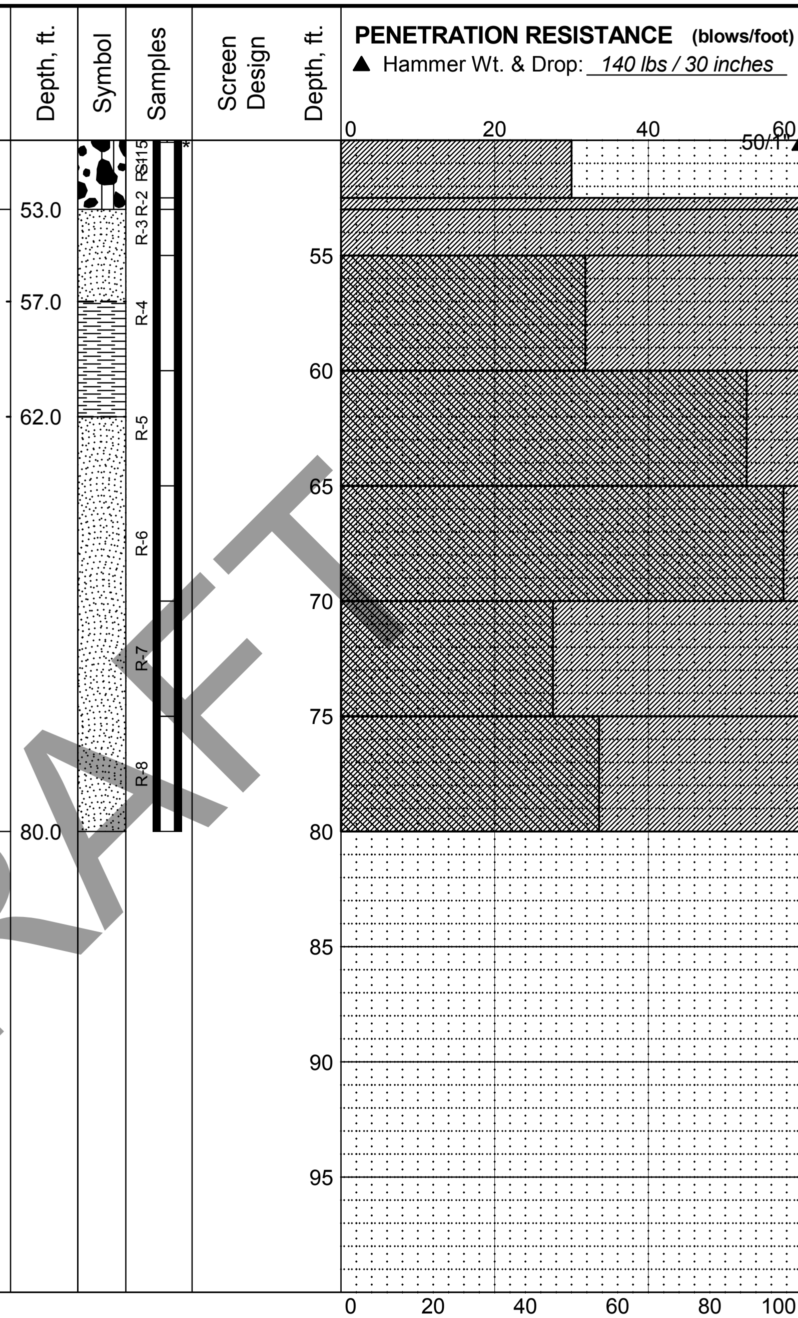
SILTSTONE: weak to medium strong, gray, fine grained; smooth to rough, close to moderately spaced, low to high angle joints, with clay and mineral infilling; slightly weathered; 70-degree contact with sandstone at 57 feet.
Chumstick Formation (Tc)

SANDSTONE: strong to very strong, light gray, fine to coarse; smooth, close to moderately spaced, low to high angle joints, with mineral and clay infilling, trace slickensides; slightly weathered; 70-degree contact with siltstone at 62 feet.
Chumstick Formation (Tc)

80.0

Bottom of Boring
Completed 04/14/2016
Latitude: 47.52798
Longitude: -120.48963

Note: The presence of cobbles within layers described above is based on drill action and cuttings observation at the ground surface.



LEGEND

* Sample Not Recovered
2.0" O.D. Split Spoon Sample
Rock Core
Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-4

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-4
Sheet 2 of 2

REV 3 - Approved for Submittal

Total Depth:	90 ft.	Northing:		Drilling Method:	HSA and Rock Core
Top Elevation:	~ 820.56 ft.	Easting:		Drilling Company:	HazTech Drilling
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.:	8 in.
Rod Diam.:	NWJ (2 5/8" OD)
Hammer Type:	Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Screen Design | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

Loose to medium dense, brown, *Silty Sand with Gravel (SM)*; moist; fine to coarse, subangular gravel; fine to coarse sand; nonplastic to low plasticity fines.
Fill (Hf)

- Blow counts may be artificially high due to the presence of gravel, cobbles, and boulders.

10.0

Medium dense to dense, gray-brown, *Silty Sand with Gravel (SM)* to *Poorly Graded Sand with Silt, Gravel, and Cobbles (SP-SM)*; wet; subangular to subrounded, fine to coarse gravel and cobbles up to 4-inches; fine to coarse sand; nonplastic to low plasticity fines; weathered clasts.
Alluvium (Qal)

- Blow counts may be artificially high due to the presence of gravel and cobbles.

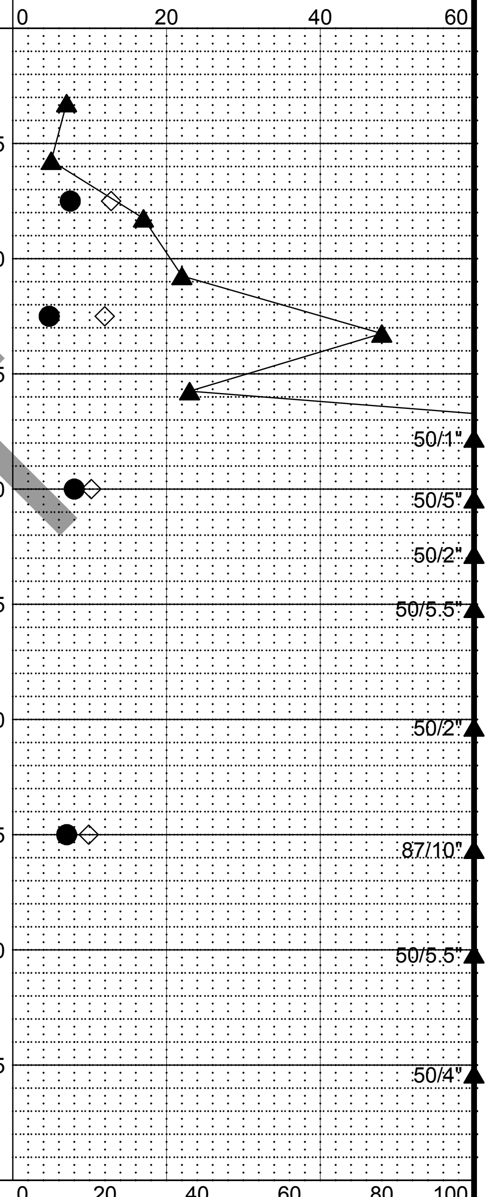
18.0

Dense, gray-brown, *Well Graded Gravel with Silt, Sand, Cobbles, and Boulders (GW-GM)* to *Well Graded Sand with Silt, Gravel, Cobbles, and Boulders (SW-SM)*; wet; subangular to subrounded, fine to coarse gravel, cobbles, and boulders; fine to coarse sand; nonplastic to low plasticity fines.
Alluvium (Qal)
- Weathered gravel at 35 feet.

- Blow counts may be artificially high due to the presence of gravel, cobbles, and boulders.

Samples: S-1, S-2 *, S-3, S-4, S-5, S-6, S-7, S-8, S-9, S-10, S-11, S-12, S-13, S-14

During Drilling



CONTINUED NEXT SHEET

RQD (%) Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content (use scale at top)

LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Rock Core

Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-5

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

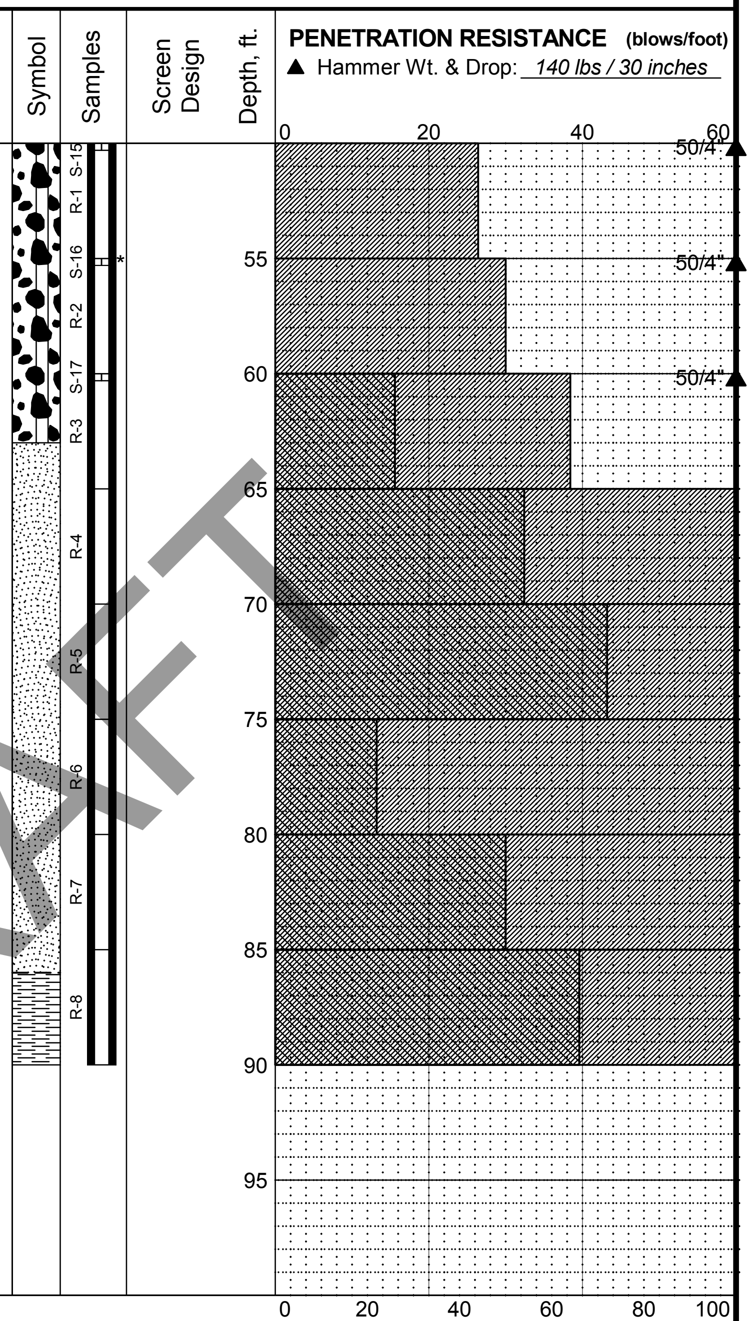
FIG. A-5
Sheet 1 of 3

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16 Log: JWT Rev: CAW Typ: JWT

REV 3 - Approved for Submittal

Total Depth: 90 ft.	Northing:	Drilling Method:	HSA and Rock Core	Hole Diam.:	8 in.
Top Elevation: ~ 820.56 ft.	Easting:	Drilling Company:	HazTech Drilling	Rod Diam.:	NWJ (2 5/8" OD)
Vert. Datum:	Station:	Drill Rig Equipment:	CME 85	Hammer Type:	Automatic
Horiz. Datum:	Offset:	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.*	Depth, ft.
- Switched from hollow stem auger to HQ core drilling at 50 feet.	
SANDSTONE: strong to very strong, gray to light gray, fine to medium grained; smooth to rough, close to moderately spaced, low to high angle joints, with clay infilling and iron-oxide staining; fresh to slightly weathered with highly to completely weathered layers. Chumstick Formation (Tc) - Sandy siltstone interbed from 82.6 to 83.6 feet. 70 degree contact with sandstone.	63.0
SILTSTONE: medium strong, gray, fine grained; smooth, closely spaced, low to high angle joints, with clay infilling and iron-oxide staining, trace slickensides; slightly weathered; 70 degree contact with sandstone at 86 feet. Chumstick Formation (Tc)	86.0
Bottom of Boring Completed 04/17/2016 Latitude: 47.52865 Longitude: -120.49004	90.0



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered
2.0" O.D. Split Spoon Sample
Rock Core
Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-5

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-5
Sheet 2 of 3

REV 3 - Approved for Submittal

Log: JWT Rev: CAW Typ: JWT

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16

Total Depth:	90 ft.	Northing:		Drilling Method:	HSA and Rock Core
Top Elevation:	~ 820.56 ft.	Easting:		Drilling Company:	HazTech Drilling
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 8 in.
Rod Diam.: NWJ (2 5/8" OD)
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Screen Design | Depth, ft.

PENETRATION RESISTANCE (blows/foot)
▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

0 20 40 60

Note: The presence of cobbles and boulders within layers described above is based on drill action and cuttings observation at the ground surface.

105
110
115
120
125
130
135
140
145

0 20 40 60 80 100

RQD (%) Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content ←*(use scale at top)*

LEGEND

* Sample Not Recovered
2.0" O.D. Split Spoon Sample
Rock Core
Ground Water Level ATD

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-5

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-5
Sheet 3 of 3

REV 3 - Approved for Submittal

Total Depth:	41.3 ft.	Northing:		Drilling Method:	HSA and Rock Core
Top Elevation:	~ 832.82 ft.	Easting:		Drilling Company:	HazTech Drilling
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 8 in.
Rod Diam.: NWJ (2 5/8" OD)
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Screen Design | Depth, ft.

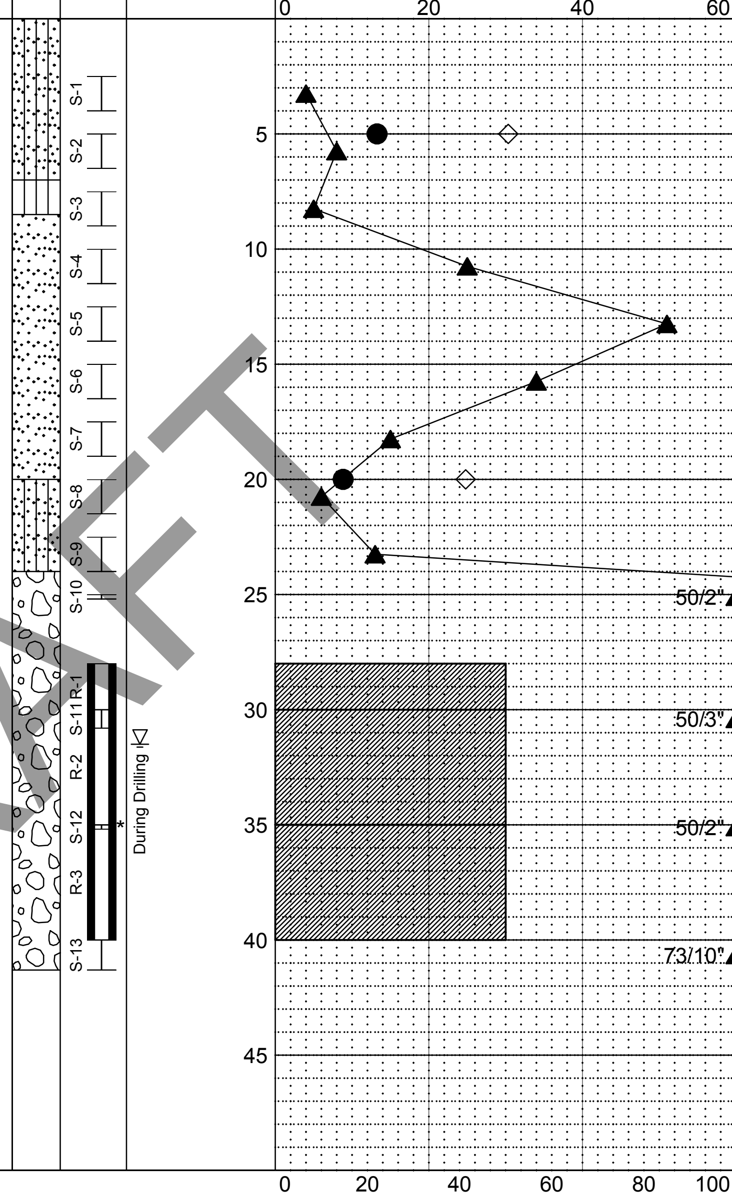
PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

Soil Description	Depth, ft.
Loose, brown, *Silty Sand with Gravel (SM)*; moist; fine gravel; fine to coarse sand; low plasticity fines; trace organics. Fill (Hf)	7.0
Loose, brown, *Silt with Sand (ML)*; moist; fine sand; nonplastic fines. Alluvium (Qal)	8.5
Medium dense, brown, *Poorly Graded Sand with Gravel (SP)* to *Poorly Graded Gravel with Sand (GP)*; moist; fine to coarse, angular to subrounded gravel; fine to coarse sand. Alluvium (Qal) - Blow counts may be artificially high due to the presence of coarse gravel.	20.0
Loose to medium dense, brown, *Silty Sand (SM)*; moist; trace fine gravel; fine to coarse sand; nonplastic fines. Alluvium (Qal)	24.0
Dense, gray, *Poorly Graded Gravel with Sand, Cobbles, and Boulders (GP)*; wet; subangular to subrounded, fine to coarse gravel, cobbles and boulders; fine to coarse sand; trace nonplastic to low plasticity fines. Alluvium (Qal) - Blow counts may be artificially high due to the presence of gravel, cobbles, and boulders. - Rough drill action starting at 24 feet. - Switch from hollow stem auger to HQ core drilling at 28 feet.	41.3

Bottom of Boring
Completed 04/16/2016
Latitude: 47.52913
Longitude: -120.49010

Note: The presence of cobbles and boulders



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered
2.0" O.D. Split Spoon Sample
Rock Core
Ground Water Level ATD

RQD (%) Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content (use scale at top)

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-6

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-6
Sheet 1 of 2

REV 3 - Approved for Submittal

DRAFT

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16 Log: JWT Rev: CAW Typ: JWT

Total Depth:	41.3 ft.	Northing:		Drilling Method:	HSA and Rock Core	Hole Diam.:	8 in.
Top Elevation:	~ 832.82 ft.	Easting:		Drilling Company:	HazTech Drilling	Rod Diam.:	NWJ (2 5/8" OD)
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Screen Design | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

0 20 40 60

within layers described above is based on drill action and cuttings observation at the ground surface.

55
60
65
70
75
80
85
90
95

0 20 40 60 80 100

RQD (%) Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content (use scale at top)

DRAFT

LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Rock Core

Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-6

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-6
Sheet 2 of 2

REV 3 - Approved for Submittal

Log: JWT Rev: CAW Typ: JWT

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16

Total Depth: 31.5 ft.	Northing:	Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	*8 in.*
Top Elevation: ~ 833.98 ft.	Easting:	Drilling Company:	*HazTech Drilling*	Rod Diam.:	*NWJ (2 5/8" OD)*
Vert. Datum:	Station:	Drill Rig Equipment:	*CME 85*	Hammer Type:	*Automatic*
Horiz. Datum:	Offset:	Other Comments:			

SOIL DESCRIPTION

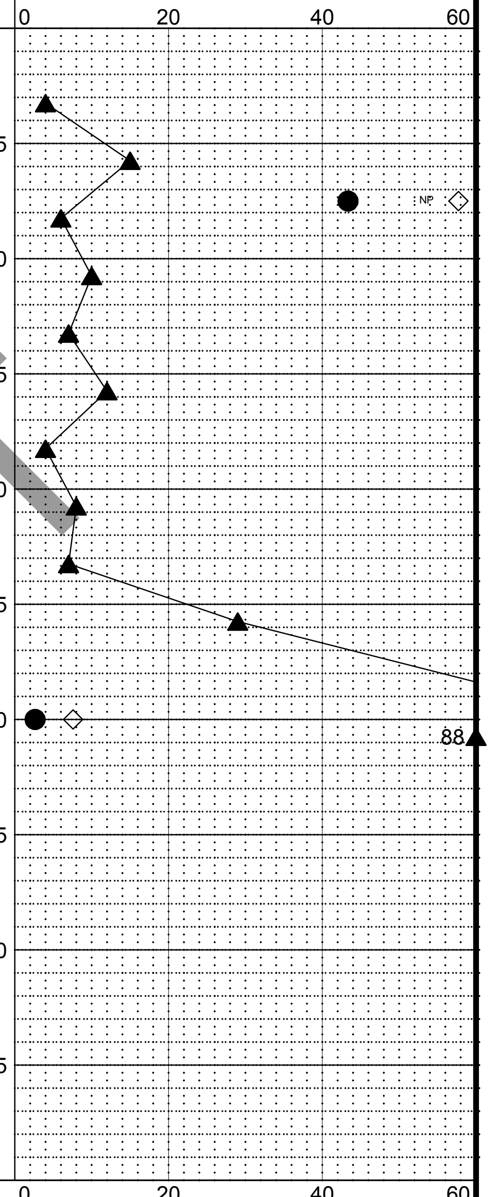
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples
Loose, brown, *Silty Sand with Gravel (SM)*; moist; fine to coarse gravel; fine to coarse sand; nonplastic fines; trace organics. Fill (Hf)		S-1
Medium dense, brown, *Silty Gravel with Sand (GM)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; nonplastic fines. Possible disturbed zone from previous orchard activities	4.5	S-2
Loose, tan, *Sandy Silt (ML)*; moist; nonplastic fines. Alluvium (Qal)	7.5	S-3
Medium dense, brown, *Poorly Graded Sand with Gravel (SP)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; trace fines. Alluvium (Qal)	10.5	S-4
Loose to medium dense, brown, *Silty Sand with Gravel (SM)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; nonplastic fines. Alluvium (Qal) - Grades to trace gravel at 17.5 feet.	12.5	S-5, S-6, S-7, S-8
Loose, brown, *Silty Sand (SM)*; moist; fine sand; nonplastic fines. Alluvium (Qal)	22.5	S-9, S-10
Dense, gray, *Well Graded Gravel with Silt, Sand and Cobbles (GW-GM)*; wet; subangular to subrounded, fine to coarse gravel, possible cobbles up to 6-inches based on drill action; fine to coarse sand; nonplastic fines. Alluvium (Qal) - Blow counts may be artificially high due to the presence of coarse gravel and cobbles.	26.0	S-11
	31.5	

Screen Design: No groundwater encountered

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge (Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-7

July 2016 | 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-7
Sheet 1 of 2

Log: JWT Rev: CAW Typ: JWT

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16

REV 3 - Approved for Submittal

Total Depth: *31.5 ft.*	Northing:	Drilling Method: *Hollow Stem Auger*	Hole Diam.: *8 in.*
Top Elevation: *~ 833.98 ft.*	Easting:	Drilling Company: *HazTech Drilling*	Rod Diam.: *NWJ (2 5/8" OD)*
Vert. Datum:	Station:	Drill Rig Equipment: *CME 85*	Hammer Type: *Automatic*
Horiz. Datum:	Offset:	Other Comments:	

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Screen Design | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

0 20 40 60

Bottom of Boring
Completed 04/15/2016
Latitude: 47.52915
Longitude: -120.48934

Note: The presence of cobbles and boulders within layers described above is based on drill action and cuttings observation at the ground surface.

