



# SLIDE RIDGE CULVERT REPLACEMENT PROJECT

## Bridge Feasibility Report and Alternatives Analysis

July 2020 | Final Report





# Bridge Feasibility Report and Alternatives Analysis

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Appendix E – Preliminary Geotechnical Report

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# 1.0 Introduction

The Slide Ridge Culvert Replacement Project involves the replacement of the existing 96-inch corrugated metal pipe culvert beneath South Lakeshore Road with a structure that will give sediment-laden flows unobstructed passage under the roadway and towards Lake Chelan. The current culvert is regularly blocked with sediment and debris. The debris flow overtops the sediment basin and roadway, causing flooding and blockage of the roadway.

Slide Ridge historically produces frequent and violent flows of water, soil and debris during rain events on an almost annual basis. Flows are discharged down a combined natural and manmade drainage channel that empties into a sedimentation basin on the west (uphill) side of South Lakeshore Road. An existing corrugated metal culvert is intended to convey the material under the road along its historical path to the lake.

Frequent removal of slide material and channel maintenance is costly for the County and the reoccurring overtopping is a major safety hazard to the public. This project aims to eliminate or significantly reduce the need for cleanout of the drainage channel, catch basin, and roadway.

The goal of this study, and focus of this report, is to evaluate roadway alignment alternatives, identify site constraints, and establish hydraulic, geotechnical, civil, structural, and environmental/permit requirements in order to develop a preferred roadway crossing replacement concept, which supports conveyance of the material to the lake. This report documents the steps taken to arrive at the preferred alternative and presents the findings and recommendations for a bridge feasibility study.

## 1.1 PROJECT BACKGROUND

### 1.1.1 Existing Conditions

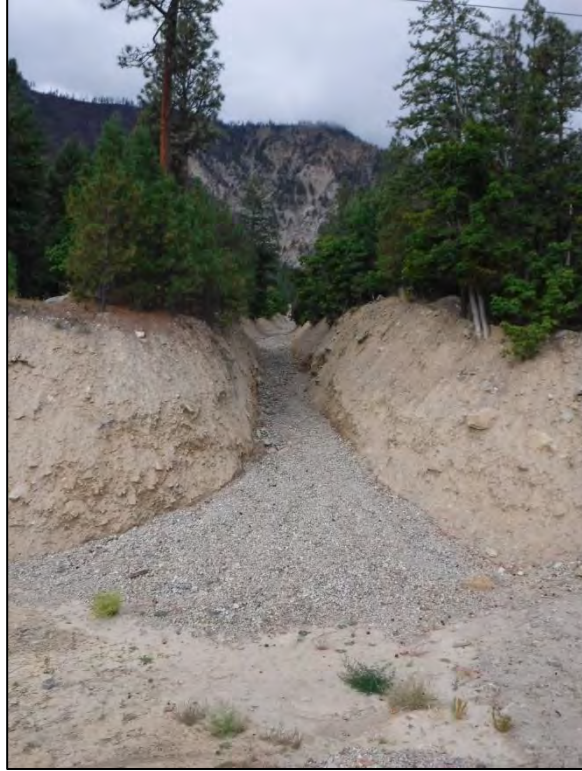
The project is located along South Lakeshore Road to the west of Lake Chelan approximately 13 miles northwest of the City of Chelan, just south of Shrine Beach, in Chelan County, Washington. This roadway is the only year-round north-south corridor for residents along the west side of Lake Chelan. A location, vicinity, and site map is shown in Figure 1.

In the *Final Comprehensive Flood Hazard Management Plan (2017)*, this project is identified as Action Item No. 15 to mitigate regular occurrences of the costly cleanup and safety concerns from flash flooding along South Lakeshore Road (Road Log No. 94710). Slide Ridge, also known as Granite Ridge, is a ridge characterized by steep rock outcrops split by loose rock slopes with little areas of soil or vegetation. Rainfall on the ridge results in water eroding and dislodging rock and mobilizes loose rock down the slope to the apex of the alluvial fan.

In 1995, an earthen engineered drainage channel was constructed in the slide area from the apex to South Lakeshore Road in an attempt to protect public and private property downstream. The drainage channel conveys material to a sedimentation basin on the west side of the road, where a 96-inch corrugated metal pipe culvert is intended to convey material under the roadway along its historical path to Lake Chelan. Pictures of the drainage channel and sedimentation basin are provided in Figure 2 and Figure 3, respectively. A picture of the 96-inch culvert is provided in Figure 4.