55
60
65
70
75
80
85
90
95

0 20 40 60

DRAFT

LEGEND

* Sample Not Recovered

⌶ 2.0" O.D. Split Spoon Sample

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

LOG OF BORING B-7

July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-7
Sheet 2 of 2

MASTER_LOG_E 22-1-03144.GPJ SHAN_WIL.GDT 7/6/16 Log: JWT Rev: CAW Typ: JWT

REV 3 - Approved for Submittal

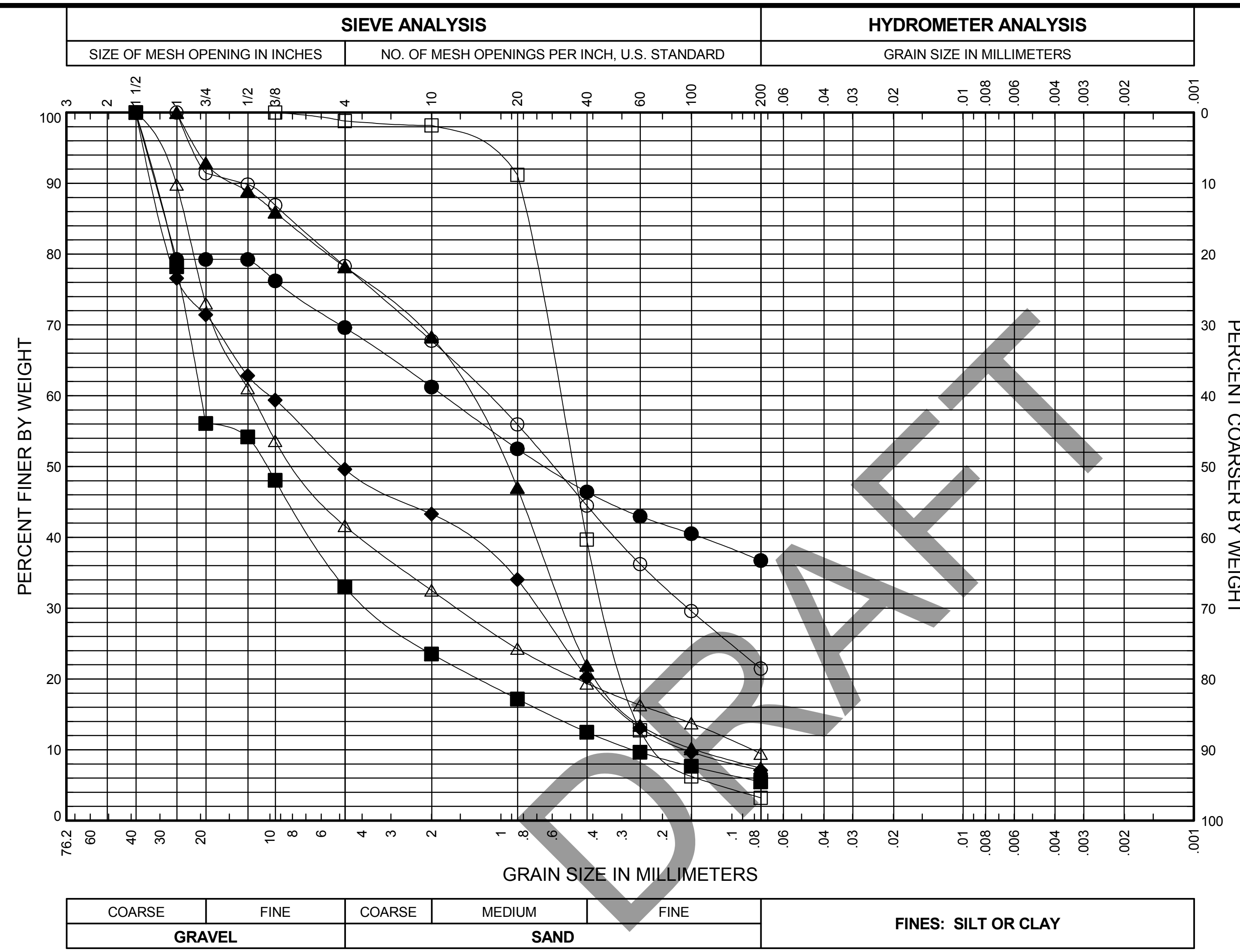
APPENDIX B

LABORATORY TEST RESULTS

APPENDIX B

LABORATORY TEST RESULTS

TABLE OF CONTENTS



LEGEND

USCS: Unified Soil Classification System

COBBLE %: Percentage of cobbles in test specimen; based on pre-removal total dry mass for normalized tests

NAT WC %: Natural water content

Cu: Coefficient of uniformity

Cc: Coefficient of curvature

ASTM DES: ASTM International test standard designation

*: *Sample specimen weight did not meet required minimum mass for ASTM test method.*

BORING AND SAMPLE NO.	DEPTH (feet)	USCS GROUP SYMBOL	USCS GROUP NAME	COBBLE %	GRAVEL %	SAND %	FINES %	< 0.02mm %	< 2um %	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● B-1, S-2*	5.0	SM	Silty Sand with Gravel		30	33	37			10.3			EAB	JFL	C136
■ B-2, S-4*	10.0	GW-GM	Well-Graded Gravel with Silt and Sand		67	28	5.5			4.7	74.8	2.4	EAB	JFL	C136
▲ B-2, S-8*	20.0	SW-SM	Well-Graded Sand with Silt and Gravel		22	71	7.4			12.8	10.0	1.4	EAB	JFL	C136
◆ B-2, S-12*	37.5	GP-GM	Poorly Graded Gravel with Silt and Sand		50	43	7.1			7.8	63.3	0.3	EAB	JFL	C136
○ B-4, S-1*	2.5	SM	Silty Sand with Gravel		22	57	21			7.6			EAB	JFL	C136
□ B-4, S-10	25.0	SP	Poorly Graded Sand		1	96	3.2			21.1	2.8	1.1	EAB	JFL	C136
△ B-4, S-12*	35.0	GW-GM	Well-Graded Gravel with Silt and Sand		58	32	9.5			7.8	146.9	2.4	EAB	JFL	C136

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

GRAIN SIZE DISTRIBUTION

July 2016 22-1-03144

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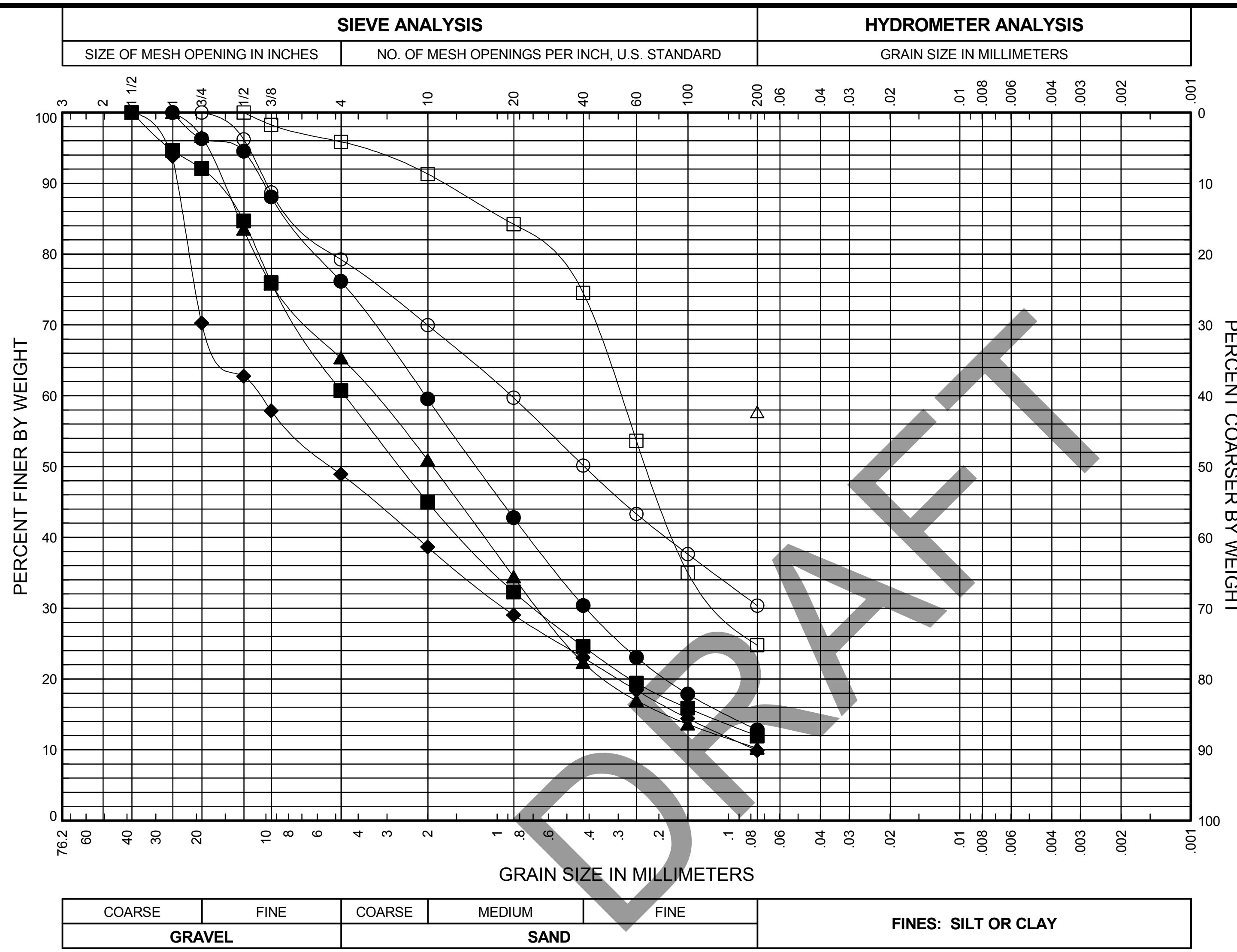
FIG. B-1
Sheet 1 of 3

FIG. B-1



LEGEND

USCS: Unified Soil Classification System

COBBLE %: Percentage of cobbles in test specimen; based on pre-removal total dry mass for normalized tests

NAT WC %: Natural water content

Cu: Coefficient of uniformity

Cc: Coefficient of curvature

ASTM DES: ASTM International test standard designation

*: *Sample specimen weight did not meet required minimum mass for ASTM test method.*

BORING AND SAMPLE NO.	DEPTH (feet)	USCS GROUP SYMBOL	USCS GROUP NAME	COBBLE %	GRAVEL %	SAND %	FINES %	< 0.02mm %	< 2um %	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● B-5, S-3*	7.5	SM	Silty Sand with Gravel		24	63	13			7.5			EAB	JFL	C136
■ B-5, S-5*	12.5	SM	Silty Sand with Gravel		39	49	12			4.7	85.9	2.0	EAB	JFL	C136
▲ B-5, S-8*	20.0	SW-SM	Well-Graded Sand with Silt and Gravel		35	55	10			8.0	48.2	1.8	EAB	JFL	C136
◆ B-5, S-12*	35.0	GW-GM	Well-Graded Gravel with Silt and Sand		51	39	9.9			7.0	140.3	1.0	EAB	JFL	C136
○ B-6, S-2*	5.0	SM	Silty Sand with Gravel		21	49	30			13.2			EAB	JFL	C136
□ B-6, S-8*	20.0	SM	Silty Sand		4	71	25			8.8			EAB	JFL	C136
△ B-7, S-3*	7.5	ML	Sandy Silt				58			43.4			EAB	JFL	D1140

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

GRAIN SIZE DISTRIBUTION

July 2016 22-1-03144

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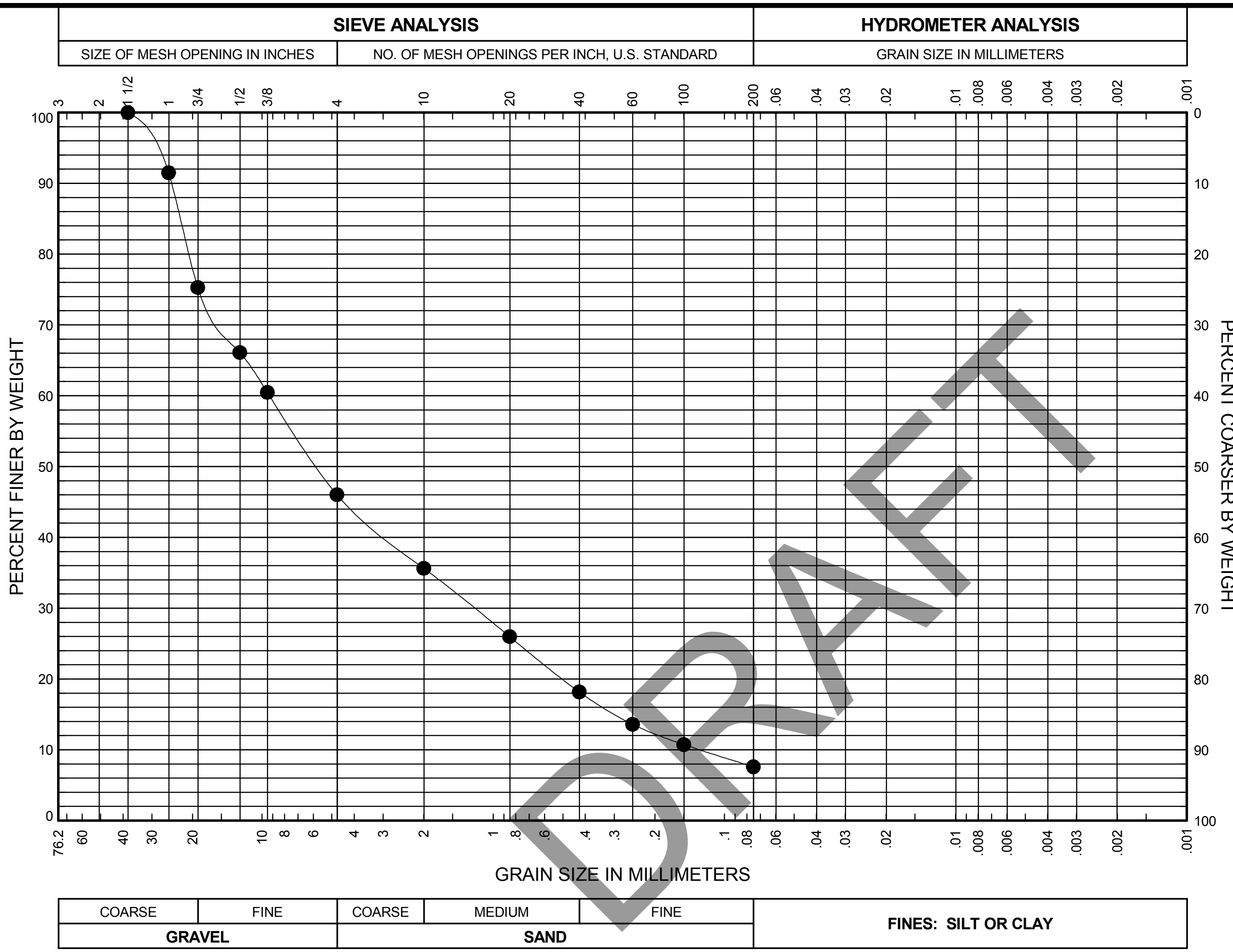
FIG. B-1
Sheet 2 of 3

Chelan County Public Works

AASHTO GSA TESTA 22-1-03144.GPJ SHAN WIL.GDT 7/6/16

FIG. B-1



AASHTO GSA TESTA 22-1-03144.GPJ SHAN WIL.GDT 7/6/16

LEGEND

USCS: Unified Soil Classification System

COBBLE %: Percentage of cobbles in test specimen; based on pre-removal total dry mass for normalized tests

NAT WC %: Natural water content

Cu: Coefficient of uniformity

Cc: Coefficient of curvature

ASTM DES: ASTM International test standard designation

*: *Sample specimen weight did not meet required minimum mass for ASTM test method.*

BORING AND SAMPLE NO.	DEPTH (feet)	USCS GROUP SYMBOL	USCS GROUP NAME	COBBLE %	GRAVEL %	SAND %	FINES %	< 0.02mm %	< 2um %	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● B-7, S-11*	30.0	GW-GM	Well-Graded Gravel with Silt and Sand		54	38	7.6			2.6	72.8	1.2	EAB	JFL	C136

CRP693 - West Cashmere Bridge (Wenatchee River) Replacement
Chelan County, Washington

GRAIN SIZE DISTRIBUTION

July 2016 22-1-03144

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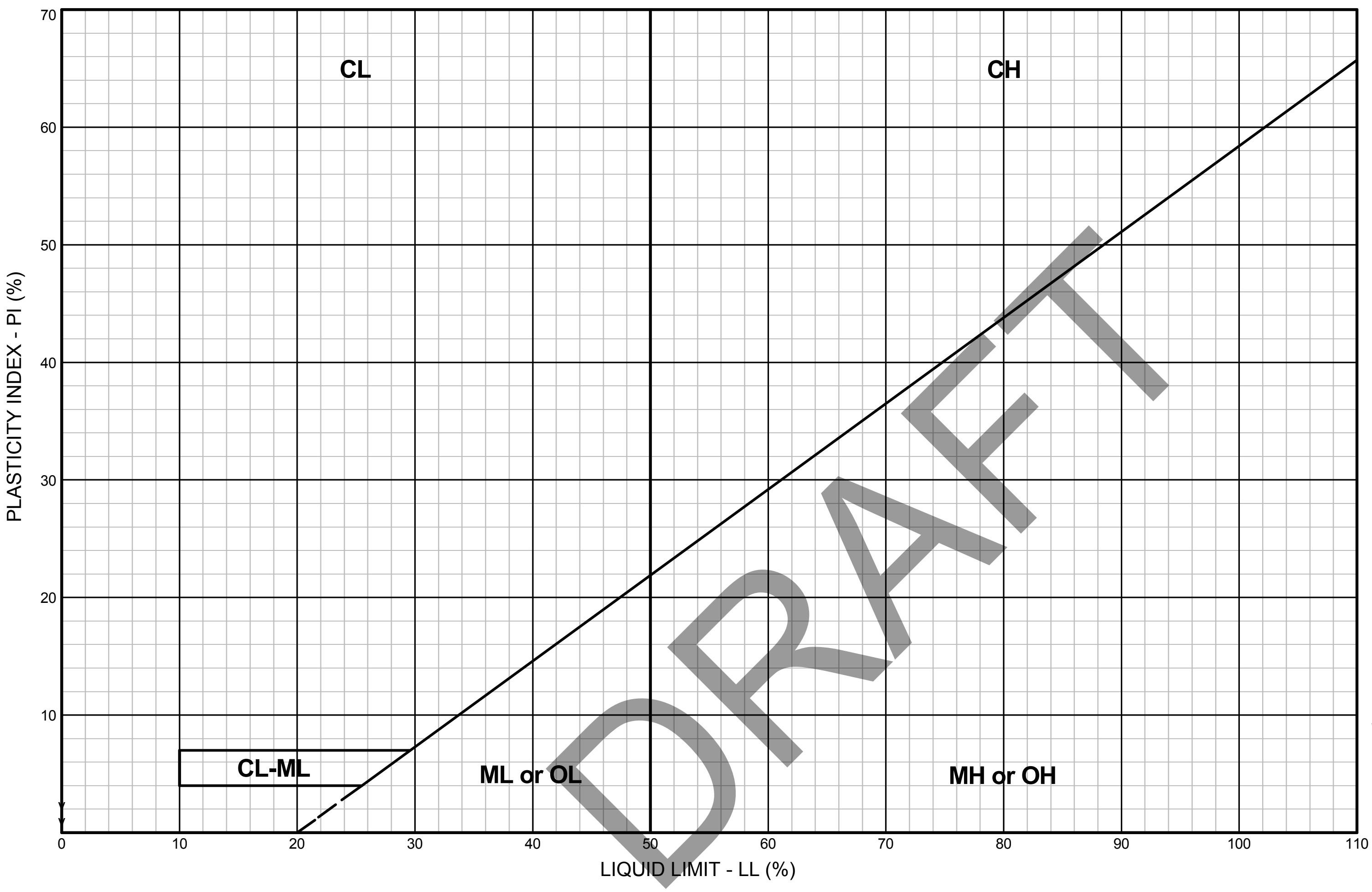
FIG. B-1
Sheet 3 of 3

Chelan County Public Works

FIG. B-1



LEGEND

USCS: Unified Soil Classification System

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML: Inorganic silts and clayey silts of low plasticity

MH: Inorganic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

OL: Organic silts and clays of low plasticity

OH: Organic silts and clays of high plasticity

LL: Liquid limit

PL: Plastic limit

PI: Plasticity index; PI=LL-PL

NP: Nonplastic

⩡ : Nonplastic

^,>>: Test value exceeds limit of graph

FINES %: Percentage of specimen mass passing the No. 200 sieve

NAT WC%: Natural water content

< 2 um %: Percentage of soil particles finer than 2 micrometers (0.002 mm); clay-size fraction

ASTM DES: ASTM International test standard designation

BORING AND SAMPLE NO.	DEPTH (feet)	USCS GROUP SYMBOL	USCS GROUP NAME	LL %	PL %	PI %	NAT WC %	FINES %	< 2 µm %	TEST BY	REVIEW BY	ASTM DES
B-7, S-3	7.5	ML	Sandy Silt	**NP**	52	**NP**	43.4	58		AKV	JFL	D4318

CRP693 - West Cashmere Bridge
(Wenatchee River) Replacement
Chelan County, Washington

PLASTICITY CHART

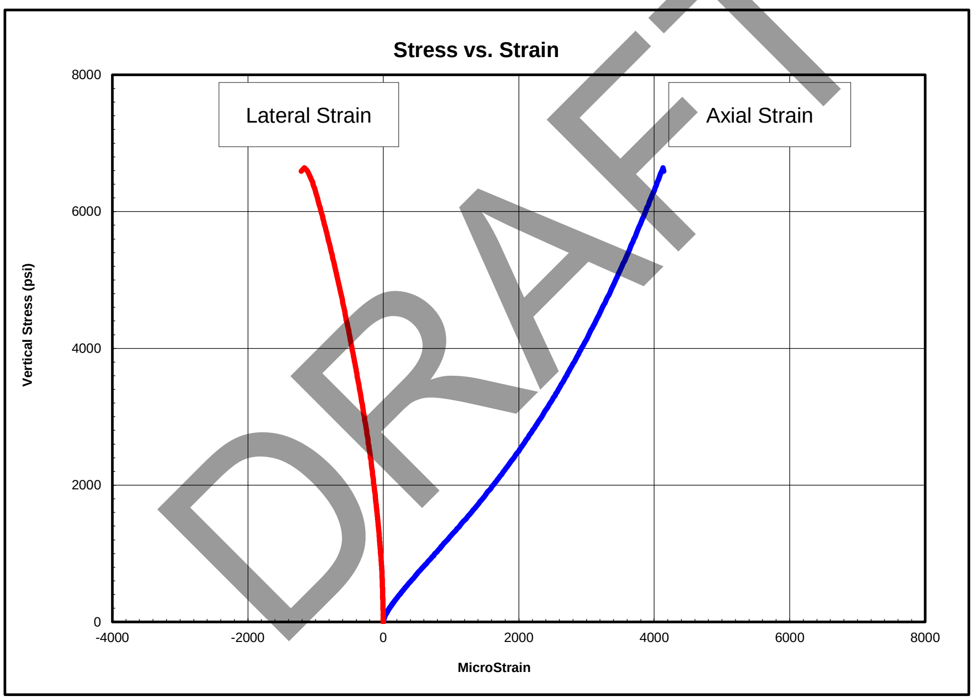
July 2016 22-1-03144

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-2
Sheet 1 of 1

Client:	Shannon & Wilson, Inc.
Project Name:	Cashmere Bridge
Project Location:	---
GTX #:	304704
Test Date:	5/12/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	B-2
Sample ID:	---
Depth, ft:	90.5
Sample Type:	rock core
Sample Description:	See photographs Intact material & Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 7,035 psi

The graph above does not include values up to the peak stress value. The strain gauges failed before the peak value was attained.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
700-2600	1,190,000	0.12
2600-4500	1,690,000	0.31
4500-6300	2,240,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Shannon & Wilson, Inc.	Test Date:	5/11/2016
Project Name:	Cashmere Bridge	Tested By:	daa
Project Location:	---	Checked By:	jsc
GTX #:	304704		
Boring ID:	B-2		
Sample ID:	---		
Depth:	90.5 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	5.48	5.48	5.48
Specimen Diameter, in:	2.41	2.41	2.41
Specimen Mass, g:	1054.28		
Bulk Density, lb/ft^3	160	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? NO

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? NO

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00020	0.00020	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00120	0.00110	0.00090	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010

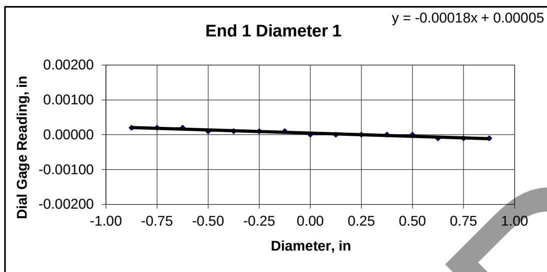
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00130

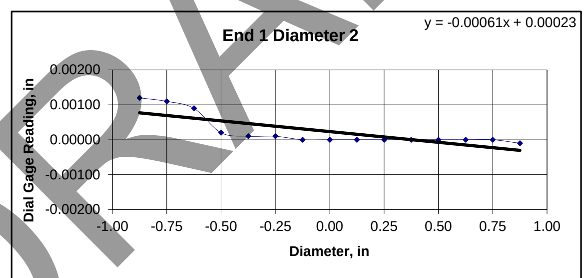
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00040	0.00040	0.00040	0.00030	0.00020	0.00020	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00030	-0.00040	-0.00050	-0.00080

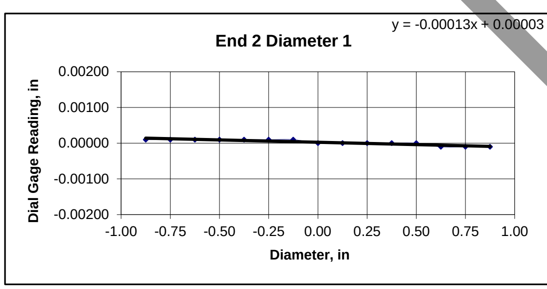
Difference between max and min readings, in:
0° = 0.0002 90° = 0.0012

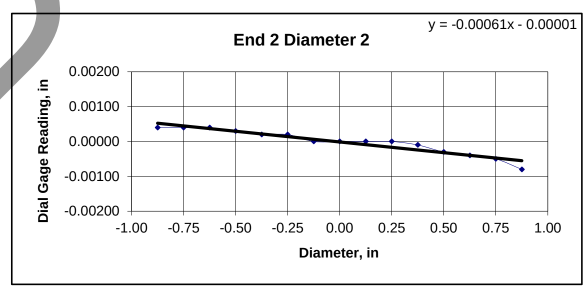
Maximum difference must be < 0.0020 in. Difference = ± 0.00065

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line -0.00018
Angle of Best Fit Line: -0.01031

End 2:
Slope of Best Fit Line -0.00013
Angle of Best Fit Line: -0.00745

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line -0.00061
Angle of Best Fit Line: -0.03495

End 2:
Slope of Best Fit Line -0.00061
Angle of Best Fit Line: -0.03495

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	2.410	0.00012	0.007	YES
Diameter 2, in (rotated 90°)	0.00130	2.410	0.00054	0.031	YES
END 2					
Diameter 1, in	0.00020	2.410	0.00008	0.005	YES
Diameter 2, in (rotated 90°)	0.00120	2.410	0.00050	0.029	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

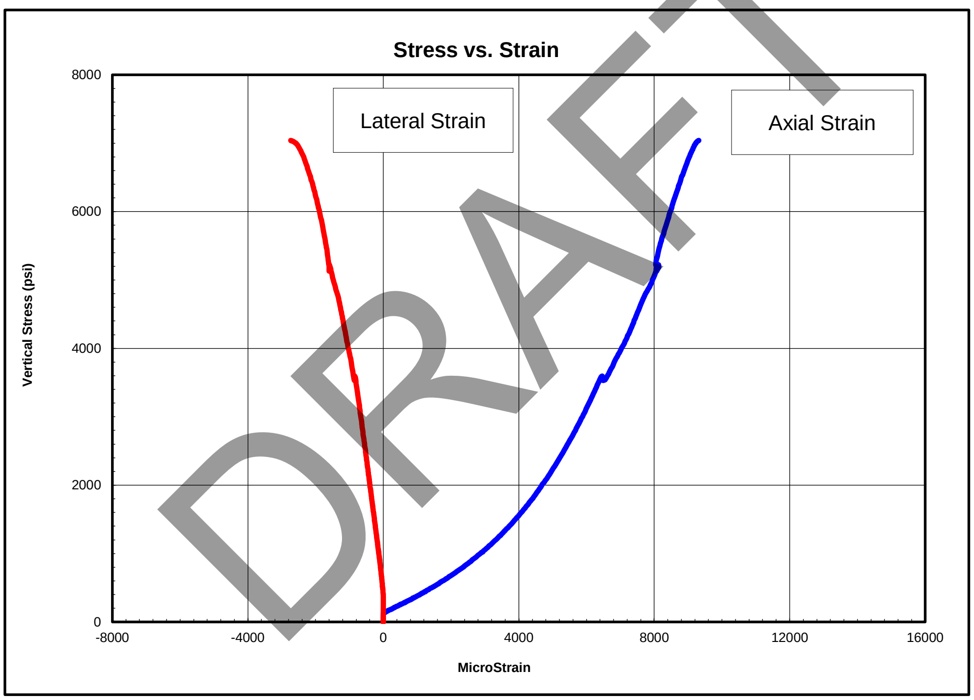
Client:	Shannon & Wilson, Inc.
Project Name:	Cashmere Bridge
Project Location:	---
GTX #:	304704
Test Date:	5/12/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	B-2
Sample ID:	---
Depth, ft:	90.5

After cutting and grinding

After break

Client:	Shannon & Wilson, Inc.
Project Name:	Cashmere Bridge
Project Location:	---
GTX #:	304704
Test Date:	5/12/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	B-4
Sample ID:	---
Depth, ft:	75.3
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 7,015 psi

One axial and one lateral strain gauge failed to record meaningful data. Young's Modulus and Poisson's Ratio reported based on results of a single axial and lateral strain gauge.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
700-2600	552,000	0.14
2600-4500	888,000	0.33
4500-6300	1,630,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Shannon & Wilson, Inc.	Test Date:	5/11/2016
Project Name:	Cashmere Bridge	Tested By:	daa
Project Location:	---	Checked By:	jsc
GTX #:	304704		
Boring ID:	B-4		
Sample ID:	---		
Depth:	75.3 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average		
Specimen Length, in:	5.37	5.37	5.37		
Specimen Diameter, in:	2.39	2.39	2.39		
Specimen Mass, g:	988.31				
Bulk Density, lb/ft^3	156			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.2			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00030	0.00030	0.00030	0.00020	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00020	-0.00030	-0.00040	-0.00050

Difference between max and min readings, in:

0° = 0.00030 90° = 0.00080

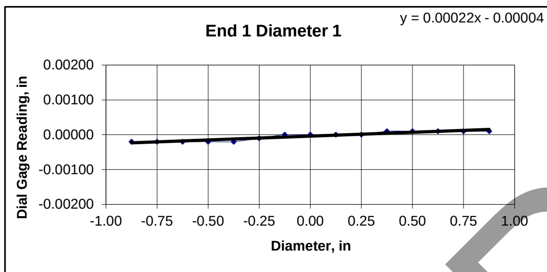
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00040	0.00040	0.00030	0.00020	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00020	-0.00030	-0.00040	-0.00040

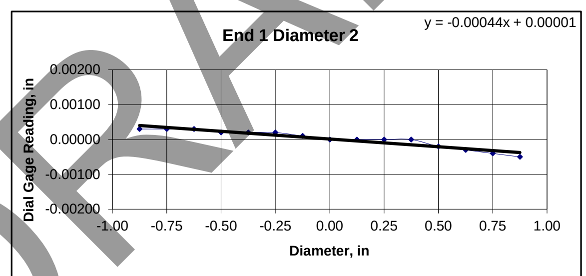
Difference between max and min readings, in:

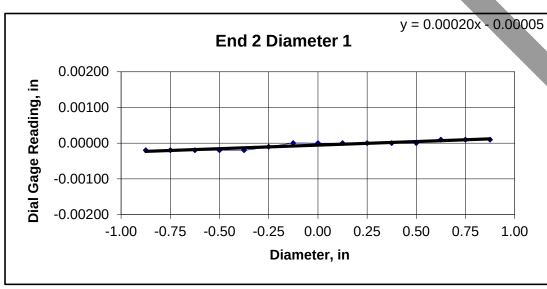
0° = 0.0003 90° = 0.0008

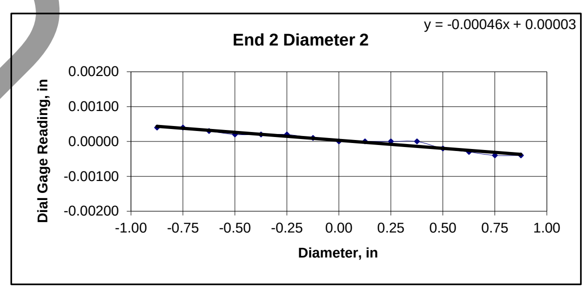
Maximum difference must be < 0.0020 in. Difference = ± 0.00040

Flatness Tolerance Met? YES









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00022
Angle of Best Fit Line:	0.01261
End 2:	
Slope of Best Fit Line	0.00020
Angle of Best Fit Line:	0.01146
Maximum Angular Difference:	0.00115

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	-0.00044
Angle of Best Fit Line:	-0.02521
End 2:	
Slope of Best Fit Line	-0.00046
Angle of Best Fit Line:	-0.02636
Maximum Angular Difference:	0.00115

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	2.390	0.00013	0.007	YES
Diameter 2, in (rotated 90°)	0.00080	2.390	0.00033	0.019	YES
END 2					
Diameter 1, in	0.00030	2.390	0.00013	0.007	YES
Diameter 2, in (rotated 90°)	0.00080	2.390	0.00033	0.019	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

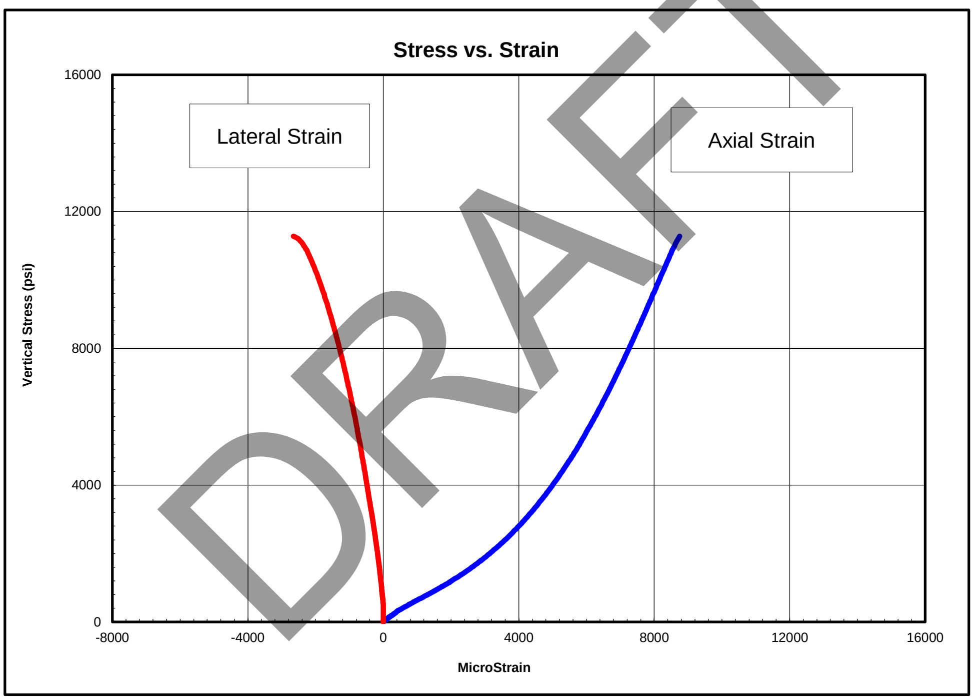
Client:	Shannon & Wilson, Inc.
Project Name:	Cashmere Bridge
Project Location:	---
GTX #:	304704
Test Date:	5/12/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	B-4
Sample ID:	---
Depth, ft:	75.3

After cutting and grinding

After break

Client:	Shannon & Wilson, Inc.
Project Name:	Cashmere Bridge
Project Location:	---
GTX #:	304704
Test Date:	5/12/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	B-5
Sample ID:	---
Depth, ft:	64.1
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 11,275 psi

One lateral strain gauge failed to record meaningful data. Poisson's Ratio reported based on results of a single lateral strain gauge.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1100-4100	920,000	0.13
4100-7100	1,720,000	0.33
7100-10200	2,210,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Shannon & Wilson, Inc.	Test Date:	5/11/2016
Project Name:	Cashmere Bridge	Tested By:	daa
Project Location:	---	Checked By:	jsc
GTX #:	304704		
Boring ID:	B-5		
Sample ID:	---		
Depth:	64.1 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average		
Specimen Length, in:	5.57	5.57	5.57		
Specimen Diameter, in:	2.40	2.40	2.40		
Specimen Mass, g:	1040.17				
Bulk Density, lb/ft^3	157			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90^o)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Difference between max and min readings, in:

0° = 0.00000 90° = 0.00010

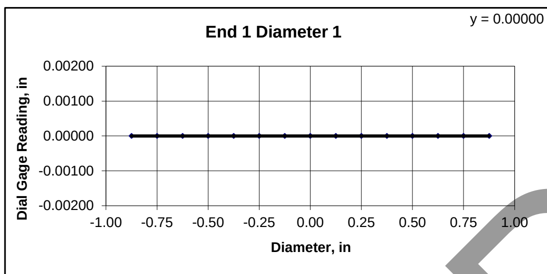
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90^o)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

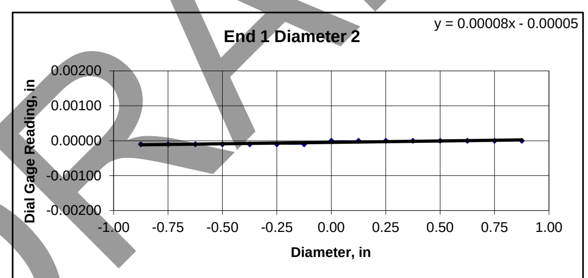
Difference between max and min readings, in:

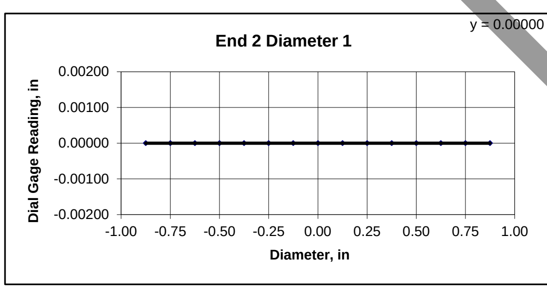
0° = 0 90° = 0.0001

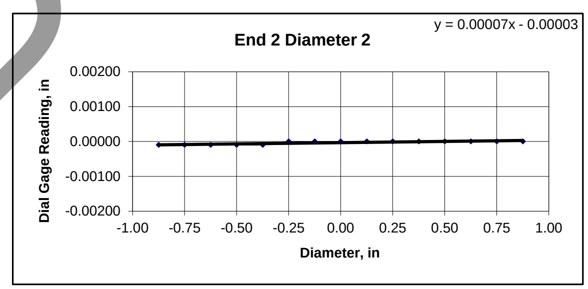
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00005

Flatness Tolerance Met? YES









DIAMETER 1

End 1:		
	Slope of Best Fit Line	0.00000
	Angle of Best Fit Line:	0.00000
End 2:		
	Slope of Best Fit Line	0.00000
	Angle of Best Fit Line:	0.00000
Maximum Angular Difference:		0.00000

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:		
	Slope of Best Fit Line	0.00008
	Angle of Best Fit Line:	0.00458
End 2:		
	Slope of Best Fit Line	0.00007
	Angle of Best Fit Line:	0.00401
Maximum Angular Difference:		0.00057

Parallelism Tolerance Met? YES

Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00000	2.400	0.00000	0.000	YES
Diameter 2, in (rotated 90^o)	0.00010	2.400	0.00004	0.002	YES
END 2					
Diameter 1, in	0.00000	2.400	0.00000	0.000	YES
Diameter 2, in (rotated 90^o)	0.00010	2.400	0.00004	0.002	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

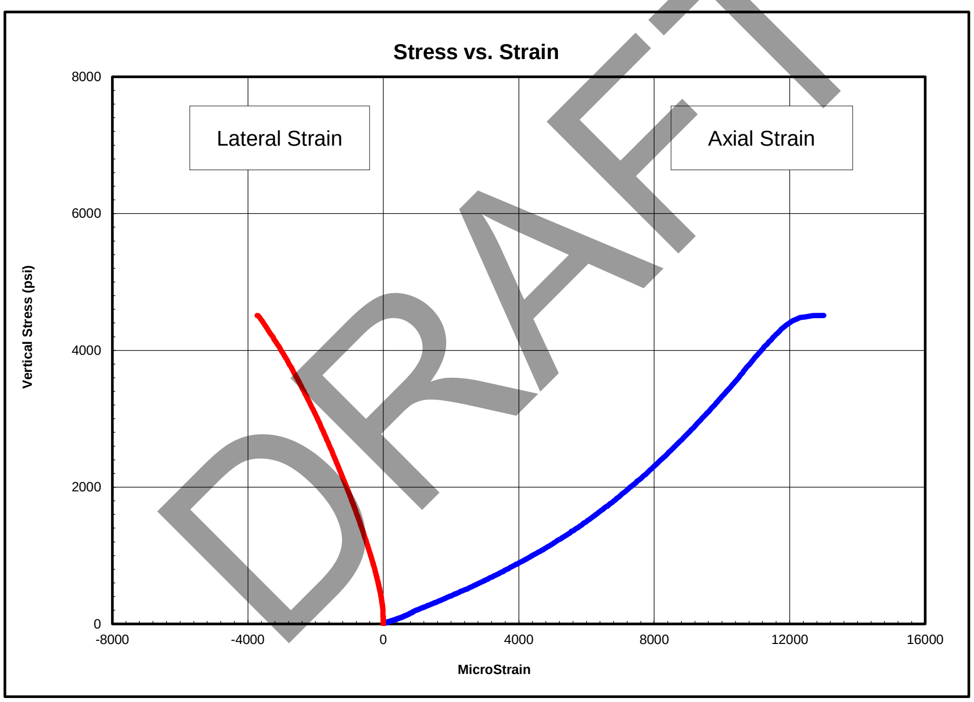
Client:	Shannon & Wilson, Inc.
Project Name:	Cashmere Bridge
Project Location:	---
GTX #:	304704
Test Date:	5/12/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	B-5
Sample ID:	---
Depth, ft:	64.1

After cutting and grinding

After break

Client:	Shannon & Wilson, Inc.
Project Name:	Cashmere Bridge
Project Location:	---
GTX #:	304704
Test Date:	5/12/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	B-5
Sample ID:	---
Depth, ft:	70
Sample Type:	rock core
Sample Description:	See photographs Intact material and Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 4,510 psi

One lateral strain gauge failed to record meaningful data. Poisson's Ratio reported based on results of a single lateral strain gauge.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
500-1700	286,000	0.18
1700-2900	450,000	0.37
2900-4100	569,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Shannon & Wilson, Inc.	Test Date:	5/11/2016
Project Name:	Cashmere Bridge	Tested By:	daa
Project Location:	---	Checked By:	jsc
GTX #:	304704		
Boring ID:	B-5		
Sample ID:	---		
Depth:	70 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average		
Specimen Length, in:	5.51	5.51	5.51		
Specimen Diameter, in:	2.41	2.41	2.41		
Specimen Mass, g:	1024.52				
Bulk Density, lb/ft^3	155			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00030	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00020
Diameter 2, in (rotated 90^o)	0.00020	0.00020	0.00020	0.00020	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00030	-0.00040