Figure 1: Location, Vicinity, and Site Maps for the Slide Ridge Culvert Replacement Project



**Figure 2: Drainage Channel as Seen from South Lakeshore Road (Looking West)**



**Figure 3: Sediment Basin Looking South From Mouth of Channel**



**Figure 4: 96-Inch Culvert from the West Side of South Lakeshore Road**

### **1.1.2 Debris Flows Events**

The existing basin is capable of holding approximately 4,000 cubic yards of sediment, based on a 2017 survey by Chelan County. During significant debris flow events, the culvert is inadequate to convey the material and becomes blocked. The volume overwhelms the basin and overtops the road, resulting in safety hazards, road closures and costly cleanups for removal of the material. Figure 5 and Figure 6 provide examples of the magnitude of debris deposited after an October 2017 event.

Over the last 15 years, these events have occurred every 1 to 4 years, with anywhere from 1,050 to 15,900 cubic yards of debris being removed after each event. These events restrict access along South Lakeshore Road until access can be regained on an unpaved single-lane primitive bypass road immediately to the east of South Lakeshore Road. The bypass road is shown in Figure 7 and is used until the main road can be cleaned. This can take more than one to two weeks, depending on conditions.



**Figure 5: Deposited Debris in Drainage Channel after October 2017 Event (Looking West)**



**Figure 6: Deposited Debris in Sediment Basin after October 2017 Event, Looking South from Mouth of Drainage Channel**



**Figure 7: Sediment Basin to Left (West), Bypass Road to Right (East), Looking North after Debris Cleared from Roadway**

## **1.2 LIST OF RELEVANT PRIOR STUDIES AND REPORTS**

The following are relevant prior studies and reports that were reviewed and/or performed to aid in the development of this study. The latter two were conducted as part of this project to support the design of the culvert replacement structure.

- *Final Environmental Impact Statement Chelan County Public Works Slide Ridge Control Channel* (1993), prepared by Hammond, Collier & Wade-Livingstone Associates, Inc.
- *Chelan County Comprehensive Flood Hazard Management Plan* (October 2017), prepared by Tetra Tech
- *Slide Ridge Debris Flow – Alternatives Analysis Hydraulic Report - Revised* (June 2019), prepared by Indicator Engineering
- *Draft Geotechnical Report Slide Ridge Culvert Replacement South Lakeshore Road, Lake Chelan, Chelan County, Washington* (February 2019), prepared by Pan Geo

## **1.3 SUMMARY OF THE HYDRAULIC ALTERNATIVES ANALYSIS**

The *Slide Ridge Debris Flow – Alternatives Analysis Hydraulic Report - Revised*, dated June 2019, was completed in support of the design and construction of the project. It summarizes the hydrologic, hydraulic, and geomorphic analyses of the Slide Ridge debris with a focus on the South Lakeshore Road crossing and conveying the debris material towards the lake. The report recommended three potential alternatives (or a combination of these alternatives) to manage the debris flows.

- **Conveyance to the lake:** This alternative proposes the construction of a larger channel opening at the road crossing to convey the material towards the lake. Portions of the upstream and downstream berms will need to be enhanced to prevent overtopping in this alternative.
- **Debris retention:** This alternative proposes to use retention basin(s), net(s), and/or other structure(s) to capture material before the road crossing.
- **Modifications to operations and maintenance:** This alternative proposes to eliminate the check dams and/or routinely clean out the channel to reduce stored material in the channel. This alternative also proposes to construct retaining structures that retain larger material to increase the water content and fluidity of the material remaining in the channel.

## 1.4 ALTERNATIVES DESIGN WORKSHOP

On October 26, 2018, the design team hosted an Alternatives Design Workshop. During the half-day workshop, key personnel from each discipline met with County staff to present their preliminary study findings, discuss potential design alternatives, and establish key design criteria. The alternatives presented and discussed are summarized below. Key design criteria are discussed in Section 2.0. Bridge alternatives considered at the workshop and notes from the workshop are provided in Appendix D.

### Hydraulic Alternatives

- Conveyance to the lake
- Debris retention
- Modifications to operations and maintenance

### Roadway Alternatives

- Maintain roadway on current alignment and raise roadway to provide increased vertical channel clearance (carried forward as Roadway Alignment 1)
- Shift the roadway toward the lake and construct a new road on the current bypass road (carried forward as Roadway Alignment 2)

### Bridge Alternatives

- Prefabricated precast concrete three-sided structure (also called a “bottomless culvert”)
- Prestressed girder superstructure with traditional cast-in-place concrete abutments
- Prestressed girder superstructure with soldier pile wall abutments – top-down construction (Carried forward as Bridge Alternative 2)
- Prestressed girder superstructure with spread footing abutments on structural earth wall abutments
- Prestressed girder superstructure with spread footings set back on sloped channel banks (Carried forward as Bridge Alternative 1)

## Superstructure Alternatives

- Deck bulb tee prestressed concrete girders
- Voided slab prestressed concrete girders
- Cast-in-place concrete slab
- Precast concrete three-sided structure

## Geotechnical Alternatives

- Spread footings
- Driven H-Piles
- Drilled shafts
- MSE abutment walls

As a result of the discussion at the Alternatives Design Workshop, it was decided to move forward with the conveyance option design, replacing the culvert with a bridge crossing that would better convey the debris flow material to the lake. Two roadway alignments were evaluated. The two preferred bridge alternatives were the prestressed concrete girder superstructure on spread footings set back on sloped channel banks (Bridge Alternative 1 - Longer Span, Spread Footings) and the prestressed concrete girder superstructure on soldier pile wall abutment (Bridge Alternative 2 – Shorter Span, Soldier Pile Abutment). These two bridge alternatives are further evaluated in the feasibility study, together with the various roadway and geotechnical alternatives. The preferred alternatives selected for further study are discussed in detail in Sections 3 and 4.

## 1.5 SUMMARY OF GEOTECHNICAL INVESTIGATION

The *Draft Geotechnical Report Slide Ridge Culvert Replacement South Lakeshore Road, Lake Chelan, Chelan County, Washington*, dated February 2019, was completed to assist with development of the project and is located in Appendix E. It summarizes the geology, seismicity, findings from site reconnaissance, and geotechnical requirements for the project.

Geotechnical explorations (drilled borings) were performed. Boulders were encountered in both boring locations. On the north side of the channel, the first attempt encountered drilling refusal at 10 feet and the second at 46.3 feet. On the south side, the first attempt refused at 18.5 feet and the second at approximately 33.0 feet. Sample recovery was generally poor, as expected, given the cobbles and boulders in the alluvial fan.

The presence of large boulders are expected to pose a significant construction risk and, as a result, could be a driving factor in considering the preferred alternative. Geotechnical requirements and constraints are discussed in greater detail later in this report in Section 2.4.

# 2.0 Design Criteria and Standards

## 2.1 CODES AND STANDARDS

- Chelan County Code, Chapter 8: Roads and Bridges, January 2019



- Chelan County Code, Chapter 11.86: Geologically Hazardous Areas Overlay District (GHOD), January 2019
- Washington State Department of Transportation (WSDOT) Design Manual M22-01, July 2018
- WSDOT Bridge Design Manual (LRFD) M23-50.18, June 2018
- WSDOT Standard Specifications for Road, Bridge and Municipal Construction M41-10, 2018
- AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2017
- American Association of State Highway and Transportation Officials (AASHTO) Guide Specifications for LRFD Seismic Bridge Design, Second Edition, 2011, with Interim Revisions
- AASHTO LRFD Bridge Construction Specifications, Third Edition, 2010, with Interim Revisions
- AASHTO Policy on Geometric Design of Highways and Streets

### 2.3 HYDRAULIC DESIGN CRITERIA

The hydraulic design criteria are documented in the Slide Ridge Debris Flow – Alternatives Analysis Hydraulic Report – Revised (6/13/2019), which is provided in Appendix F. Hydraulic design criteria relevant to the bridge crossing is summarized in this section.

#### 2.3.1 Design Debris Flow

The recommended design debris flow is the 100-year return period event, which corresponds to a 20,000-cubic-yard debris volume. As a point of reference, the current sedimentation basin is capable of holding 4,000 cubic yards, or a three-year return period event.

#### 2.3.2 Channel Geometry

The recommended channel geometry to convey the 100-year debris flows to Lake Chelan with reduced risk of overtopping South Lakeshore Road is provided in Table 1. The horizontal channel alignment at the bridge crossing is shown in Figure 8.