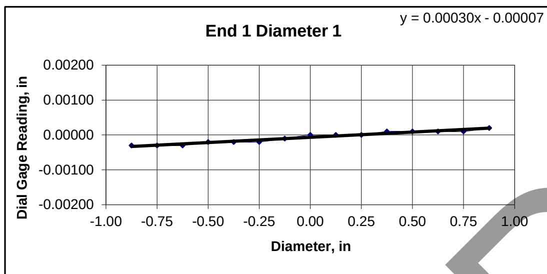
Difference between max and min readings, in:
0° = 0.00050 90° = 0.00060

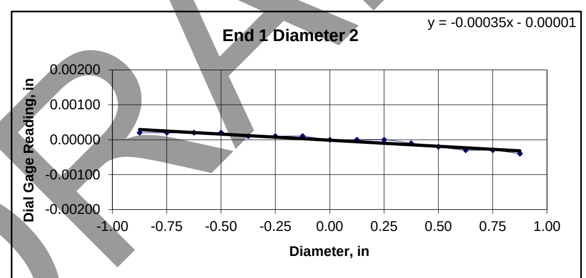
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00020	0.00030
Diameter 2, in (rotated 90^o)	0.00020	0.00020	0.00020	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00040

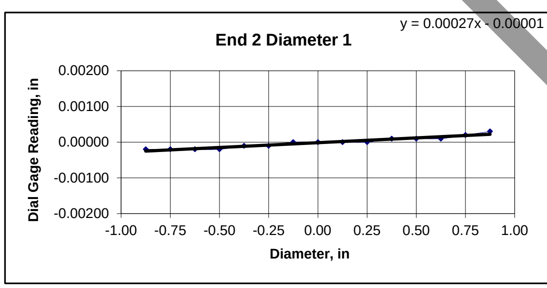
Difference between max and min readings, in:
0° = 0.0005 90° = 0.0006

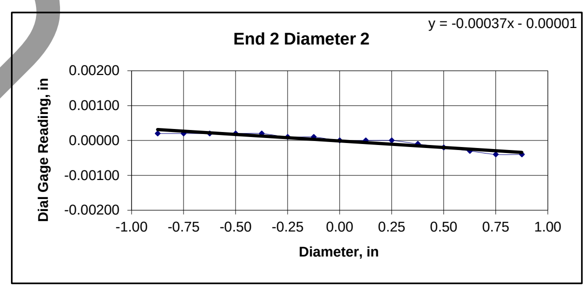
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00030

Flatness Tolerance Met? YES









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00030
Angle of Best Fit Line:	0.01719

End 2:

Slope of Best Fit Line	0.00027
Angle of Best Fit Line:	0.01547

Maximum Angular Difference: 0.00172

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	-0.00035
Angle of Best Fit Line:	-0.02005

End 2:

Slope of Best Fit Line	-0.00037
Angle of Best Fit Line:	-0.02120

Maximum Angular Difference: 0.00115

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	2.410	0.00021	0.012	YES
Diameter 2, in (rotated 90^o)	0.00060	2.410	0.00025	0.014	YES
END 2					
Diameter 1, in	0.00050	2.410	0.00021	0.012	YES
Diameter 2, in (rotated 90^o)	0.00060	2.410	0.00025	0.014	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Shannon & Wilson, Inc.
Project Name:	Cashmere Bridge
Project Location:	---
GTX #:	304704
Test Date:	5/12/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	B-5
Sample ID:	---
Depth, ft:	70

After cutting and grinding

After break

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

Attachment to and part of Report 22-1-03144-001

Date: July 2016
To: Mr. Kash Nikzad, Ph.D., P.E.
TranTech Engineering, LLC

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

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

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

READ RESPONSIBILITY CLAUSES CLOSELY.

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

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

Appendix C - Hydrology & Geomorphology Technical Memo

NHC Ref. No. 2001295

29 June 2016

Trantech Engineering LLC
12011 NE First St Suite 305
Bellevue, WA
98005

Attention: **Kash Nikzad, PHD, PE**
Principal

Via email: knikzad@trantecheng.com

Re: **CRP693 - W Cashmere Bridge Replacement Project**
Draft TS&L – Preliminary Hydraulic Technical Analysis

Dear Mr. Nikzad:

This letter report summarizes the assumptions and results of a preliminary hydraulic analysis conducted by Northwest Hydraulic Consultants (NHC) for the subject project. The preliminary hydraulic results will assist the Trantech team in furthering the design of Chelan County's West Cashmere Bridge Replacement project.

1 INTRODUCTION AND PURPOSE

This letter report documents the preliminary engineering analysis completed for the West Cashmere Bridge (Bridge 401) over the Wenatchee River, herein called the West Cashmere Bridge, to determine the basis for design. The preliminary analysis includes a hydrologic and hydraulic analysis, which are discussed herein.

The West Cashmere Bridge crossing on Goodwin Road is located in Chelan County, WA crossing the Wenatchee River just west of City of Cashmere. The Wenatchee River flows from west to east in the project vicinity before it empties into the Columbia River approximately 10.7 river miles downstream. The existing West Cashmere Bridge travels northwest to southeast and consists of eight spans with two piers within the 100-year water surface elevation.

2 HYDROLOGIC ANALYSIS

The Wenatchee watershed drains approximately 1,188 square miles at the West Cashmere Bridge crossing (Figure 1). The watershed's maximum elevation is 9,370 feet above mean sea level with a mean basin elevation and annual precipitation of 800 feet above mean sea level and 66.7 inches, respectively.

The U.S. Geological Survey (USGS) operates two stream gages applicable to the site; Wenatchee River at Monitor Gage (USGS 12462500) located approximately 4.3 river miles downstream and Wenatchee River at Peshatin (USGS 12459000) located 9 river miles upstream of the West Cashmere Bridge crossing.

Flow data from the USGS Wenatchee River gages were evaluated utilizing the U.S. Army Corps of Engineers (USACE) Statistical Software Package (HEC-SSP), developed by the Hydrologic Engineering Center. The software allows the user to perform a statistical analysis of the hydrological data. Utilizing the HEC-SSP software, a USGS Bulletin 17B flow frequency analysis was performed and flood flow frequency curves were developed for the two USGS gages. Table 1 summarizes the calculated peak flows and the peak flows from 2001 Flood Insurance Study (FIS) for Wenatchee River.

Table 1. Peak Flows for Wenatchee River at USGS Gages

Mean Recurrence Interval (MRI)	Wenatchee River at Monitor USGS Gage #12462500 (cfs)	Wenatchee River at Peshastin USGS Gage #12459000 (cfs)	USACE 2001 FIS at Monitor USGS Gage #12462500 (cfs)
2-Year	17,160	15,920	-
10-Year	28,135	24,635	**26,500**
20-Year	33,495	28,205	-
50-Year	41,545	33,050	**38,500**
100-Year	48,510	36,880	**48,700**
500-Year	56,360	40,875	**82,000**

In addition to peak flows, the project team requested flows that may be encountered during construction. Based on a fish window from July 1st – August 15th (assumed construction window for in-water work), NHC conducted a flow duration analysis using mean daily flows form the USGS Wenatchee River at Monitor Gage (#124625000). The gage recorded mean daily flows from 1962 through present (54 years). Table 2 shows the flow exceedance values for the 1-, 5-, 10-, 50-, and 95-percent exceedance within the assumed July 1st – August 15th construction window.

Table 2. Flow Exceedance Values for July 1st through August 15th at Wenatchee River Monitor Gage

Percent Exceedance	Mean Daily Flow (cfs)
1-Percent	11,400
5-Percent	8,640
10-Percent	7,080
50-Percent	2,300
95-Percent	615

3 HYDRAULIC ANALYSIS

The hydraulic analysis of the existing and proposed West Cashmere Bridge Crossings was performed utilizing the USACE HEC-RAS 5.0.1 computer program, a one-dimensional gradually varied steady flow numerical model. Four unique alternatives were provided by the Trantech team which were analyzed with the HEC-RAS model, namely: 1) existing conditions; 2) a two-span cable stay; 3) a three-span steel girder; and 4) a four-span concrete girder.

The channel and floodplain geometry data for the model was obtained from topographic surveys of the site provided by Chelan County. Topographic data was submitted to NHC in electronic files formatted for AutoCAD Civil 3D. Cross section locations and extents for the HEC-RAS model were laid out on the topographic drawing using the standard requirements for one-dimensional hydraulic model development (e.g. cross section oriented perpendicular to flow direction, cross section extending to or beyond limits of effective flow, maximum spacing between cross sections to keep EGL changes less than one foot, cross sections located to capture all unique channel and floodplain changes, etc.). The geometry of each cross section was then obtained from AutoCAD Civil 3D; the program established ground elevations along the length of the cross section based on the location of the cross section on the digital terrain model created from the survey.

To obtain hydraulic characteristics within the project reach, the model geometry extends approximately 1,650 feet downstream and about 1,250 feet upstream of the existing West Cashmere bridge. The upstream and downstream boundary locations are sufficiently far enough away from the project site to not influence hydraulics at the West Cashmere Bridge .

Hydraulic roughness values in the model were estimated based on observed site conditions and standard values for those conditions listed in engineering textbooks. Expansion and contraction loss coefficients were set to standard values, and are higher at significant flow contractions and expansions such as immediately upstream and downstream of a structure, as recommended by the HEC-RAS User's Manual. The hydraulic model was run with a sub-critical flow regime, which matches the flow regime of Wenatchee River at and near the project site.

3.1 Existing Conditions

The existing condition model includes the existing West Cashmere Bridge geometry. A Manning's n value (i.e. a coefficient estimating channel roughness) for the natural stream channel upstream and downstream of the project site was determined to be 0.04. Manning's n for the overbank areas which includes the existing grasses, shrubs and trees was estimated as 0.05.

The Wenatchee River along the project site is in a FEMA regulated Zone AE Floodway with effective baseflood elevations (BFE) for the 1% annual recurrence (100-year) flow (Figure 2). Therefore any fill or structure placed within the floodway must meet the no-rise requirement. Determination of meeting the zero-rise requirement was important for this preliminary phase of the project, therefore the USACE's 2001 FIS 100-year event peak flow of 48,700 cfs was used to evaluate water surface elevations for the proposed alternatives.

The 100-year event was calibrated to match the BFE from the 2001 USACE FIS at the upstream and downstream boundary conditions. NHC determined the 2001 USACE study was based on limited bathymetric data, where the USACE assumed the channel bed was flat across the entire bed width. This assumption resulted in higher water surface elevations (approximately 2.5 feet) in the vicinity of the bridge as compared to existing conditions developed based on the topographic and bathymetric data provided by Chelan County. Table 3 shows the water surface elevations and the velocities at the upstream bridge face during the 2-year and 100-year peak flows and the flow exceedance values for the 1-, 5-, 10-, 50-, and 95-percent exceedance within the assumed July 1st – August 15th construction window .

Table 3. Water Surface Elevations and Velocities for Existing Conditions

Mean Recurrence Interval (MRI) / Exceedance	Water Surface Elevation (Feet, NAVD88)	Channel Average Velocity (Feet/Second)
2-Year	800.7	8.4
100-Year	808.7/811.2*	13.4/13.5*
1-Percent	799.1	7.1
5-Percent	798	6.2
10-Percent	797.2	5.4
50-Percent	794.1	2.8
95-Percent	792.1	1.2

Water Surface Elevations and Velocities at Upstream Bridge Face (HEC-RAS XS 3034)
*Water Surface Elevation and Velocity from 2001 USACE Study

3.2 Proposed Alternatives Conditions

The future conditions scenario simulated with HEC-RAS modified the existing model geometry to include the proposed West Cashmere bridge alternatives provided by the Trantech team. All three proposed bridge alternatives will have the bridge decks sufficiently above the 100-year maximum water level, therefore will have sufficient freeboard. Bridge piers were assumed to consist of 5-foot columns with 9-foot shafts, where the shafts were assumed to be 2 feet below the existing river bed. The main difference of each proposed alternative from a hydraulics perspective is the number of in-water piers. The two-span, three-span, and four-span proposed bridges have zero, one and two in-water piers, respectively. Cross sections outside of the proposed bridge alternatives were not altered from existing conditions. The Manning's n value for the stream channel is the same as the calibrated existing condition model. All three proposed alternatives lower the 100-year water surface elevation as compared to existing conditions, therefore meeting zero-rise criteria. The decrease varies depending on the bridge alternative and the corresponding number of piers that will be in contact with 100-year flow. Table 4 summarizes the 100-year water surface elevation and cross sectional average velocity at the upstream bridge face.

Table 4. 100-Year Water Surface Elevations and Velocities for Proposed Conditions

Proposed Alternative	100-Year Water Surface Elevation (Feet, NAVD88)	100-Year Channel Average Velocity (Feet/Second)
2-Span Bridge	808.2	14.0
3-Span Bridge	808.4	13.8
4-Span Bridge	808.6	13.7

Water Surface Elevations and Velocities at Upstream Bridge Face (HEC-RAS XS 3034)

4 SUMMARY

Based on the preliminary hydrologic and hydraulic analysis described in this letter report, all three bridge alternatives should meet zero-rise criteria, the WDFW 2013 Water Crossing Design Guidelines and WAC 220-660-190. Due to high flows, subsurface soils have not been collected in the area of the proposed in-water pier(s), which will assist in determining depths of scour for the proposed alternatives. We plan on conducting a site visit to collect pertinent data to support the team as the design progresses and support the need to collect geotechnical data at the proposed in-water pier(s) to be able to refine depth of scour calculations.

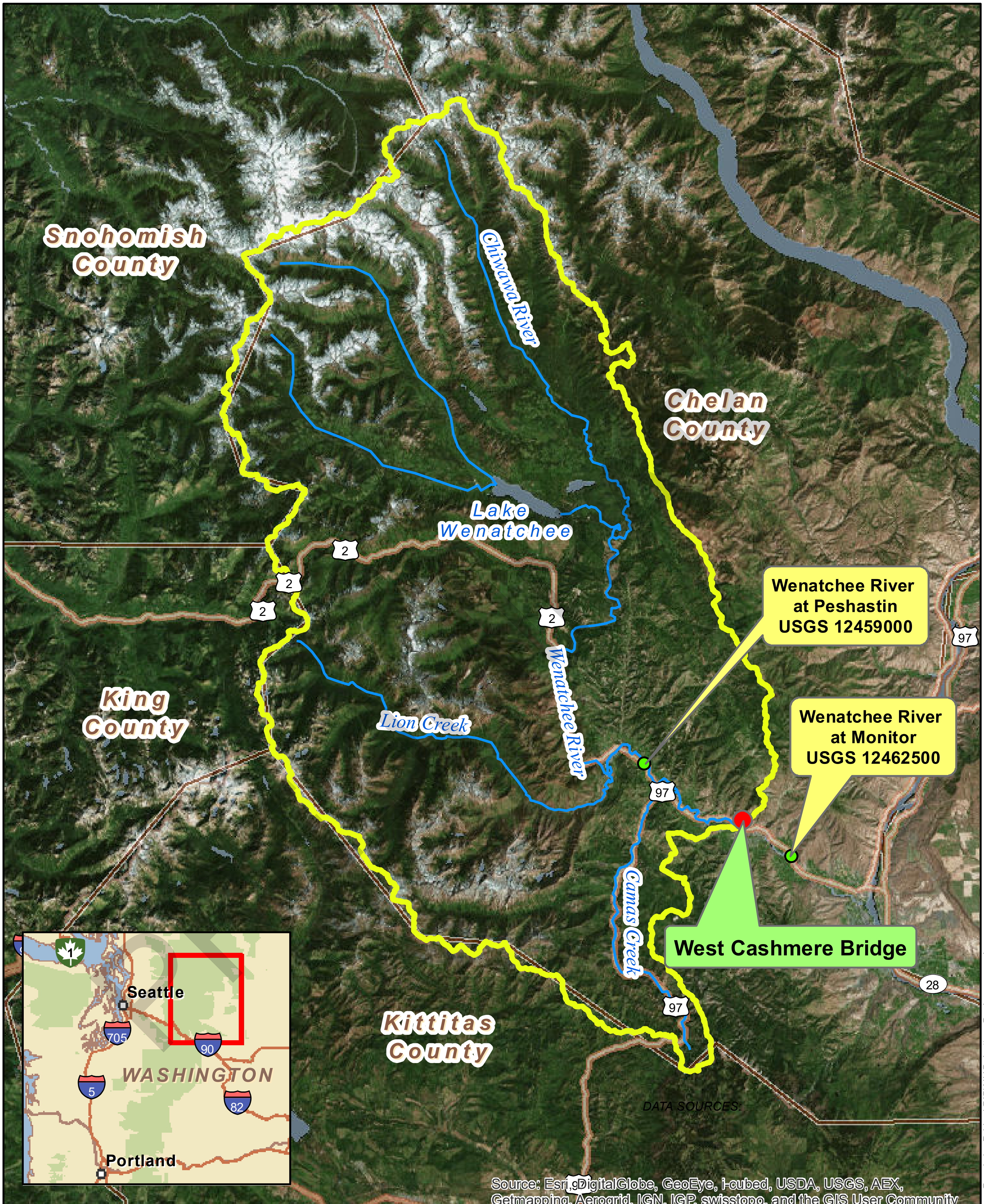
If you have any questions regarding the analysis, please feel free to contact me at ckramer@nhcweb.com or 360-584-9810.

Sincerely,

Casey Kramer

Casey Kramer, PE
Principal Engineer

Attachments: Figure 1 – Watershed Map
Figure 2 – Flood Insurance Rate Map



Chelan County	*SCALE - 1:506,000* 0 2 4 6 8 *Miles* N	*Job: 2001295* *Date: JUNE 2016*	DRAFT *West Cashmere Bridge at Wenatchee River Watershed Map*
nhc northwest hydraulic consultants	Units: US Survey Feet	*FIGURE 1*	

ABC, P:\2001295-WCashmere_Goodwin_Bridge\GIS\NHC_Letter_Cashmere.mxd

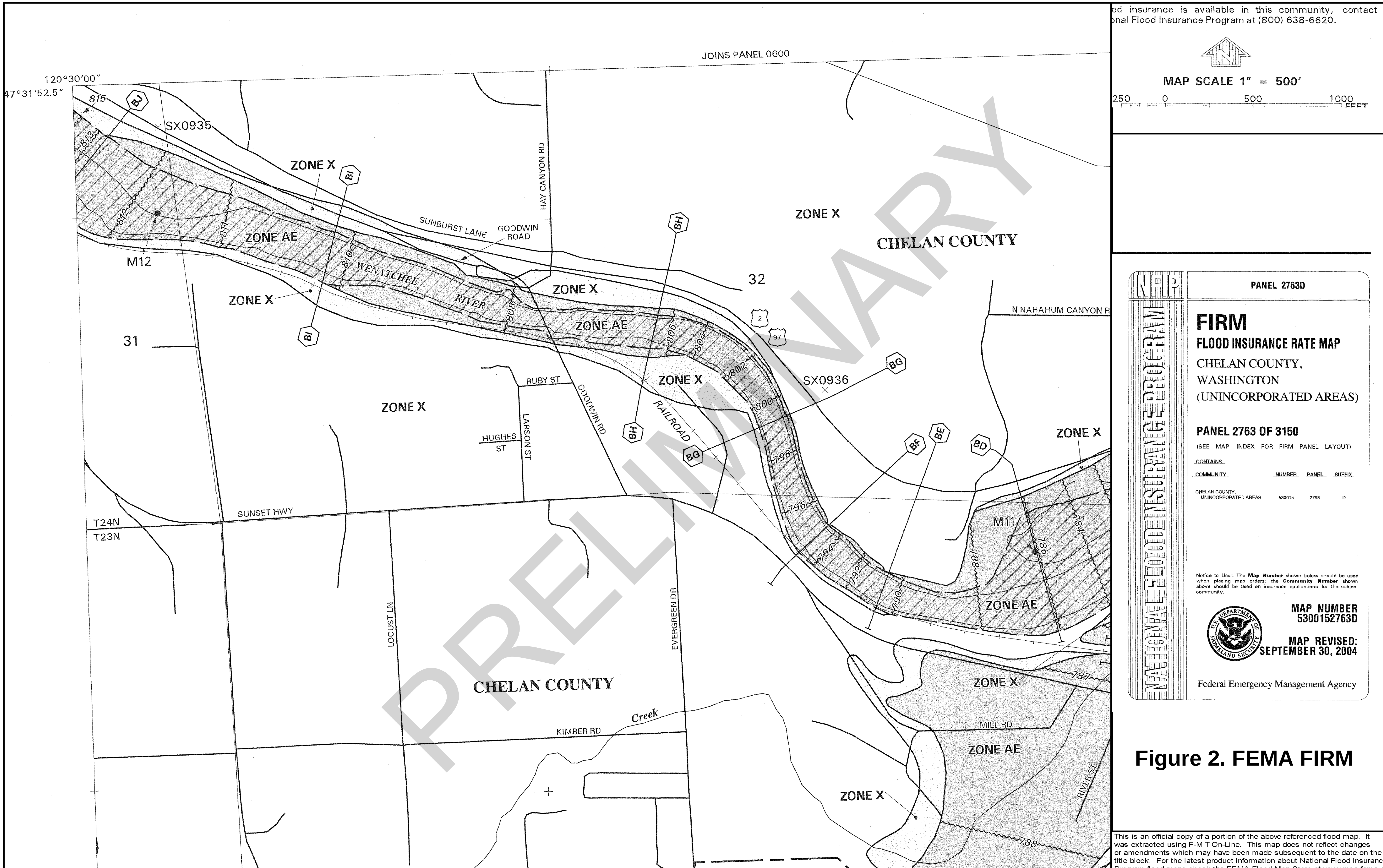


Figure 2. FEMA FIRM

Appendix D - Mobility of Traffic Technical Memo

MEMORANDUM

Date:	June 28, 2016	**TG:**	15371.00
To:	Kash Nikzad, PhD, PE		
From:	Ryan Peterson, PE, PTOE		
cc:			
Subject:	Cashmere Bridge Replacement – Design Study Alternatives Evaluation		

Per request, this memorandum summarizes the findings of an alternatives analysis with regards to construction traffic control and traffic operations. Alternatives evaluated include the following:

- Alignment A – This alignment includes demolition of the existing bridge and utilizing the existing alignment for new construction:
  - 2-Span cable stayed bridge
  - 3-span steel plate girder bridge
  - 4-span concrete girder bridge
- Alignment B – This alignment would maintain the existing bridge during the construction of the new bridge just east of the existing alignment:
  - 2-Span cable stayed bridge
  - 3-span steel plate girder bridge
  - 4-span concrete girder bridge

Alignment A

Detours

Alignment A will involve the closure of the existing bridge and will therefore require temporary detours for bridge traffic. Eastbound traffic on US 2 and southbound traffic on Hay Canyon Rd would generally be routed as follows:

- East to Aplets Way / US 2 intersection
- Turn right onto Aplets Way
- Continue straight onto N Division St
- Continue straight onto S Division St
- Turn right onto Sunset Hwy
- End detour at Goodwin Rd

Westbound traffic on US 2 would generally be routed as follows:

- West to Johnson Rd / US 2 intersection
- Turn left onto Johnson Rd
- Turn left onto Stines Hill Rd
- Continue straight onto Sunset Hwy
- End detour at Goodwin Rd

Northbound traffic on Goodwin Rd would generally be routed as follows:

- North to Ruby St / Goodwin Rd intersection
- Turn left onto Ruby St
- Turn left onto Larson St

- Turn left onto Sunset Hwy
- Turn left onto S Division St
- Continue straight onto N Division St
- Continue straight onto Aplets Way
- End detour at US 2
-

Temporary Traffic Control

The options for temporary traffic control vary depending on what type of bridge is constructed. Anticipated pier locations are shown in the attached exhibit.

2-Span Cable Stay Bridge

The advantage of the 2-Span Cable Stay bridge is that US 2 could remain open through the duration of the project. As can be seen in the attached exhibit, the 2-Span Cable Stay bridge is anticipated to need a pier that would directly conflict with the access road that connects the existing bridge to US 2. However, under Alignment A, the existing bridge would be closed and no traffic would be utilizing this access road.

3-Span Steel Plate Girder Bridge

The 3-Span Steel Plate Girder bridge would require temporary closure of US 2 in order to place girders over the roadway. The temporary closure will require a detour route for US 2 traffic. One option for detouring traffic is to utilize the access road that connects the existing bridge with US 2. Minor grading and the construction of temporary pavement could provide a temporary route south of the existing US 2 alignment. This option and alignment would require a pier to be constructed between US 2 and the access road but the location of this pier is not anticipated to negatively impact the ability to provide the temporary route.

4-Span Concrete Girder Bridge

A 4-Span Concrete Girder bridge would require the same temporary detour route mentioned above for the 3-Span Steel Plate Girder bridge. The proposed pier locations for this bridge type would not negatively impact the ability to provide this route.

Alignment B

Detours

Alignment B would generally maintain traffic across the existing bridge and would require no long term detours. Short duration closures and detours would be required to transition the alignment to the new bridge. These temporary detours would be similar as those described above for Alignment A.

Temporary Traffic Control

The options for temporary traffic control vary depending on what type of bridge is constructed.

2-Span Cable Stay Bridge

The 2-Span Cable Stay Bridge has the same advantage under Alignment B as Alignment A in that no closure of US 2 would be required for construction. Similarly, based on the proposed location of

piers for this alignment and bridge type, it is anticipated that the access road connecting the existing bridge to US 2 would be impacted. Movements utilizing this access road include those heading northbound on Godwin Rd (to US 2) and southbound on Hay Canyon Rd (to Goodwin Rd). Entering vehicles from the west would not be impacted.
A possible mitigation to this impact would include realigning the north approach to the existing bridge so that the geometry of the approach would exit the bridge, curve fairly sharply to the south, then curve again to the north to intersect US 2 at an angle that would allow full or partial access to US 2. While this option would provide an alternative route for existing bridge traffic, the costs of re-grading the existing bridge approach is anticipated to be substantial. In addition, it is likely that a temporary signal would need to be installed at this intersection and tied to the existing signal at Hay Canyon Road.

<u>3-Span Steel Girder Bridge</u>

The 3-Span Steel Girder bridge built on Alignment B would require a temporary closure of US 2, similar to that described for Alignment A. However, with the existing bridge in use, the option of providing a temporary detour route utilizing the access road that connects the existing bridge to US 2 would not be possible; therefore alternative detour routes would be required. The anticipated route for eastbound US 2 traffic would include the Johnson Rd intersection, Stine Hill Road/Sunset Highway, then utilizing the Aplet's Way bridge and/or the Cottage Ave. bridge. The reverse of this route would be used for westbound US 2 traffic. This route is approximately 5 miles in length and would result in US 2 traffic be rerouted directly through the City of Cashmere. This would most likely only be feasible for night closures.

<u>4-Span Concrete Girder Bridge</u>

The 4-Span Concrete Girder bridge on Alignment B would require the same closure of US 2 with associated impacts as described above for the 3-Span Steel Girder bridge.

Emergency Vehicles Access

Cashmere does not have a hospital with emergency room. The nearest hospitals are Cascade Medical Center, located in Leavenworth, and Central Washington Hospital, located in Wenatchee. The project location is located almost equidistance from each hospital at 10.5 miles from Cascade Medical Center, and 14 miles from Central Washington Hospital. Given the hospitals' locations to the project, and their emergency responders probable use of US-2 as an emergency route, it is likely they utilize the current bridge to access Goodwin Road and its immediate surrounding area. In the event the existing bridge and/or US-2 is closed to through traffic (short term or long term) at the project site, the contractor is required to halt operations and allow emergency access through the project site. If providing a route through the project site is not feasible, emergency responders for the hospitals can utilize marked detour routes.

The nearest fire department to the project location is Cashmere Fire Department, which is located at 101 Woodring St in Cashmere. It is the only fire department in Cashmere, and is located to provide easy access to US-2 via Aplets Way or Cottage Avenue. Additionally, it has easy access to Sunset Hwy via Division St. Given the current geometry of the existing bridge and Goodwin Road, and the likely emergency routes, it is unlikely Cashmere Fire Department utilizes the current bridge. In the event US-2 is closed to through traffic (short term or long term) at the project site, the contractor is required to halt operations and allow emergency access through the project site via US-2. If providing a route through the project site is not feasible, Cashmere Fire Department can utilize marked detour routes, or Chelan County Fire District #3, located in Leavenworth, may be able to respond.

Appendix E – Roadway Alignment & Profile Technical Memo

TECHNICAL

Memorandum

Client: Trantech

Project: Chelan County – West Cashmere Bridge Replacement

Project File: TRAN 216.054.01.102 **Project Manager:** Angi Waligorski, P.E.

Composed by: Angi Waligorski, P.E./ Michael Voth, P.E.

Reviewed by: Randy Asplund, P.E.

Subject: Preliminary Roadway Design Factors

Date: June 28, 2016

Date Signed:
6/xx/2016

Date Signed:
6/xx/2016

This technical memorandum summarizes the assumptions for establishing the preliminary roadway elements for the West Cashmere Bridge Replacement project.

Alignment

RH2 Engineering, Inc., (RH2) prepared two alignments for the West Cashmere Bridge replacement project. The first alignment, Alignment A, is centered on the existing bridge. The second alignment, Alignment B, is to the east of the existing bridge with approximately 10 feet between the existing bridge and the new bridge to allow room for construction. The easterly alignment would allow traffic to use the existing bridge through most of construction, but would have the greatest impact to property owners on the south end of the bridge. Both alignments were constrained with tying into the private road on the north end. The design speed for Alignment A is 25 miles per hour (mph). Alignment B has a design speed between 15 to 20 mph because of the tighter curve required to tie into the private road. These alignments can be seen on **Figure 1**.

Consideration was given to tying in further north along Hay Canyon to increase the design speed for Alignment B, but Chelan County (County) stated that it was their desire to have this alignment tie into Hay Canyon across from the private road.

Profile

Along previously-described Alignment A, profiles were created for a 7-foot and 10.5-foot girder depth. Profiles were controlled by the clearance needed over the Burlington Northern & Santa Fe (BNSF) railroad, US Highway 2, and the frontage road. A 30-mile-per-hour design speed was used for vertical curves and a maximum grade of 7 percent was used. These profiles are show on **Figures 2** and **3**.

A minimum clearance of 16 feet, 10 inches was used over US 2 and the frontage road. A minimum clearance of 23.5 feet was used between the top of the railroad rail and the bottom of the bridge. It was assumed that any future tracks would be at the same elevation as the existing. The envelope, where this clearance is required, extends between a 45-foot southerly offset and a 25-foot northerly offset from the centerline of the existing tracks. This clearance envelope is shown on **Figure 4**.

It should be noted that the existing bridge does not provide the required clearance at the railroad, therefore any of the options will be higher than the existing bridge. This will result in impacts to south-end properties and to the intersection at Ruby Street. Slopes can be minimized using retaining walls, but access to the properties will be an issue because of the grade difference.

Roadway Section

The roadway section used for the project is two 11-foot lanes, two 5-foot shoulders, and a 10-foot sidewalk separated by a barrier. Bridge rails on either side bring the total width of the bridge to 47 feet.

The preliminary design assumes the bridge will be on a 2-percent cross slope so the bridge will drain west to east. A maximum 4-percent superelevation table was used for the preliminary design of the curve at the north end of the bridge. Because the curve is so close to the bridge, the superelevation transition will likely need to begin on the bridge.

Bridge Abutments

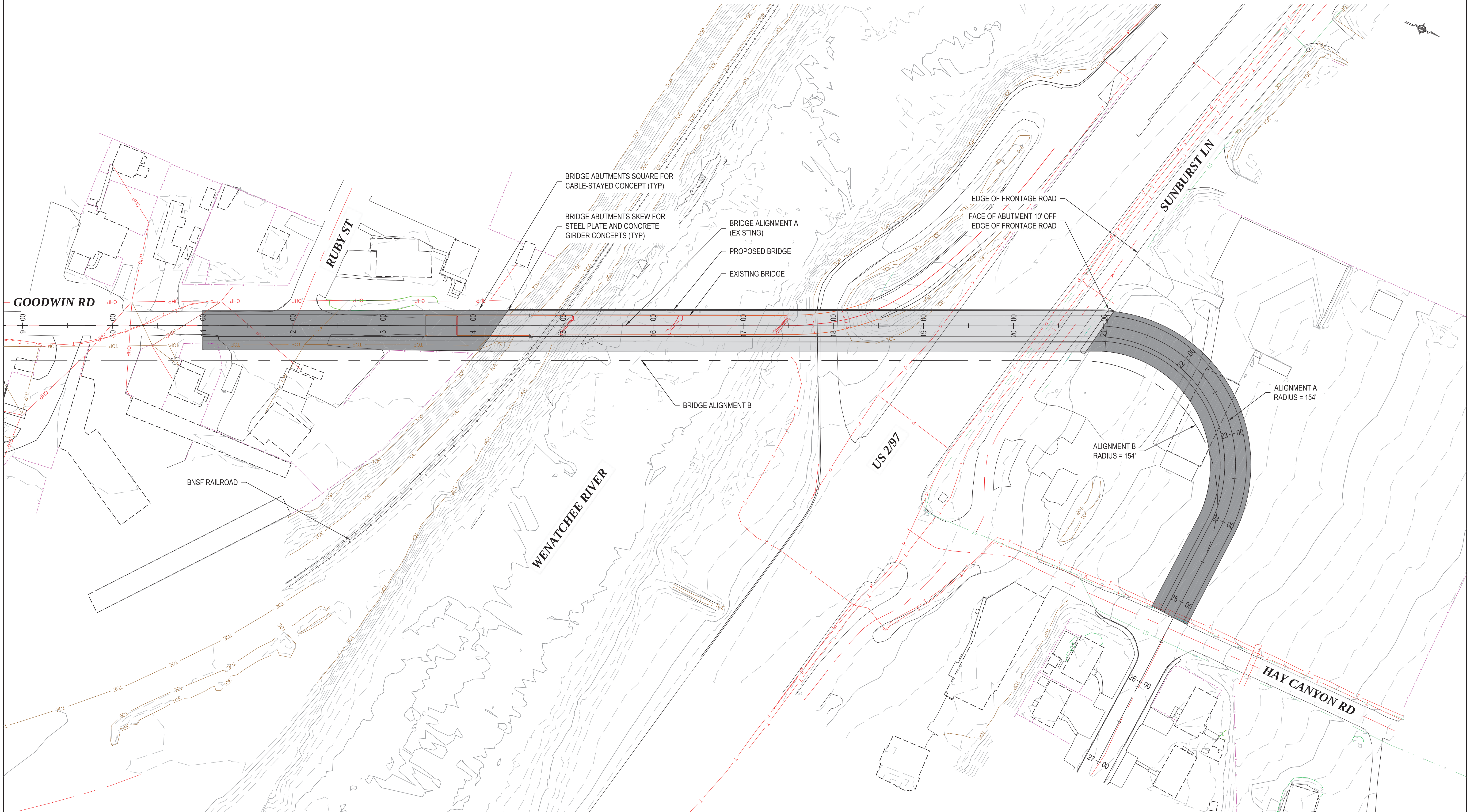
The north-end bridge abutment was set based on the location of the frontage road. Based on Exhibit 1600-2 in the Washington State Department of Transportation Design Manual, the clear zone is 10 feet from the edge of travelled way. For the preliminary design, the abutment was set 10 feet off the edge of the frontage road pavement. The location of the southerly abutment was set at a 45 foot offset from the centerline of the existing rail to accommodate future tracks.

Attachments:

1. Figure 1 – Bridge Alignment Exhibit
2. Figure 2 – 7-foot deep bridge Preliminary Profile
3. Figure 3 – 10.5-foot deep Preliminary Profile
4. Figure 4 – BNSF Crossing Exhibit

Figures

Figure 1



BRIDGE ALIGNMENT PLAN

1" = 50'

WEST CASHMERE BRIDGE REPLACEMENT

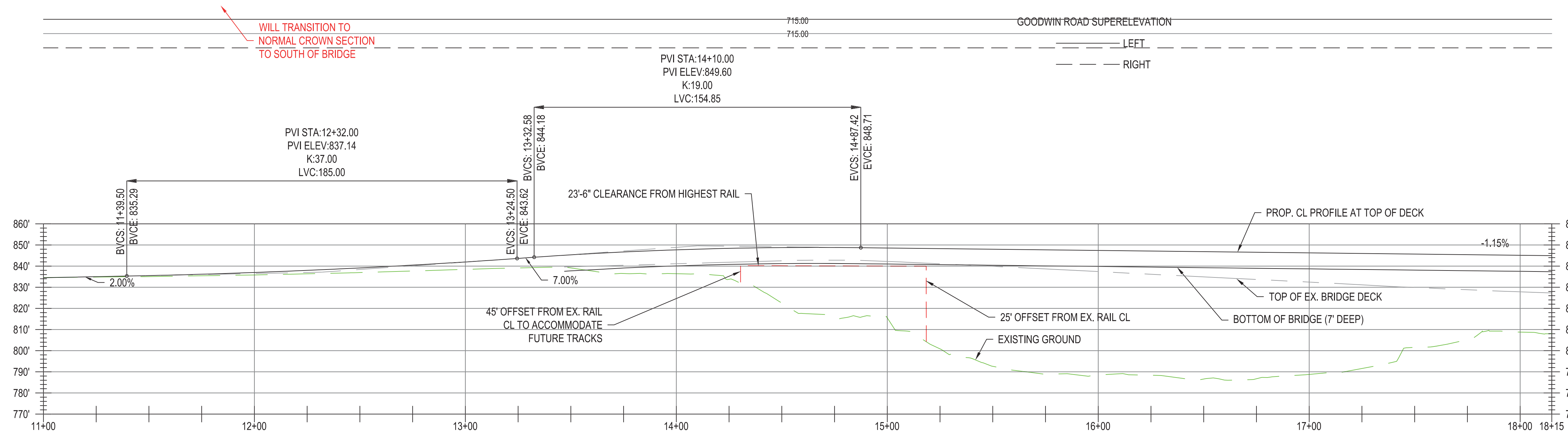
FIGURE 1
BRIDGE ALIGNMENT EXHIBIT

RH2 ENGINEERS PLANNERS SCIENTISTS

0 1" 2"
DRAWING IS NOT TO SCALE IF BAR IS NOT 2" LONG

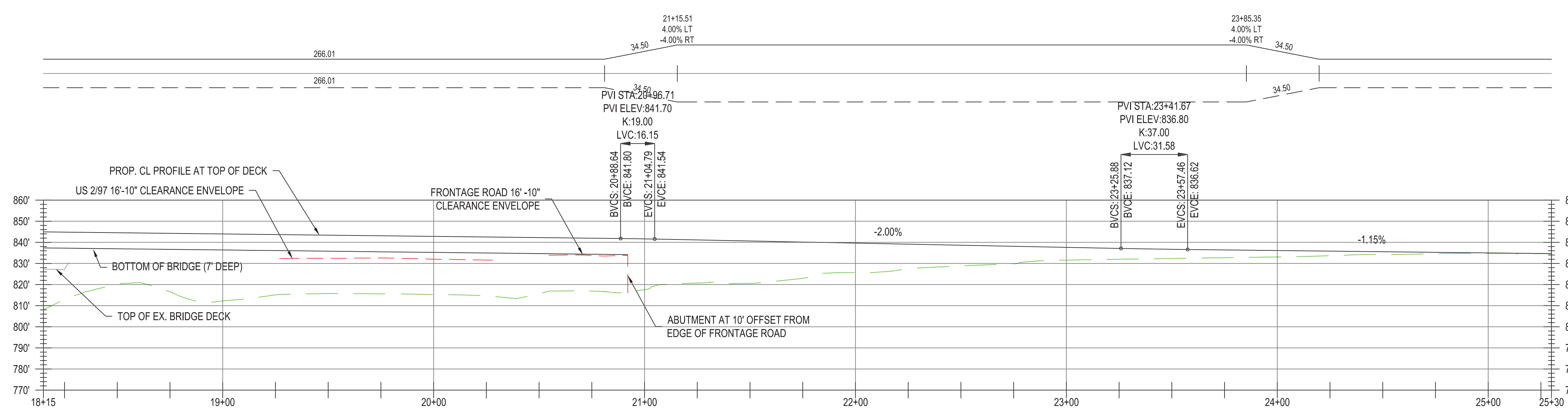
06/22/2016

Figure 2



BRIDGE ALIGNMENT A PROFILE

H: 1" = 30', V: 1" = 30'



BRIDGE ALIGNMENT A PROFILE

H: 1" = 30', V: 1" = 30'

WEST CASHMERE BRIDGE REPLACEMENT

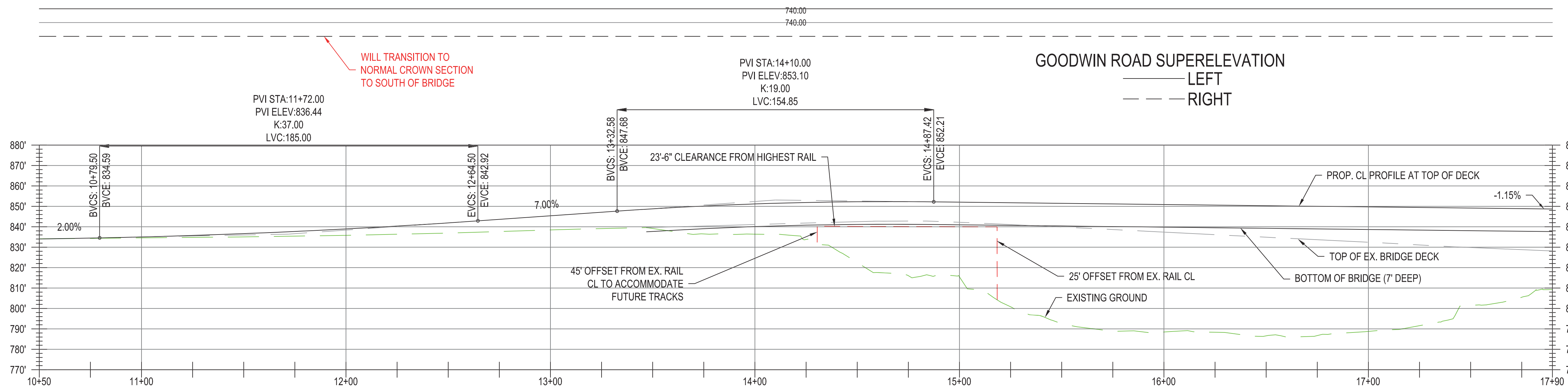
FIGURE 2
7' DEEP BRIDGE PRELIMINARY PROFILE

RH2 ENGINEERS PLANNERS SCIENTISTS

0 1" 2"
DRAWING IS NOT TO SCALE
IF BAR IS NOT 2" LONG

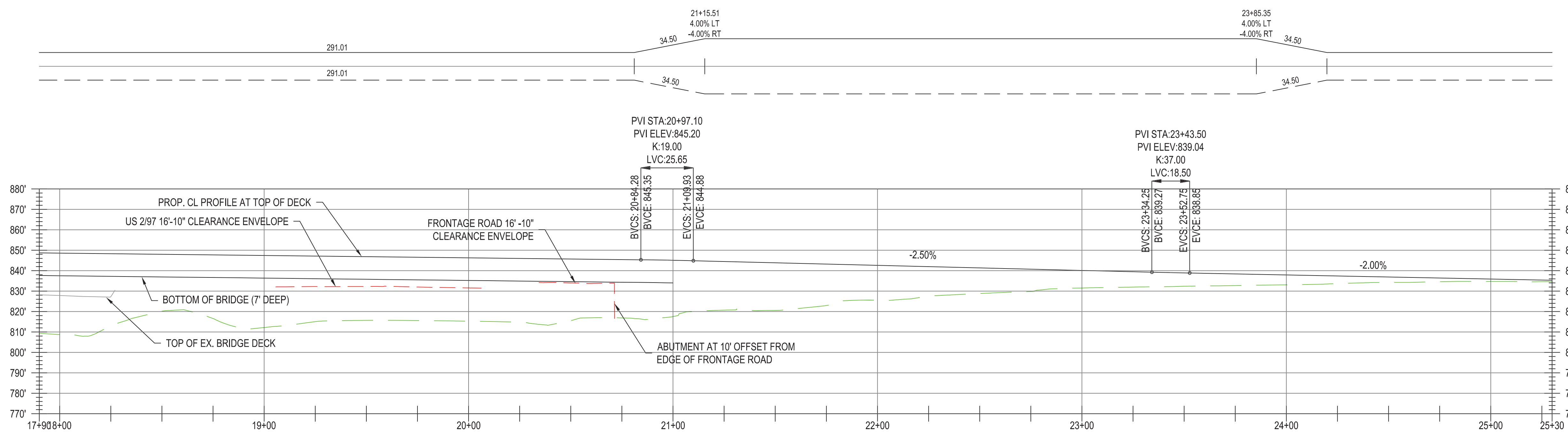
06/22/2016

Figure 3



BRIDGE ALIGNMENT A PROFILE

H: 1" = 30', V: 1" = 30'