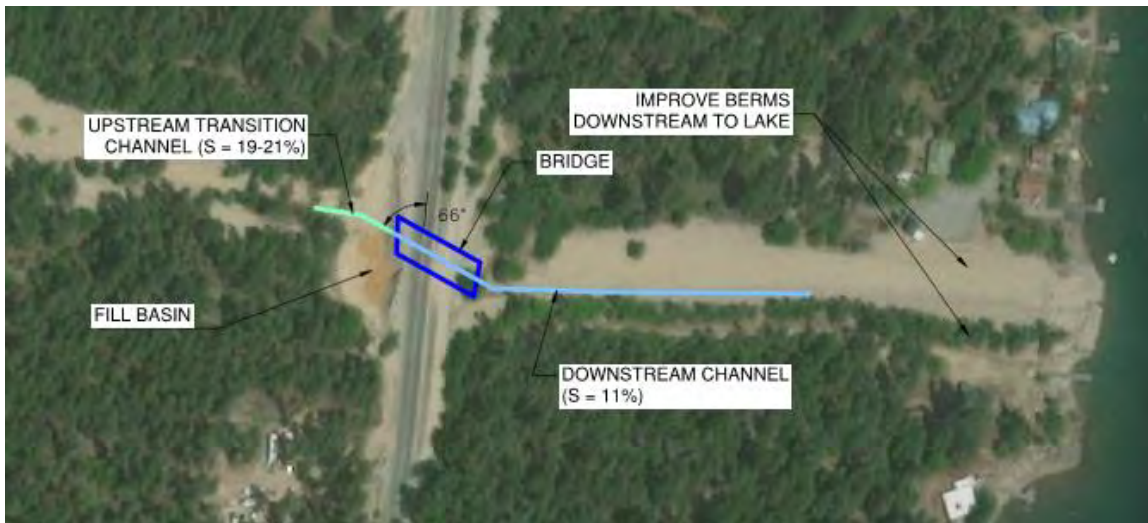
**Table 1: Conceptual Channel Configuration**

Section	Upstream Channel	Under Bridge	Downstream Channel
Channel Slope	19-21%	11%	11%
Channel Bottom Width (feet)	20	40*	30
Channel Side Slope	1:1	Vertical*	1:1
100-Year Event Depth of Material (feet)	16	14.9	15

\*Hydraulic analysis also confirmed the downstream channel section can be maintained under the bridge in lieu of the vertical channel walls. This channel section under the bridge would apply with the spread footings alternative set back on sloped channel banks.

Channel work will need to occur upstream and downstream of the bridge structure to tie in the upstream and downstream reaches and to construct the channel section beneath the roadway crossing. Work will also

include filling the existing sedimentation basin, plugging or removing the existing culverts, regrading the downstream channel to the lake, and elevating portions of the berms. The work elements to support the bridge crossing are described in the hydraulic report. Planning level cost estimates are provide below in Section 5.0.



**Figure 8: Conceptual Layout of Proposed Improvements for Conveyance Alternative (from Hydraulic Technical Report)**

### 2.3.3 Structure Freeboard and Debris Collision

The hydraulic analysis recommends a design freeboard (vertical clearance between bridge soffit and top of debris flow) of 3 to 5 feet to prevent debris collision with the bridge structure. However, it also states that boulder collisions are possible and should be considered in design of the bridge structure. The snout of the debris flow could carry boulders upwards of 4 feet in diameter traveling at 20 to 26 feet per second. The design should consider elements such as full depth intermediate diaphragms and/or steel armoring to mitigate damage from boulder impacts.

## 2.4 GEOTECHNICAL DESIGN CRITERIA

*The Draft Geotechnical Report Slide Ridge Culvert Replacement South Lakeshore Road, Lake Chelan, Chelan County, Washington, is provided in Appendix E. The relevant criteria are summarized below.*

### 2.4.1 Geologic Setting and Site Seismicity

Slide Ridge is located on an uplifted bedrock complex of the Cascade Range and is found to be not as seismically active as regions west of the Cascades. The nearest potentially active fault to the site is the Class B Straight Creek/Evergreen Fault system, about 50 miles west of the site. From a probabilistic design point of view, the fault responsible for Earthquake Ridge up the Entiat is considered not active due to it still being unidentified. However, if it was considered, the seismic design criteria would be similar to the Straight Creek/Evergreen Fault system for this site. The site is classified as Site Class D – Stiff Soil.

### 2.4.2 Geologic Hazards

The project site was evaluated for liquefaction potential and it was found that liquefaction was not to be expected. Seismic-induced landslides were also investigated. It was found that there was no historical proof of this occurring at Slide Ridge, and although evidence suggests it may be possible, considering such an event in design would probably be overly conservative and certainly cost prohibitive.

### 2.4.3 Foundation Recommendations

The geotechnical conditions of the project site allow for both deep and shallow foundations to be feasible. However, the presence of boulders in both test boring locations indicates a relatively high risk of constructability issues for deep foundations in the form of obstructions during pile driving.

Shallow spread footing foundations may be used if measures are taken to prevent scour, such as armoring and channel lining. The shallow foundations should bear in the medium dense to dense gravel and sand at or below approximately 10 feet below the existing roadway surface elevation.

Deep foundations will likely encounter obstructions such as cobbles and boulders. Drilled shafts are advantageous in this case due to their ability to penetrate through the obstructions; however, they do have a higher cost and require a specialty contractor to install them. Due to the remote location and expected skill required, it is not recommended for this small project. High-displacement driven piles, such as cast-in-place driven shell piles or precast concrete piles are not recommended as they may refuse on shallow obstructions. Low-displacement piles, such as heavy H-pile sections with driving shoes, are recommended and are likely to obtain better penetration.