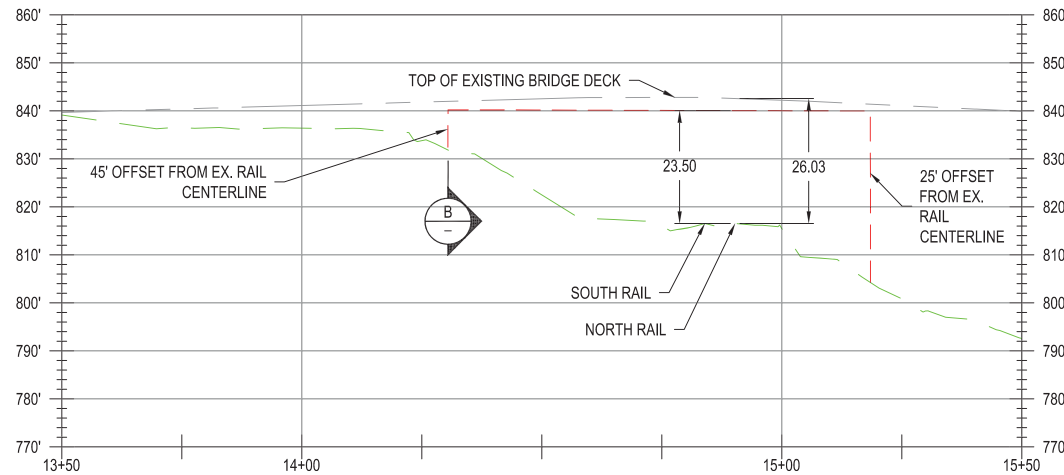


BRIDGE ALIGNMENT A PROFILE

H: 1" = 30', V: 1" = 30'

WEST CASHMERE BRIDGE REPLACEMENT	RH2 ENGINEERS PLANNERS SCIENTISTS
FIGURE 3 **10.5' DEEP BRIDGE PRELIMINARY PROFILE**	0 1" 2" DRAWING IS NOT TO SCALE IF BAR IS NOT 2" LONG 06/22/2016

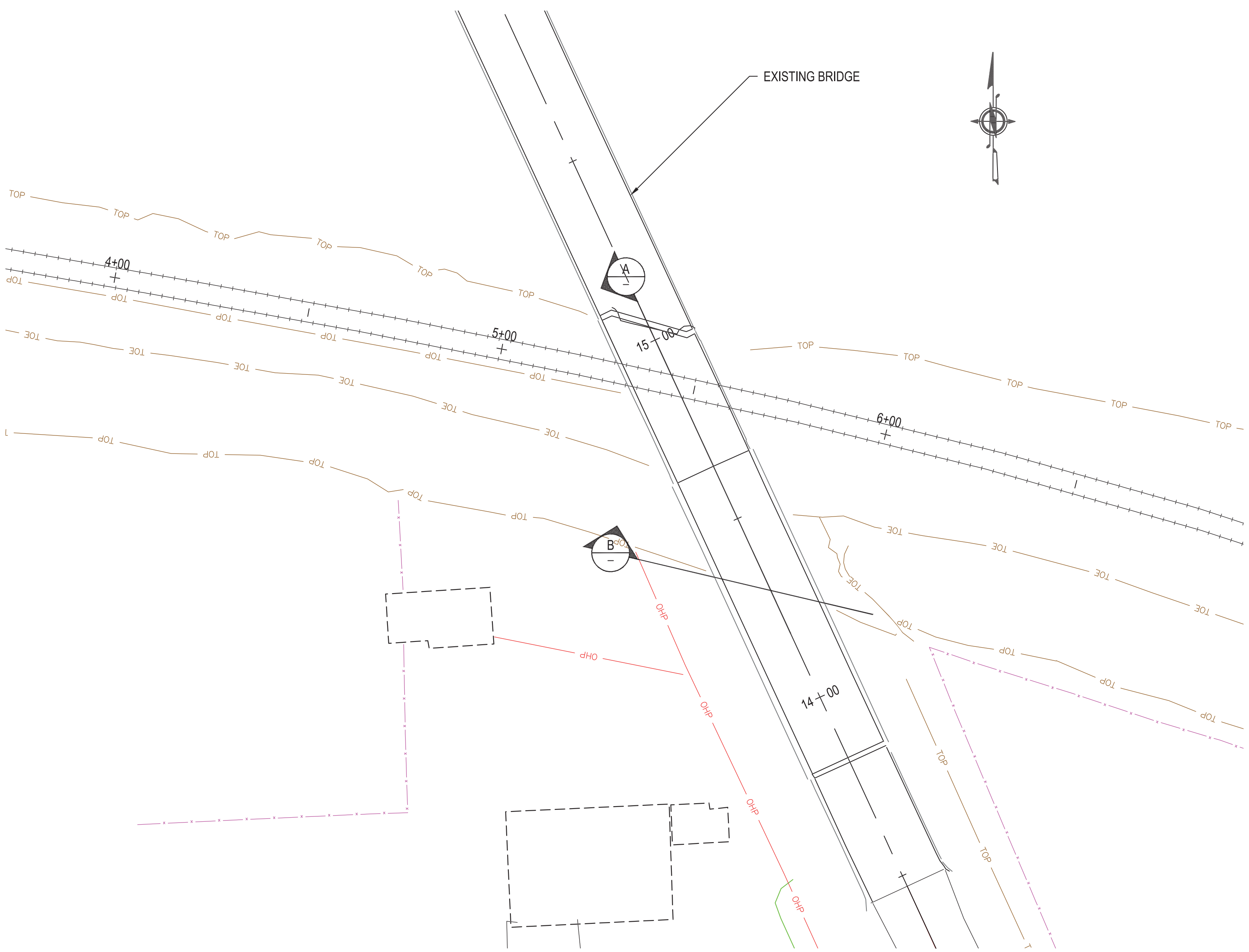
Figure 4



BRIDGE ALIGNMENT A PROFILE

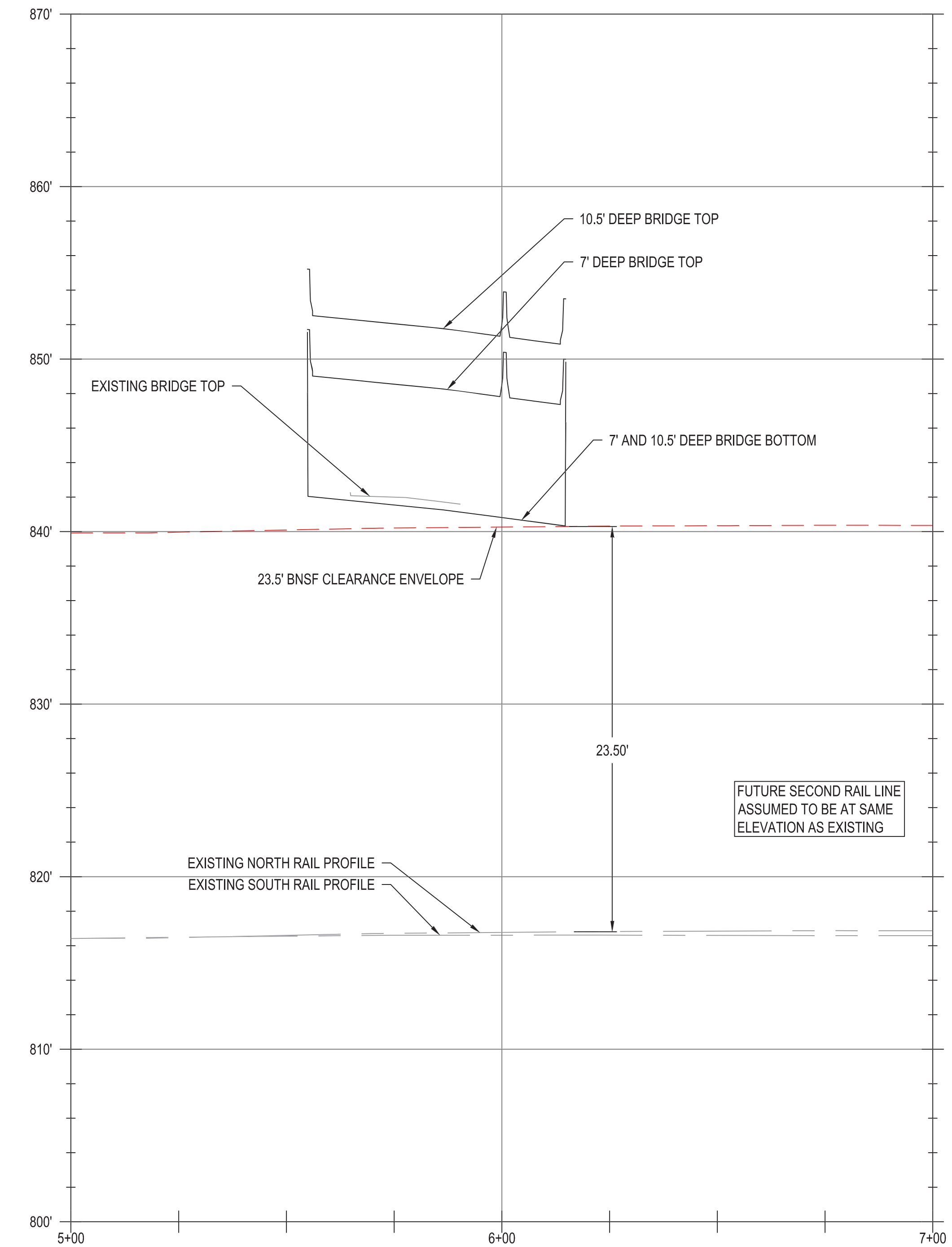
A

H: 1" = 20', V: 1" = 20'



BNSF RAILROAD VICINITY PLAN

1" = 20'



RAILROAD 45' SOUTH OFFSET FROM CENTERLINE PROFILE

B

H: 1" = 20', V: 1" = 5'

WEST CASHMERE BRIDGE REPLACEMENT

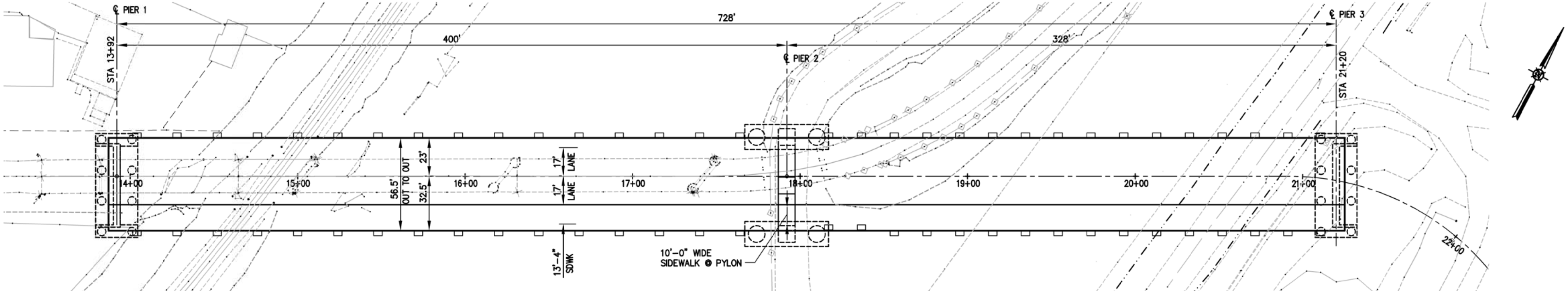
FIGURE 4
BNSF CROSSING EXHIBIT

RH2 ENGINEERS PLANNERS SCIENTISTS

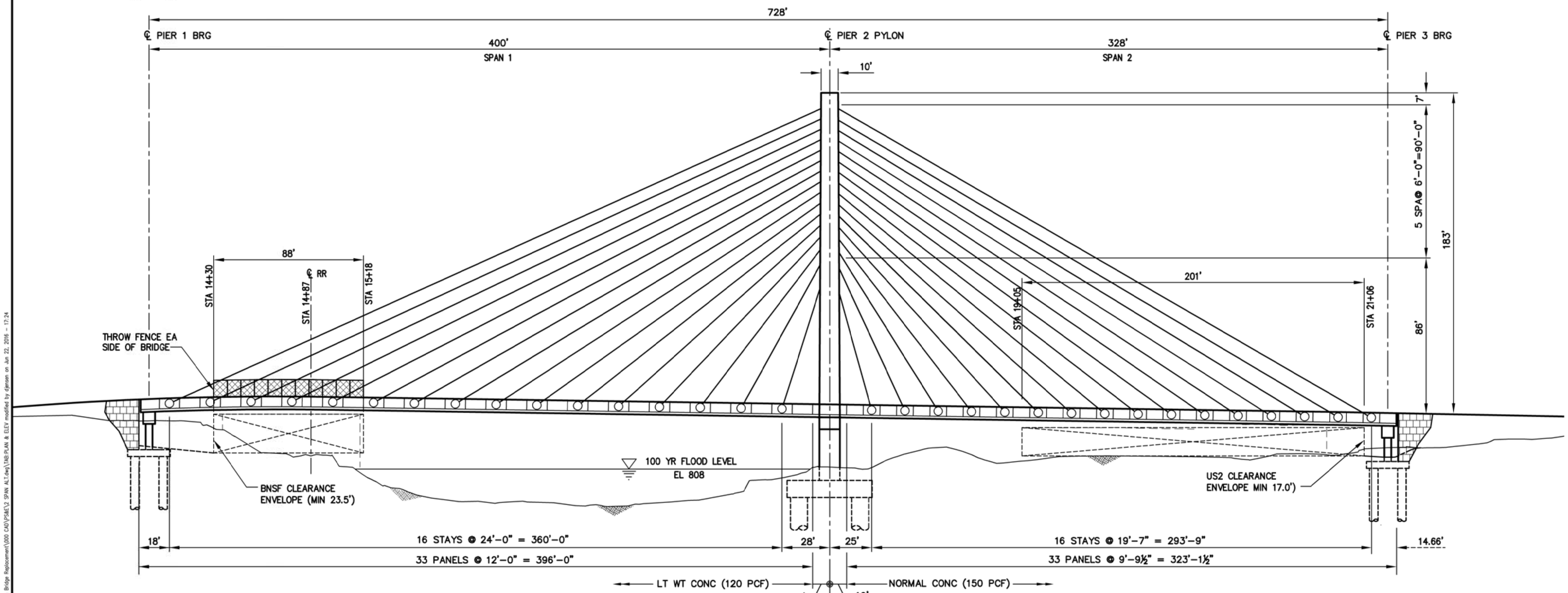
0 1" 2"
DRAWING IS NOT TO SCALE IF BAR IS NOT 2" LONG

6/21/2016

Appendix F – Bridge Viable Structural Concept Alternatives Drawings



BRIDGE PLAN
BARRIERS NOT SHOWN FOR CLARITY



BRIDGE LONGITUDINAL SECTION

TRANTECH Engineering LLC

BELLEVUE OFFICE:
121011 NE 1st ST, STE 305
BELLEVUE, WA 98005
WWW.TRANTECHENG.COM
PH: 425-453-5545
FAX: 425-453-6779

DATE:	ISSUED / REVISED	BY:
.	.	.

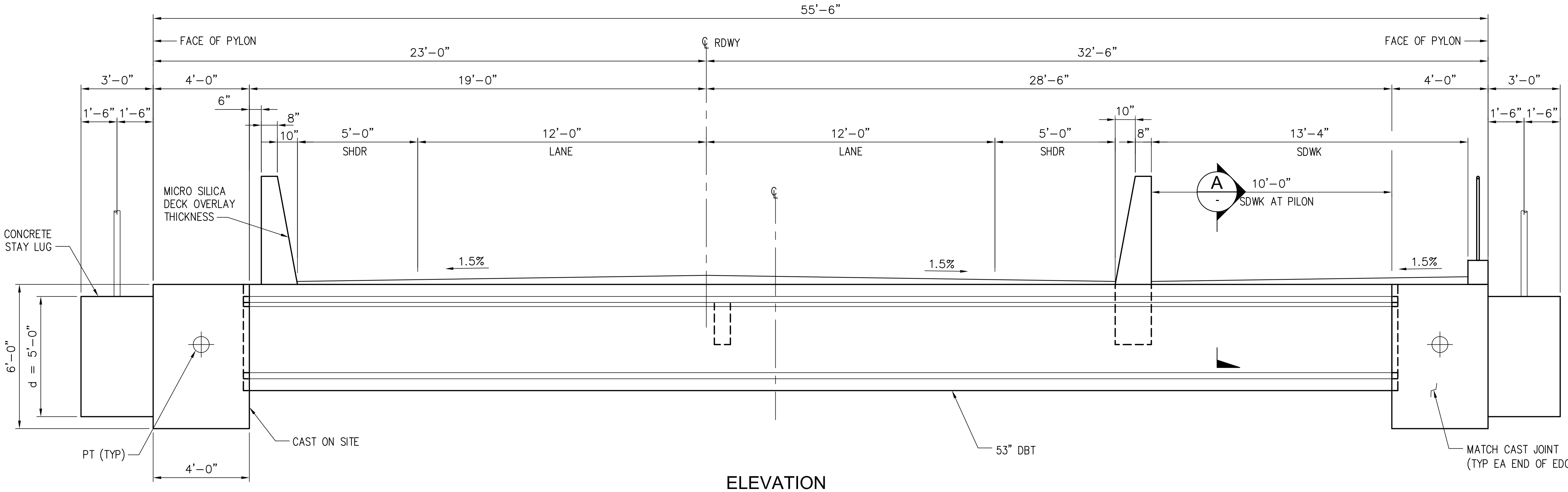
PUBLIC WORKS CHELAN COUNTY

Chelan County
Public Works Department
316 Washington Street, Suite 402
Wenatchee, Washington, 98801
Phone: 509. 667. 6415
Web Address: www.co.chelan.wa.us

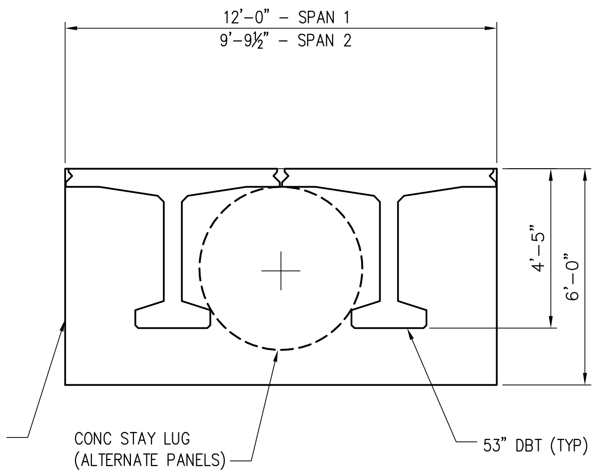
WEST CASHMERE BRIDGE REPLACEMENT PROJECT

2 - SPAN CABLE STAY ALTERNATIVE

PROJECT NUMBER #
SHEET #
PAGE # OF #

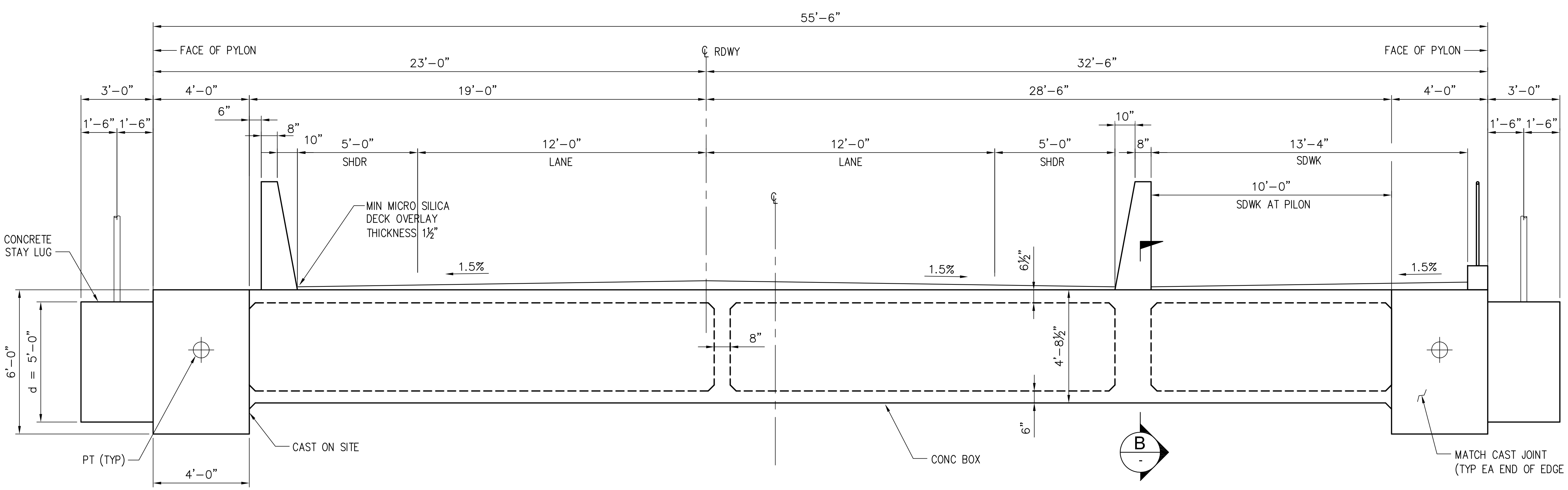


ELEVATION

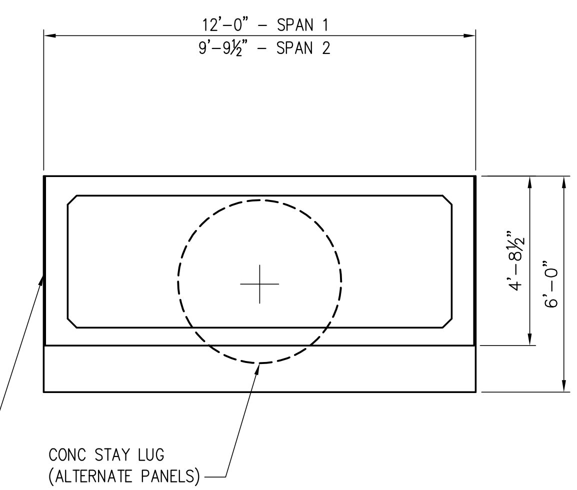


SECTION A

BRIDGE DECK - ALTERNATE 1



ELEVATION



SECTION B

BRIDGE DECK - ALTERNATE 2

TRANTECH Engineering LLC

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DATE:	ISSUED / REVISED	BY:
.	.	.