## 2.5 ROADWAY DESIGN CRITERIA

The roadway design criteria are summarized in Table 2 below.

**Table 2: Roadway Design Criteria**

Parameter	Value
Roadway Classification	Major Collector
Design Average Daily Traffic (ADT)	427
Posted Speed Limit	35 mph
Travel Lanes	(2) 12'-0"
Shoulders	(2) 7'-0"
Roadway Width	38'-0"
Bike Lanes	None
Sidewalks	None
Total Roadway Width	38'-0"
Barrier	TL-4
Roadway Surface	HMA
Bridge Surface	HMA

## 2.6 STRUCTURAL DESIGN CRITERIA

### 2.6.1 Dead and Live Loads

Dead loads are produced by the self-weight of the structural components of the bridge and superimposed loads, such as utilities.

Live loads are transient loads that the structure experiences on a temporary basis. The loads in Table 3 describe the live loads considered in design.

**Table 3: Live Loads**

Live Load	Criteria
Pedestrian	<ul style="list-style-type: none"> <li>No pedestrian facilities provided</li> </ul>
Vehicle	<ul style="list-style-type: none"> <li>HL-93 vehicle live load, as per article 3.6 of AASHTO LRFD</li> </ul>
Wind	<ul style="list-style-type: none"> <li>Designed for wind on bridge and vehicles, as per Article 3.8 of AASHTO LRFD</li> </ul>
Temperature	<ul style="list-style-type: none"> <li>The temperature range considered shall be 0°F to 100°F, as per WSDOT BDM article 3.16.6</li> <li>Installation temperature of 64°F, as per WSDOT BDM article 3.5</li> </ul>
Boulder Impact	<ul style="list-style-type: none"> <li>Mitigate damage from impact from a 4-foot-diameter boulder traveling at 20 to 26 feet/second during a debris flow event shall be considered, as per Hydraulic Technical Report</li> </ul>

### 2.6.2 Seismic Loads

The seismic design parameters determined for a seven percent probability of exceedance in 75-year event (approximately a 1,000-year return period event) at the Slide Ridge Culvert Replacement site are based on the draft geotechnical report and summarized in Table 4.

**Table 4: Seismic Design Parameters**

Parameter	Value
Site Class	D – Stiff Soil
Design Spectral Response Acceleration at a Period of 0 second, $A_s$	0.212 g
Design Spectral Response Acceleration at Short Periods, $S_{DS}$	0.477 g
Design Spectral Response Acceleration at a Period of 1 second, $S_{D1}$	0.237 g
Seismic Performance Zone	2
Seismic Design Category	B

### 2.6.3 Soil Design Parameters

Soil design parameters, including active, passive, at rest, surcharge, seismic, and bearing pressures, are considered in design as per the recommendations from the geotechnical draft report provided in Appendix E.

## 3.0 Roadway Alignment Alternatives

Two roadway alignment alternatives were developed. Goals for the alternatives are to accommodate construction of the bridge, maintain traffic along South Lakeshore Road throughout the duration of construction and facilitate modification and vertical clearances for the drainage channel. This section will present and discuss roadway alternatives. Civil roadway figures are provided in Appendix A. Alternatives 1 and 2 are shown in Figures 9 and 10, respectively.

### 3.1 ROADWAY ALTERNATIVE 1: EXISTING HORIZONTAL ALIGNMENT

This alternative maintains the roadway on its existing horizontal alignment. The roadway profile would need to be elevated to provide sufficient freeboard above the debris flow in the channel. Fill slopes would extend into the existing sedimentation basin within the right-of-way and would require some excavation of the bypass road embankment. This alternative remains in the right-of-way, maintains existing driveways, and would not require temporary construction easements. Some reconstruction of the bypass road embankments would be required to tie in the existing driveways.

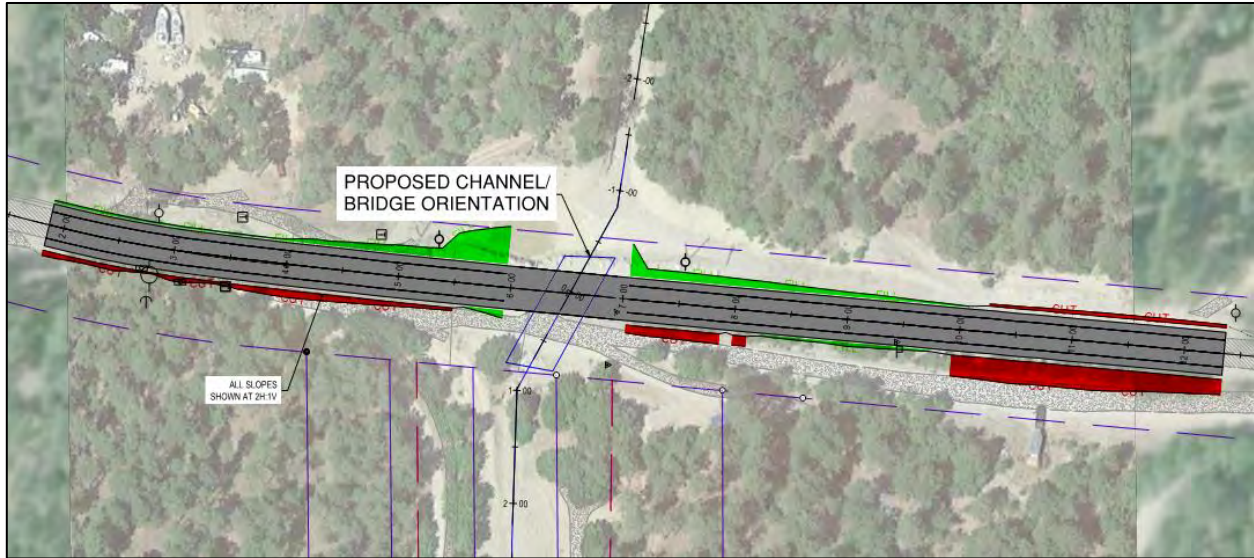
The bridge crossing would be about 50 feet to the north of the existing culvert location to align the downstream and upstream channel alignment. During construction, traffic could be temporarily routed onto the existing bypass, or the bridge could be constructed in phases. The existing bypass road may require minor improvements, such as temporary paving and grading, to maintain traffic along South Lakeshore Road. The bypass road will also likely need to be shored adjacent to the proposed bridge during construction, especially during footing construction, which requires large amounts of excavation. However, phased construction is typically more costly and time consuming, therefore it is assumed that using the existing bypass road would be preferred.

#### Pros

- Maintains driveway access
- No right-of-way impacts

#### Cons

- Requires substantial fill and roadway reconstruction
- Traffic rerouted on bypass road or phased construction is required – greater traffic impacts are expected due to a one-lane detour



**Figure 9: Roadway Alternative 1 Relative to the Channel. Red and green areas represent the cut and fill extents, respectively.**

### **3.2 ROADWAY ALTERNATIVE 2: EAST OF EXISTING HORIZONTAL ALIGNMENT**

This alternative moves the final roadway horizontal alignment to the east, toward the lake, within the existing right-of-way. Traffic can remain on the existing roadway during construction while the bridge is constructed, with limited closures required to construct roadway transitions.

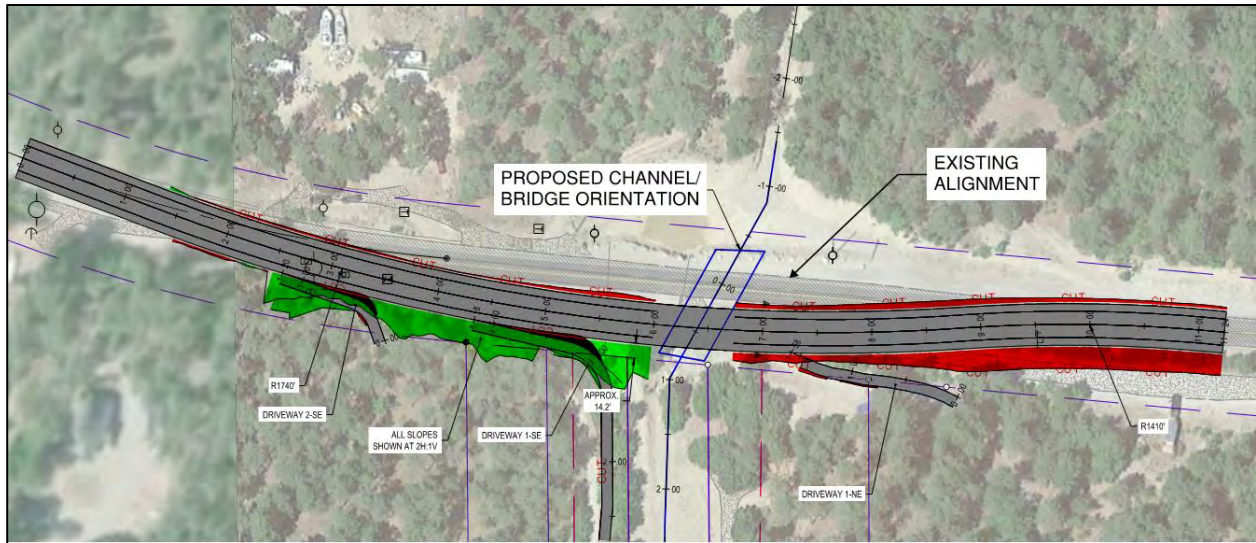
The channel gains vertical clearance as it slopes toward the lake, so this alternative does not require the roadway to be significantly raised in order to provide sufficient freeboard beneath the bridge. Instead, much of the bypass road would be excavated and lowered. The grading associated with this alternative would result in right-of-way impacts to the adjacent properties and removal of trees and vegetation. Temporary construction easements would be required to grade on private property and reconstruct driveway entrances.