PUBLIC WORKS CHELAN COUNTY

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Wenatchee, Washington, 98801
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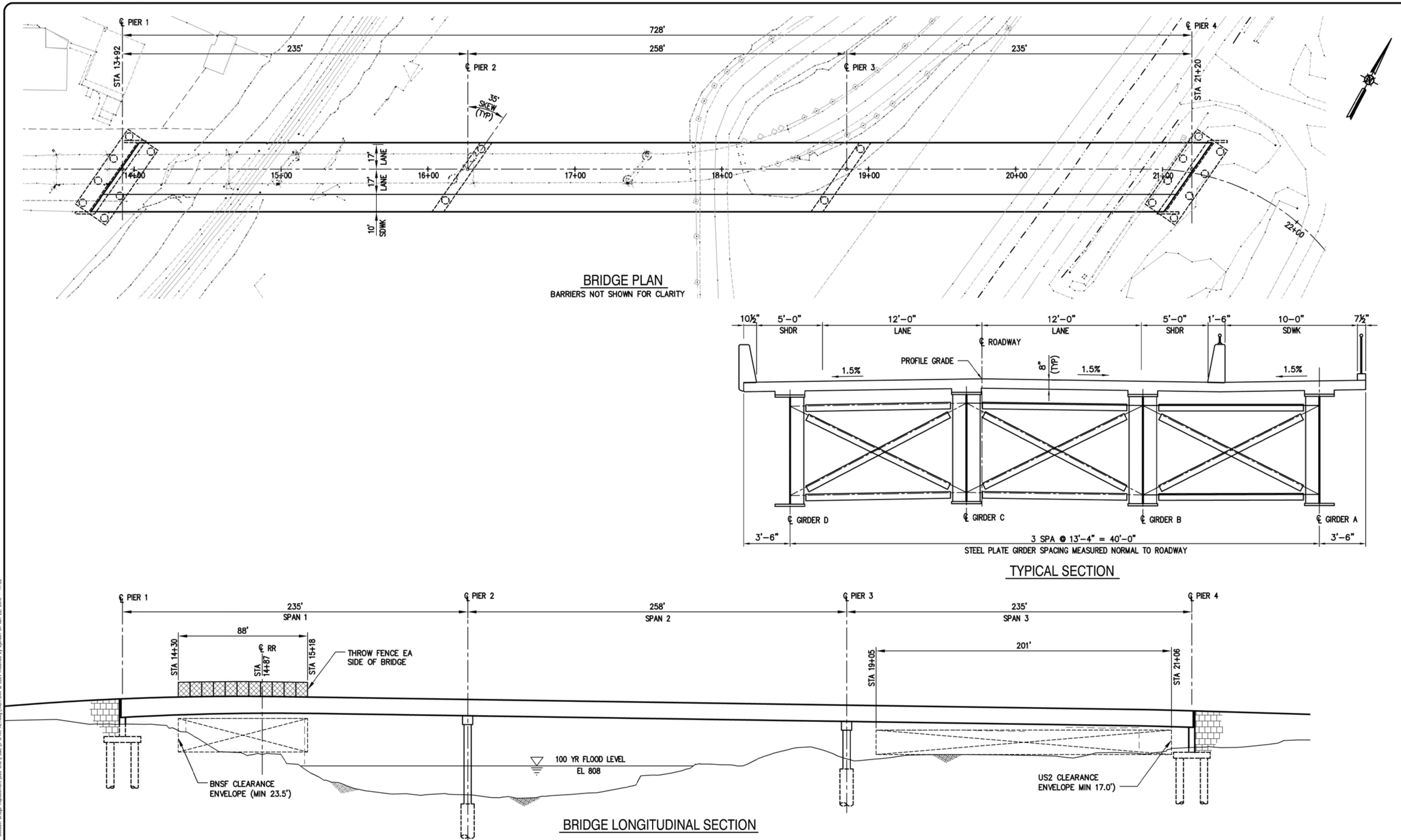
WEST CASHMERE BRIDGE REPLACEMENT PROJECT

2 - SPAN CABLE STAY BRIDGE DECK ALTS

PROJECT NUMBER #
SHEET #
PAGE # OF #



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WWW.TRANTECHENG.COM
PH: 425-453-5545
FAX: 425-453-6779

DATE:	ISSUED / REVISED	BY:
.	.	.

PUBLIC WORKS CHELAN COUNTY
Chelan County
Public Works Department
316 Washington Street, Suite 402
Wenatchee, Washington, 98801
Phone: 509. 667. 6415
Web Address: www.co.chelan.wa.us

WEST CASHMERE BRIDGE REPLACEMENT PROJECT

3 - SPAN STEEL GIRDER ALTERNATIVE

PROJECT NUMBER
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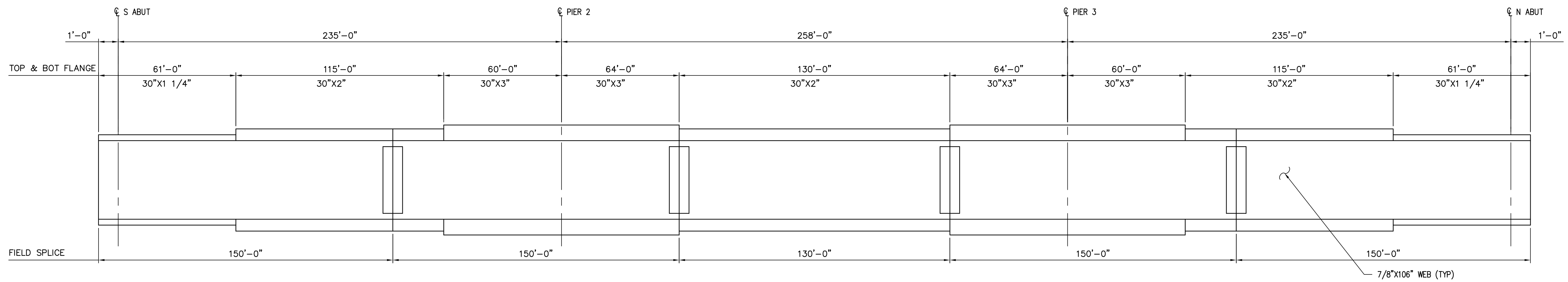
PAGE
OF



GIRDER ELEVATION

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TRANTECH Engineering LLC

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DATE:	ISSUED / REVISED	BY:
.	.	.

Chelan County
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Wenatchee, Washington, 98801
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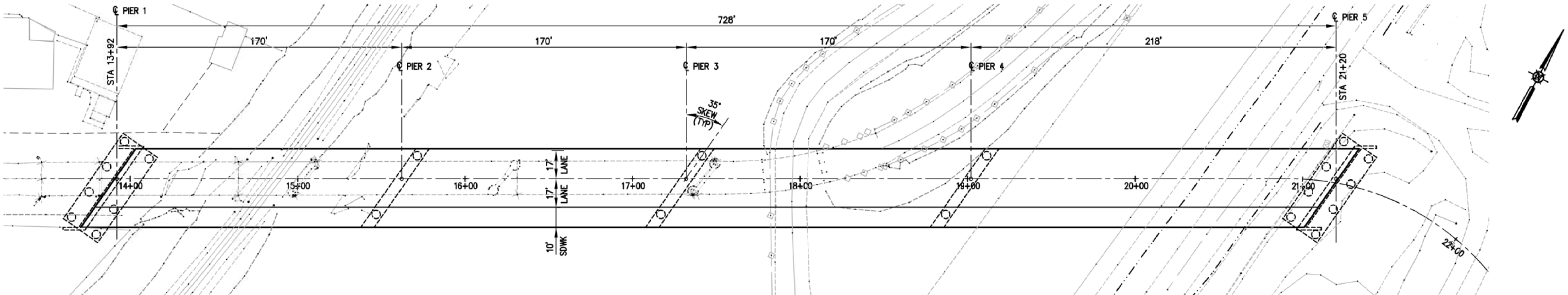
WEST CASHMERE BRIDGE REPLACEMENT PROJECT

3 - SPAN STEEL PLATE GIRDER ELEVATION

PROJECT NUMBER #

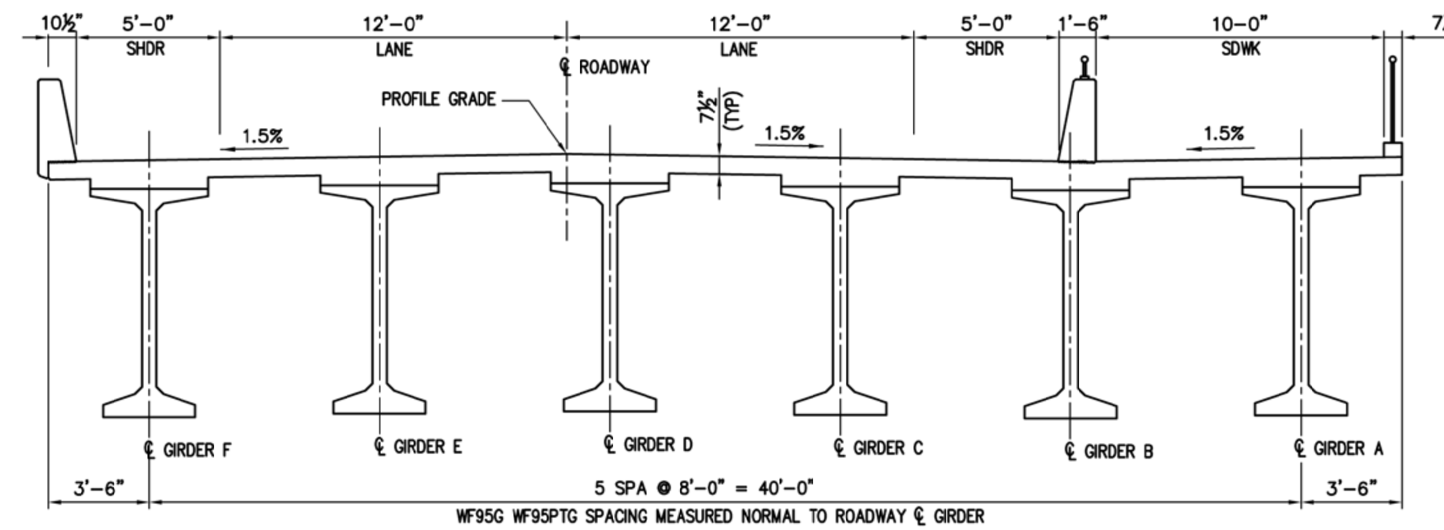
SHEET #

PAGE # OF #

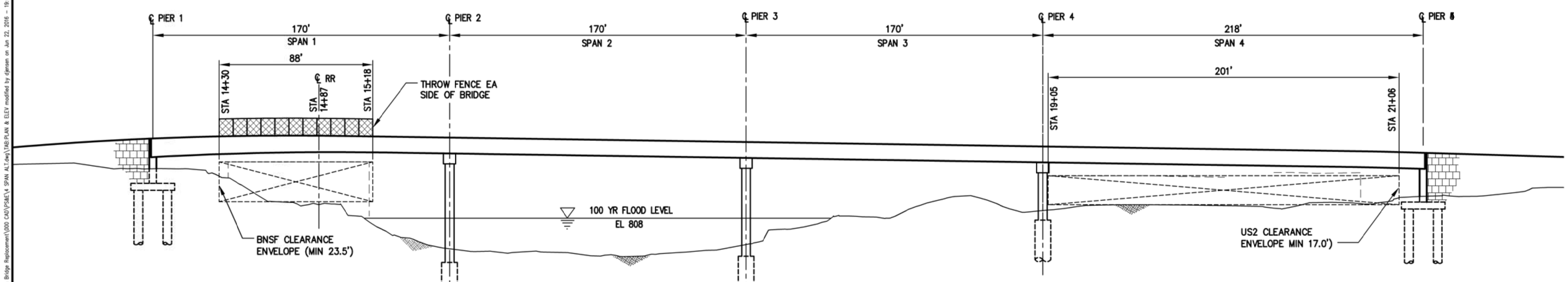


BRIDGE PLAN

BARRIERS NOT SHOWN FOR CLARITY



TYPICAL SECTION



BRIDGE LONGITUDINAL SECTION

TRANTECH Engineering LLC

BELLEVUE OFFICE:
121011 NE 1st ST, STE 305
BELLEVUE, WA 98005
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FAX: 425-453-6779

DATE:	ISSUED / REVISED	BY:
.	.	.

Chelan County
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WEST CASHMERE BRIDGE REPLACEMENT PROJECT

4 - SPAN CONCRETE GIRDER ALTERNATIVE

PROJECT NUMBER #

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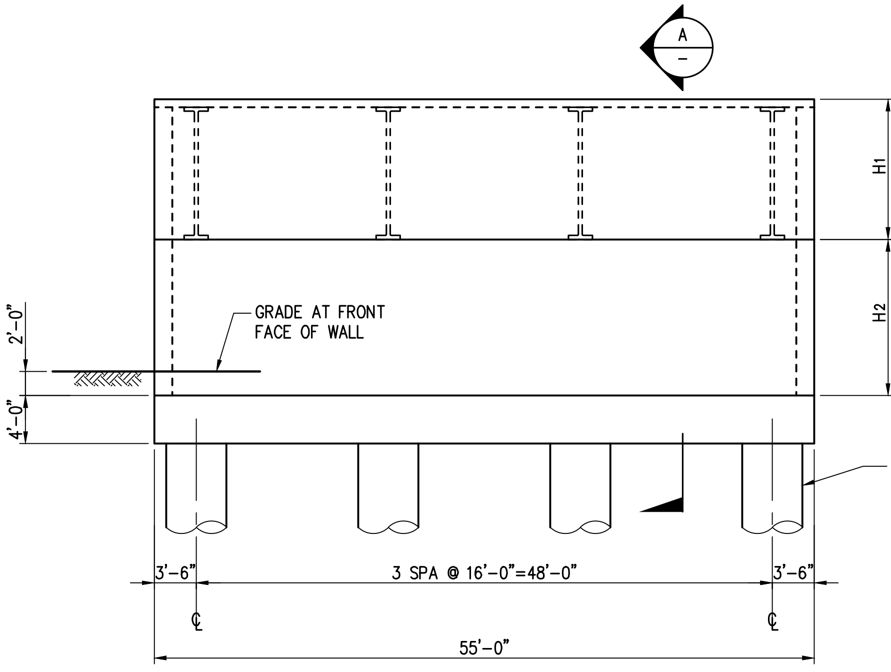
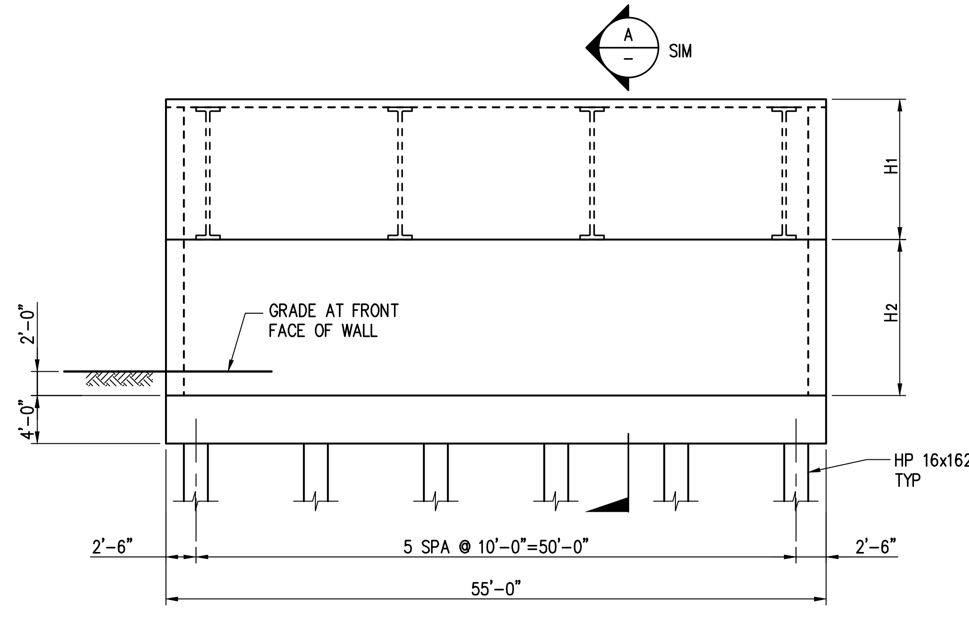


ABUTMENT ELEVATION

(DRILLED SHAFT FOUNDATION)
SEE NOTE 2

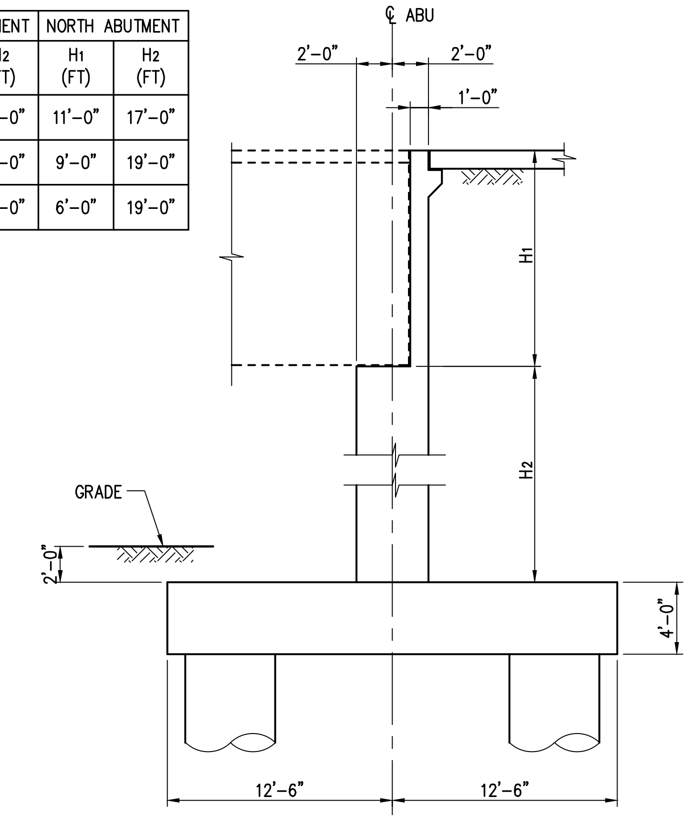


ABUTMENT ELEVATION

(H PILE FOUNDATION)
SEE NOTE 2

ABUTMENT WALL HEIGHT

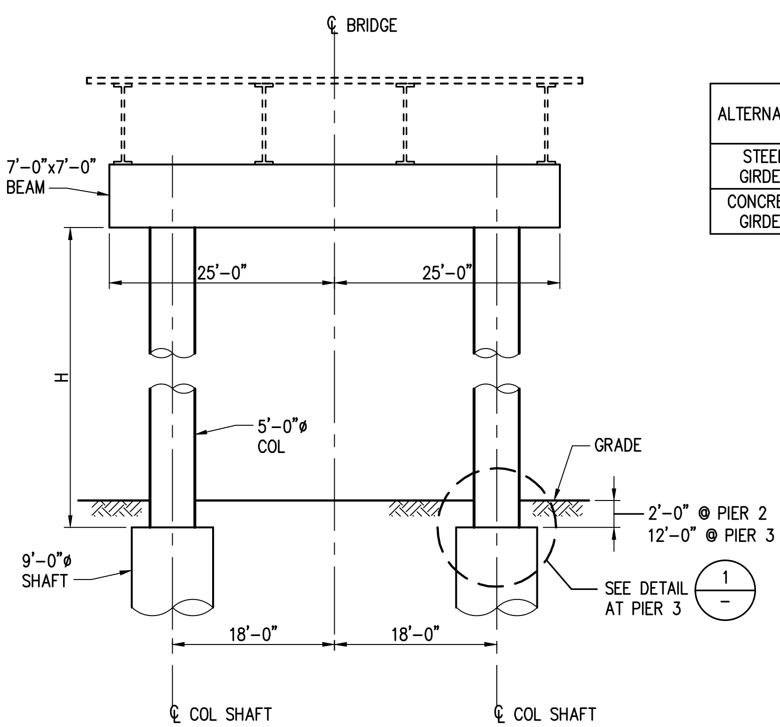
ALTERNATIVE	SOUTH ABUTMENT H1 (FT)	SOUTH ABUTMENT H2 (FT)	NORTH ABUTMENT H1 (FT)	NORTH ABUTMENT H2 (FT)
STEEL GIRDER	11'-0"	13'-0"	11'-0"	17'-0"
CONCRETE GIRDER	9'-0"	15'-0"	9'-0"	19'-0"
CABLE STAYED	6'-0"	15'-0"	6'-0"	19'-0"



SECTION A

NOTES:

1. STEEL GIRDER ALTERNATIVE IS SHOWN. CONCRETE GIRDER ALTERNATIVE IS SIMILAR.
2. STEEL GIRDER ALTERNATIVE IS SHOWN. CONCRETE GIRDER ALTERNATIVE AND CABLE STAYED ALTERNATIVE ARE SIMILAR.