#### **Pros**

- Maintains traffic along existing alignment during construction
- Reduced fill and roadway reconstruction

#### **Cons**

- Likely need to build walls or purchase right-of-way to construct roadway fills and reconstruct driveways
- More Tree removal will be required
- Disruptions to private property for reconstruction of driveways and roadway fills are greater for this alternative compared to Alternative 1



**Figure 10: Roadway Alternative 2 Relative to the Channel. Red and green areas represent the cut and fill extents, respectively.**

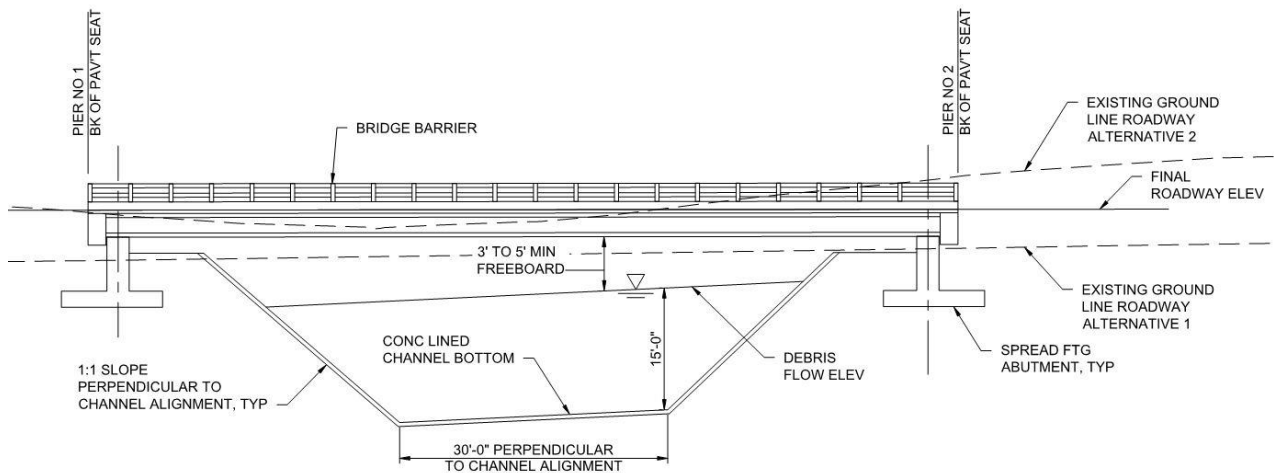
## 4.0 Bridge Alternatives

This section presents and discusses the bridge alternatives. Two bridge alternatives were developed: a long span option (bridge supported on spread footings) and a short span option (bridge supported on soldier piles). Both are compatible with either of the roadway alignment alternatives presented in Section 3.0. Conceptual structural drawings are provided in Appendix B.

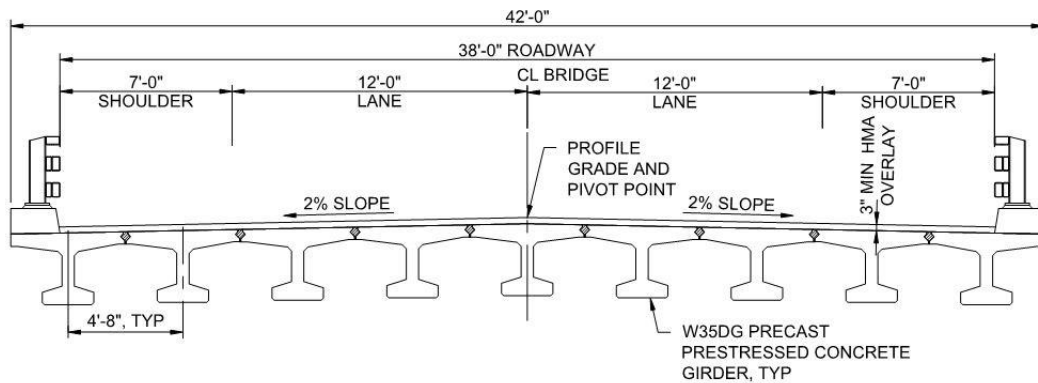
### 4.1 BRIDGE ALTERNATIVE 1: LONGER SPAN, SPREAD FOOTING

This alternative proposes a longer single-span bridge supported on spread footing abutments. Alternative 1 is shown in Figures 11 and 12. The spread footings would be set back from sloped channel banks. Under the bridge, the channel would maintain 1V:1H slopes and a 30-foot bottom width perpendicular to the channel alignment. The top width of the channel would be approximately 82 feet, measured along the roadway alignment, and the bridge span would be approximately 108 feet.