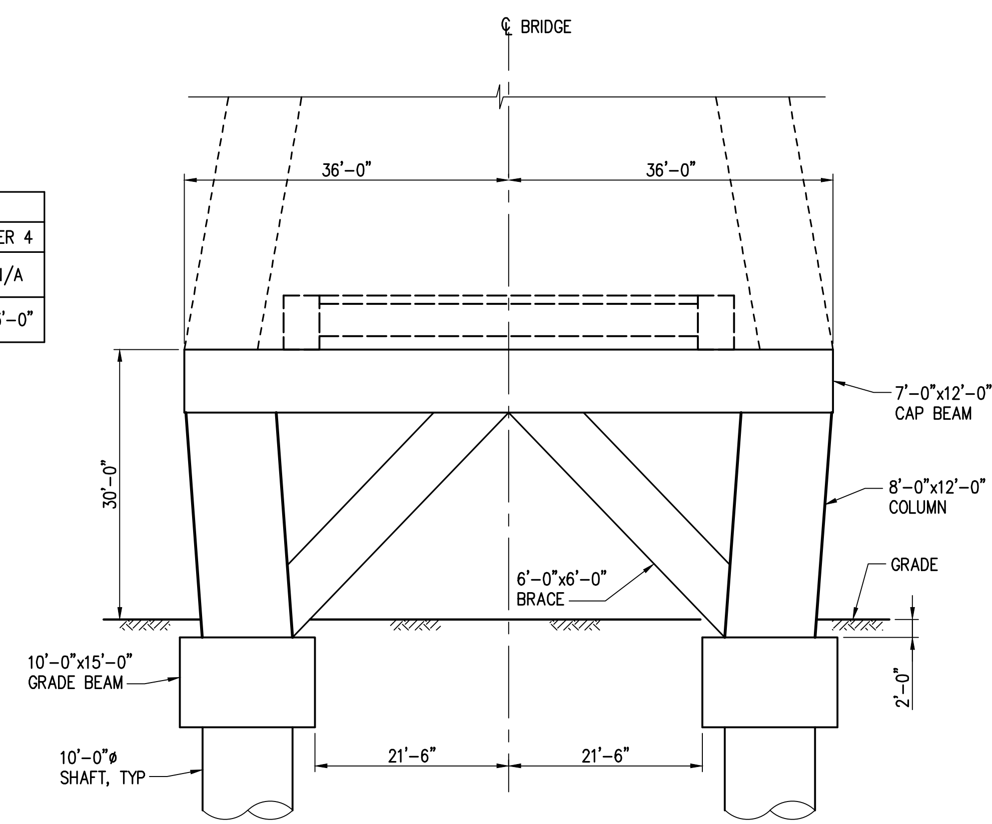


COLUMN HEIGHT

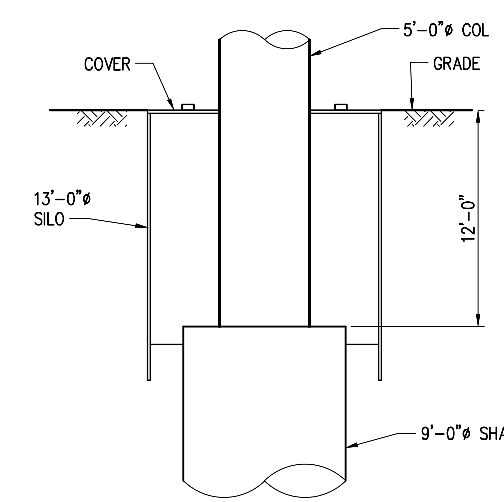
ALTERNATIVE	H (FT) PIER 2	H (FT) PIER 3	H (FT) PIER 4
STEEL GIRDER	49'-0"	32'-0"	N/A
CONCRETE GIRDER	50'-0"	45'-0"	33'-0"

INTERMEDIATE PIER ELEVATION
STEEL GIRDER AND CONCRETE GIRDER ALTERNATIVES

SEE NOTE 1



TOWER PIER ELEVATION
CABLE STAYED ALTERNATIVE



SILO DETAIL 1

AT PIER 3 (STEEL GIRDER ALTERNATIVE)
AT PIER 4 (CONCRETE GIRDER ALTERNATIVE)

kpff
1601 5th Avenue
Suite 1600
Seattle, WA 98101
206.622.5822
www.kpff.com

DATE:	ISSUED / REVISED	BY:

Chelan County
Public Works Department
316 Washington Street, Suite 402
Wenatchee, Washington, 98801
Phone: 509. 667. 6415
Web Address: www.co.chelan.wa.us

WEST CASHMERE BRIDGE REPLACEMENT PROJECT

X

PROJECT NUMBER X
SHEET X
PAGE X OF #

Appendix G – Bridge Alternatives Opinion of Cost

Ott - Sakai & Associates, LLC

West Cashmere Bridge Replacement

Item No.	Description	4 Span Concrete Girder Alter.- Alignment A	3 Span Steel Girder Alter.- Alignment A	2 Span Cable Stay Alter.- Alignment A	4 Span Concrete Girder - Alignment B	3 Span Steel Girder- Alignment B	2 Span Cable Stay - Alignment B
	Mobilization	$1,529,145	$1,424,683	$2,111,484	$1,709,745	$1,567,483	$2,216,484
1	Drilled Shafts Foundation	$1,839,445	$1,719,831	$2,112,478	$1,839,445	$1,719,831	$2,112,478
2	Work Access Trestle	$1,806,000	$1,428,000	$1,050,000	$3,612,000	$2,856,000	$2,100,000
3	Substructure Concrete	$1,583,526	$1,178,321	$3,734,378	$1,583,526	$1,178,321	$3,734,378
4	Shoring/Excavation- Abutment and Interior Piers	$370,360	$290,060	$332,210	$370,360	$290,060	$332,210
5	Bridge Superstructure	$7,221,750	$7,216,250	$11,471,400	$7,221,750	$7,216,250	$11,471,400
6	Approach Slab	$52,200	$52,200	$52,200	$52,200	$52,200	$52,200
7	Bridge Demolition	$204,150	$204,150	$204,150	$204,150	$204,150	$204,150
8	Approach Fill (Shoring and Flowable Fill)	$1,414,020	$1,358,020	$1,358,020	$1,414,020	$1,358,020	$1,358,020
9	Civil Items	$800,000	$800,000	$800,000	$800,000	$800,000	$800,000
10	Contingency- 20%	$3,364,119	$3,134,303	$4,645,264	$3,761,439	$3,448,463	$4,876,264
	Total	$20,184,716	$18,805,818	$27,871,583	$22,568,636	$20,690,778	$29,257,583

Ott - Sakai & Associates, LLC

Item 1 Analysis **West Cashmere Bridge Replacement- City of Cashmere**

4 Span Concrete Girder Alternative
Abutment with Drilled Shaft

Drilled Shaft at Abutment - 5' Diameter - 8 each x 72' Deep per Abutment

Location	**Description**	**Quantity**	**Unit of Measure**	**Unit Price**	**Total**
Pier 1 and 5	Drill and Excavate	896	CY	$350	$313,600
Pier 1 and 5	Place Concrete	896	CY	$350	$313,600
Pier 1 and 5	Rebar	237,302	LBS	$1.25	$296,628
Pier 1 and 5	CSL Tubes	7,392	LF	$5	$36,960
Pier 1 and 5	CSL Test	4	EA	$1,500	$6,000
Shaft Obstruction Provision	10% of Total	1	LS	$96,679	$96,679
				Subtotal:	$1,063,466

Drilled Shaft at Abutment - Pier 2 - 8' Diameter - 2 each x 32' Deep per Pier, Pier 3- 8' Diameter - 2 each x 36' Deep per Pier, Pier 4- 12' Diameter- 2 each x 54' Deep

Location	**Description**	**Quantity**	**Unit of Measure**	**Unit Price**	**Total**
Pier 2,3,4,	Drill and Excavate	705	CY	$350	$246,750
Pier 2,3,4,	Place Concrete	591	CY	$350	$206,850
Pier 2,3,4,	Rebar	167,253	LBS	$1.25	$209,066
Pier 2,3,4,	CSL Tubes	2,772	LF	$5	$13,860
Pier 2,3,4,	CSL Test	4	EA	$1,500	$6,000
Pier 2,3,4,	Concrete Silo	1	LS	$11,000	$11,000
Pier 2,3,4,	Temporary Casing	244	LF	$50	$12,200
Pier 2,3,4,	Shaft Casing Shoring	80	LF	$25	$2,000
Shaft Obstruction Provision	10% of Total	1	LS	$68,253	$68,253
				Subtotal	$775,979
				Total	$1,839,445

Item 2 Analysis **West Cashmere Bridge Replacement- City of Cashmere**
Work Access Trestle

Unit Price Temporary Work Trestle- $140/SF

Trestle Footprint Main Spine - 250'x 30' = 7500 SF
Trestle Pier Footprint- Pier 2, 3 Finger- 30'x90'x2= 5,400 SF

Temporary Work Trestle - Cost Analysis- Furnish - Install- Remove

Description	**Quantity**	**Unit of Measure**	**Unit Price**	**Total**
Temporary Work Trestle	12,900	SF	$140	$1,806,000

Item 3 Analysis

West Cashmere Bridge Replacement- City of Cashmere
Substrucure Concrete

Description	Quantity	Unit of Measure	Unit Price	Total
Abutment Footing- Pier 1 and 5	460	CY	$550	$253,000
Abutment Footing Rebar- Pier 1 and 5	121,716	LBS	$1.25	$152,145
Abutment Wall - Wingwall-Pier 1 and 5	335	CY	$700	$234,500
Abutment Wall Rebar - Wingwall-Pier 1 and 5	88641	LBS	$1.25	$110,801
Column Concrete - Pier 2,3,4	186	CY	$800	$148,800
Column Concrete Rebar- Pier 2,3,4	49,216	LBS	$1.25	$61,520
Pier Cap Concrete - Pier 2,3,4	506	CY	$900	$455,400
Pier Cap Rebar- Pier 2,3,4	133,888	LBS	$1.25	$167,360
		Total		$1,583,526

Item 4 Analysis

Shoring and Excavation Analysis - West Cashmere Bridge Replacement
Temporary Casing - Pier 2,3,4- $200/LF
Pier 1 and 5 Unit Price Shoring - $60/SF (Soldier Pile Wall w/ Lagging)
Unit Price Strucural Excavation- $30/CY-
Gravel Backfill - $20/CY

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Shoring Pier 1 and 5	3,296	SF	$60	$197,760	
Temporary Casing Pier 2,3,4	72	LF	$200	$14,400	12' Diameter Casing at Piers
Structural Excavation Pier 1 and 5	1,368	CY	$30	$41,040	
Structural Excavation Pier 2,3,4	302	CY	$350	$105,700	Performed During Shaft Installation
Gravel Backfill Pier 1 and 5	573	CY	$20	$11,460	
			Total	$370,360	

Item 5 Analysis

Bridge Supersrtucture

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
WF 95G Girder- Furnish and Install	960	LF	$700	$672,000	
WF 95G Girder- Spliced Furnish and Install	3408	LF	$1,200	$4,089,600	
Bearings	12	EA	$3,000	$36,000	
Intermediate and End Diaphragm Concrete -Spans 1-4	285	CY	$1,000	$285,000	
Intermediate and End Diaphragm Rebar -Spans 1-4-120#/CY	75,120	LBS	$1.25	$93,900	
Concrete Deck- Span 1-4	903	CY	$1,200	$1,083,600	
Deck Rebar- Span 1-4	225,000	LBS	$1.25	$281,250	
Concrete Barrier	1,496	LF	$175	$261,800	
BP Railing	728	LF	$100	$72,800	

Expansion Joints	2	EA	$50,000	$100,000
Utilities	728	LF	$100	$72,800
Pedestrian Railing	655	LF	$200	$131,000
Throw Fence	140	LF	$300	$42,000
			Total	$7,221,750

Item 6 Analysis

Approach Slab-47' x 25' x 2 each

Unit Price- $200/SY

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Approach Slabs	261	SY	$200	$52,200	

Item 7 Analysis

Bridge Demolition **Surface Area- 490'x25'= 12,250 sf (1361 SY)**

Unit Price- $150/SY

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Existing Bridge Demolition	1361	SY	$150	$204,150	

Item 8 Analysis

Approach Fills

South Approach - Length- 300' x Average Height - 9' High

North Approach- Length - 430'x Average Height - 13' High

MSE Wall (South Approach)- 300' x 9' x 2 sides = 5400 SF

MSE Wall (North Approach)- 430' x 13' x 2 sides = 11,180 SF

Riverbank Protection - 60' Wide x 120' Long x 3' Thick = 800 CY x 1.4 Tons/ CY = 1120 Tons

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
MSE Wall Approach	16580	SF	$40	$663,200	
Gravel Backfill	15351	CY	$20	$307,020	
HMA Pavement- 730' x 50'Wide x 6" Thick	1323	TON	$100	$132,300	
Moment Slab Barrier- 730' Length x 2 sides	1460	LF	$125	$182,500	
BP Barrier	730	LF	$100	$73,000	
Riverbank Protection	1120	Tons	$50	$56,000	
			Total	$1,414,020	

Ott - Sakai
& Associates, LLC

Item 1 Analysis **West Cashmere Bridge Replacement- City of Cashmere**

3 Span Steel Girder Alternative
Abutment with Drilled Shaft

Drilled Shaft at Abutment - 5' Diameter - 8 each x 60' Deep per Abutment

Location	Description	Quantity	Unit of Measure	Unit Price	Total
Pier 1 and 4	Drill and Excavate	896	CY	$350	$313,600
Pier 1 and 4	Place Concrete	896	CY	$350	$313,600
Pier 1 and 4	Rebar	237,302	LBS	$1.25	$296,628
Pier 1 and 4	CSL Tubes	7,392	LF	$5	$36,960
Pier 1 and 4	CSL Test	4	EA	$1,500	$6,000
Shaft Obstruction Provision	10% of Total	1	LS	$96,679	$96,679
				Subtotal:	$1,063,466

Drilled Shaft at Abutment - Pier 2- 8' Diameter - 2 each x 31' Deep per Pier, Pier 3- 12' Diameter- 2 each x 56' Deep per Pier

Location	Description	Quantity	Unit of Measure	Unit Price	Total
Pier 2,3	Drill and Excavate	585	CY	$350	$204,750
Pier 2,3	Place Concrete	476	CY	$350	$166,600
Pier 2,3	Rebar	134,708	LBS	$1.25	$168,385
Pier 2,3	CSL Tubes	7,392	LF	$5	$36,960
Pier 2,3	CSL Test	2	EA	$1,500	$3,000
Pier 2,3,4,	Concrete Silo	1	LS	$9,000	$9,000
Pier 2,3,4,	Temporary Casing	174	LF	$50	$8,700
Pier 2,3,4,	Shaft Casing Shoring	40	LF	$25	$1,000
Shaft Obstruction Provision	10% of Total	1	LS	$57,970	$57,970
				Subtotal	$656,365
				Total	$1,719,831

Item 2 Analysis **West Cashmere Bridge Replacement- City of Cashmere**
Work Access Trestle

Unit Price Temporary Work Trestle- $140/SF

Trestle Footprint Main Spine - 250'x 30' = 7500 SF
Trestle Pier Footprint- Pier 2 Finger- 30'x90'= 2700 SF

Temporary Work Trestle - Cost Analysis- Furnish - Install- Remove

Description	Quantity	Unit of Measure	Unit Price	Total
Temporary Work Trestle	10,200	SF	$140	$1,428,000

Item 3 Analysis

West Cashmere Bridge Replacement- City of Cashmere
Substrucure Concrete

Description	Quantity	Unit of Measure	Unit Price	Total
Abutment Footing- Pier 1 and 4	459	CY	$550	$252,450
Abutment Footing Rebar- Pier 1 and 4	121,451	LBS	$1.25	$151,814
Abutment Wall - Wingwall-Pier 1 and 5	306	CY	$700	$214,200
Abutment Wall Rebar - Wingwall-Pier 1 and 5	80967	LBS	$1.25	$101,209
Column Concrete - Pier 2,3	115	CY	$800	$92,000
Column Concrete Rebar- Pier 2,3	30,429	LBS	$1.25	$38,036
Pier Cap Concrete - Pier 2,3	267	CY	$900	$240,300
Pier Cap Rebar- Pier 2,3	70,650	LBS	$1.25	$88,313
		Total		$1,178,321

Item 4 Analysis

Shoring and Excavation Analysis - West Cashmere Bridge Replacement
Temporary Casing - Pier 2,3,4- $200/LF
Pier 1 and 5 Unit Price Shoring - $60/SF (Soldier Pile Wall w/ Lagging)
Unit Price Strucural Excavation- $30/CY-
Gravel Backfill - $20/CY

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Shoring Pier 1 and 5	3,296	SF	$60	$197,760	
Temporary Casing Pier 2	24	LF	$200	$4,800	12' Diameter Casing at Piers
Structural Excavation Pier 1 and 5	1,368	CY	$30	$41,040	
Structural Excavation Pier 2	100	CY	$350	$35,000	Performed During Shaft Installation
Gravel Backfill Pier 1 and 5	573	CY	$20	$11,460	
			Total	$290,060	

Item 5 Analysis

Bridge Supersrtucture

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Steel Girder and Diaphragms	2125000	LBS	$2.40	$5,100,000	
Bearings	16	EA	$5,000	$80,000	
Concrete Deck- Span 1-4	903	CY	$1,200	$1,083,600	
Deck Rebar- Span 1-4	225,000	LBS	$1.25	$281,250	
Concrete Barrier	1,496	LF	$175	$261,800	
BP Railing	728	LF	$100	$72,800	
Expansion Joints	2	EA	$50,000	$100,000	
Utilities	728	LF	100	72,800	
Pedestrian Railing	556	LF	$200	$111,200	
Throw Fence	176	LF	$300	$52,800	

				Total	$7,216,250	

Item 6 Analysis

Approach Slab-47' x 25' x 2 each

Unit Price- $200/SY

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Approach Slabs	261	SY	$200	$52,200	

Item 7 Analysis

Bridge Demolition **Surface Area- 490'x25'= 12,250 sf (1361 SY)**

Unit Price- $150/SY

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Existing Bridge Demolition	1361	SY	$150	$204,150	

Item 8 Analysis

Approach Fills

South Approach - Length- 300' x Average Height - 9' High

North Approach- Length - 430'x Average Height - 13' High

MSE Wall (South Approach)- 300' x 9' x 2 sides = 5400 SF

MSE Wall (North Approach)- 430' x 13' x 2 sides = 11,180 SF

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
MSE Wall Approach	16580	SF	$40	$663,200	
Gravel Backfill	15351	CY	$20	$307,020	
HMA Pavement- 730' x 50'Wide x 6" Th	1323	TON	$100	$132,300	
Moment Slab Barrier- 730' Length x 2 sides	1460	LF	$125	$182,500	
BP Barrier	730	LF	$100	$73,000	
			Total	$1,358,020	

Ott - Sakai & Associates, LLC

Item 1 Analysis **West Cashmere Bridge Replacement- City of Cashmere**

2 Span Cable Stay Alternative
Abutment with Drilled Shaft

Drilled Shaft at Abutment - 5' Diameter - 8 each x 60' Deep per Abutment

Location	Description	Quantity	Unit of Measure	Unit Price	Total
Pier 1 and 3	Drill and Excavate	896	CY	$350	$313,600
Pier 1 and 3	Place Concrete	896	CY	$350	$313,600
Pier 1 and 3	Rebar	237,302	LBS	$1.25	$296,628
Pier 1 and 3	CSL Tubes	7,392	LF	$5	$36,960
Pier 1 and 3	CSL Test	4	EA	$1,500	$6,000
Shaft Obstruction Provision	10% of Total	1	LS	$96,679	$96,679
				Subtotal:	$1,063,466

Drilled Shaft at Abutment - 10' Diameter - 4 each x 60' Deep per Pier

Location	Description	Quantity	Unit of Measure	Unit Price	Total
Pier 2	Drill and Excavate	907	CY	$350	$317,450
Pier 2	Place Concrete	895	CY	$350	$313,250
Pier 2	Rebar	240,384	LBS	$1.25	$300,480
Pier 2	CSL Tubes	1,848	LF	$5	$9,240
Pier 2	CSL Test	2	EA	$1,500	$3,000
Pier 2	Concrete Silo	1	LS	$5,000	$5,000
Pier 2	Temporary Casing	100	LF	$50	$5,000
Pier 2	Shaft Casing Shoring	50	LF	$25	$1,250
Shaft Obstruction Provision	10% of Total	1	LS	$94,342	$94,342
				Subtotal	$1,049,012
				Total	$2,112,478

Item 2 Analysis **West Cashmere Bridge Replacement- City of Cashmere**
Work Access Trestle

Unit Price Temporary Work Trestle- $140/SF

Trestle Footprint Main Spine - 250'x 30' = 7500 SF

Temporary Work Trestle - Cost Analysis- Furnish - Install- Remove

Description	Quantity	Unit of Measure	Unit Price	Total
Temporary Work Trestle	7,500	SF	$140	$1,050,000

Item 3 Analysis

West Cashmere Bridge Replacement- City of Cashmere
Substrucure Concrete

Description	Quantity	Unit of Measure	Unit Price	Total
Abutment Footing- Pier 1 and 3	335	CY	$550	$184,250
Abutment Footing Rebar- Pier 1 and 3	88,641	LBS	$1.25	$110,801
Abutment Wall - Wingwall-Pier 1 and 3	335	CY	$700	$234,500
Abutment Wall Rebar - Wingwall-Pier 1 and 3	88641	LBS	$1.25	$110,801
Column Concrete - Pier 2	289	CY	$800	$231,200
Column Concrete Rebar- Pier 2	76,770	LBS	$1.25	$95,963
Pier Cap Concrete - Pier 2	278	CY	$900	$250,200
Pier Cap Rebar- Pier 2,3	73,560	LBS	$1.25	$91,950
Tower - Pier 2	1,584	CY	$1,200.00	$1,900,800
Tower Rebar-Pier 2	419,130	LBS	$1.25	$523,913
		Total		$3,734,378

Item 4 Analysis

Shoring and Excavation Analysis - West Cashmere Bridge Replacement
Temporary Casing - Pier 2,3,4- $200/LF
Pier 1 and 5 Unit Price Shoring - $60/SF (Soldier Pile Wall w/ Lagging)
Unit Price Strucural Excavation- $30/CY-
Gravel Backfill - $20/CY

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Shoring Pier 1 and 3	3,296	SF	$60	$197,760	
Temporary Casing Pier 2	58	LF	$200	$11,600	12' Diameter Casing at Piers
Structural Excavation Pier 1 and 3	1,368	CY	$30	$41,040	
Structural Excavation Pier 2	201	CY	$350	$70,350	Performed During Shaft Installation
Gravel Backfill Pier 1 and 3	573	CY	$20	$11,460	
			Total	$332,210	

Item 5 Analysis

Bridge Supersrtucture

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Precast Elements & Cable Stay	42588	SF	$250	$10,647,000	
Bearings	48	EA	$3,000	$144,000	
Concrete Barrier	1,496	LF	$175	$261,800	
BP Railing	728	LF	$100	$72,800	
Expansion Joints	2	EA	$50,000	$100,000	
Utilities	728	LF	100	72,800	
Pedestrian Railing	655	LF	$200	$131,000	
Throw Fence	140	LF	$300	$42,000	
			Total	$11,471,400	

Item 6 Analysis

Approach Slab-47' x 25' x 2 each

Unit Price- $200/SY

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Approach Slabs	261	SY	$200	$52,200	

Item 7 Analysis

Bridge Demolition **Surface Area- 490'x25'= 12,250 sf (1361 SY)**

Unit Price- $150/SY

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
Existing Bridge Demolition	1361	SY	$150	$204,150	

Item 8 Analysis

Approach Fills

South Approach - Length- 300' x Average Height - 9' High

North Approach- Length - 430'x Average Height - 13' High

MSE Wall (South Approach)- 300' x 9' x 2 sides = 5400 SF

MSE Wall (North Approach)- 430' x 13' x 2 sides = 11,180 SF

Description	Quantity	Unit of Measure	Unit Price	Total	Notes
MSE Wall Approach	16580	SF	$40	$663,200	
Gravel Backfill	15351	CY	$20	$307,020	
HMA Pavement- 730' x 50'Wide x 6" T	1323	TON	$100	$132,300	
Moment Slab Barrier- 730' Length x 2 sides	1460	LF	$125	$182,500	
BP Barrier	730	LF	$100	$73,000	
			Total	$1,358,020	

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