W35DG deck bulb tee girders are the most economical girder type that balances cost and superstructure depth at this span length. The top flange of the girders can serve as the final driving surface or receive a cast-in-place or HMA (hot mix asphalt) overlay. We understand an HMA overlay is preferred by the County and is included in the cost estimate for this alternative. This type of overlay balances, cost, construction duration, and long term durability and maintenance.



**Figure 11: Bridge Alternative 1 Elevation**



**Figure 12: Bridge Alternative 1 Typical Section**

The native soils at the site are capable of maintaining a 1:1 slope. However a concrete lining is recommended at the bridge, to aid in channel maintenance and to prevent potential scour and erosion of the channel banks, from both channel maintenance and natural forces. A concrete lining along the channel would protect the channel slopes from maintenance equipment over excavating the channel while clearing deposited material in the channel beneath the bridge. It would prevent undermining and eroding the 1:1 soil slope required for the shallow spread footings to remain stable. Sediment and debris impact may mar the surface of the lining, however, it would not compromise its function of protecting the foundations and channel slope.

The channel lining would extend 40 feet beyond the bridge in the upstream and downstream directions. Concrete toes may need to be included at the upstream edge of the concrete channel lining to ensure scour does not undermine the lining and to reinforce the edge of the lining against maintenance equipment catching on it. Rip rap may also be used as an alternative to concrete channel lining. If a channel lining is not included, maintenance and clearing of the channel would be required to ensure that the 1:1 slope is maintained and that soil is not removed beneath the shallow spread footings which would reduce their bearing capacity.



While the 3 to 5 feet of freeboard clearance above the 100-year debris flow elevation reduces the likelihood of boulder impacts, it does not completely eliminate the possibility of it occurring. Boulders may project above the surface of the debris flow or ride on the snout of the flow. Protection of the upstream girder is recommended to prevent damage from boulder collisions. Protection alternatives include the inclusion of full-height intermediate diaphragms, adding steel plate armoring to the girders, and including a sacrificial member upstream. These girder protection alternatives are discussed in detail in a later section.

**Pros:**

- Ease of abutment construction, due to straight forward nature of shallow foundation.
- Less risk of encountering obstructions than driven pile foundations.
- Shallow spread footing foundations do not require specialized drilling/driving capabilities.
- Minimal change to channel shape under bridge compared to either side of the roadway crossing.
- Trapezoidal channel shape is advantageous in maintaining the movement of the debris flow.
- If concrete channel lining is used, it would ensure the foundations are not undermined during channel maintenance.
- If concrete channel lining is used, it would ease maintenance by preserving the channel section and not having to reestablish the channel section after an event.

**Cons:**

- Longer span compared to Alternative 2 and, consequently, more deck area to maintain long term.
- Depending on footing depth and potential obstructions, large excavations and/or shoring may be required for abutment construction.
- Additional cost from slope protection, such as the concrete channel lining.
- Requires more channel excavation than Alternative 2.

#### **4.2 BRIDGE ALTERNATIVE 2: SHORTER SPAN, SOLDIER PILE ABUTMENT**

This alternative would construct a shorter span bridge supported on a hybrid soldier pile wall integral abutment. The bridge superstructure would be supported on driven or augured H-piles, which could be made integral to eliminate bridge bearings and unnecessary joints. Under the bridge, the channel would transition to vertical walls with a 40-foot-wide bottom width. The bridge is intended to be constructed in a top-down fashion to minimize excavation. The roadway can potentially be reopened sooner once the deck is complete and all channel work can occur beneath the bridge under traffic. The soldier pile walls serve as the temporary shoring to excavate the channel and the final bridge abutment walls. The construction sequence is illustrated in Appendix D, and summarized below.

1. Install soldier H-piles and pier caps.
2. Construct bridge superstructure and cast integral with pier caps. Construct barriers. It is possible to open the bridge to traffic after this step.
3. Excavate channel below in lifts and install temporary soldier pile wall lagging.
4. Once channel is excavated, install permanent shotcrete facing.

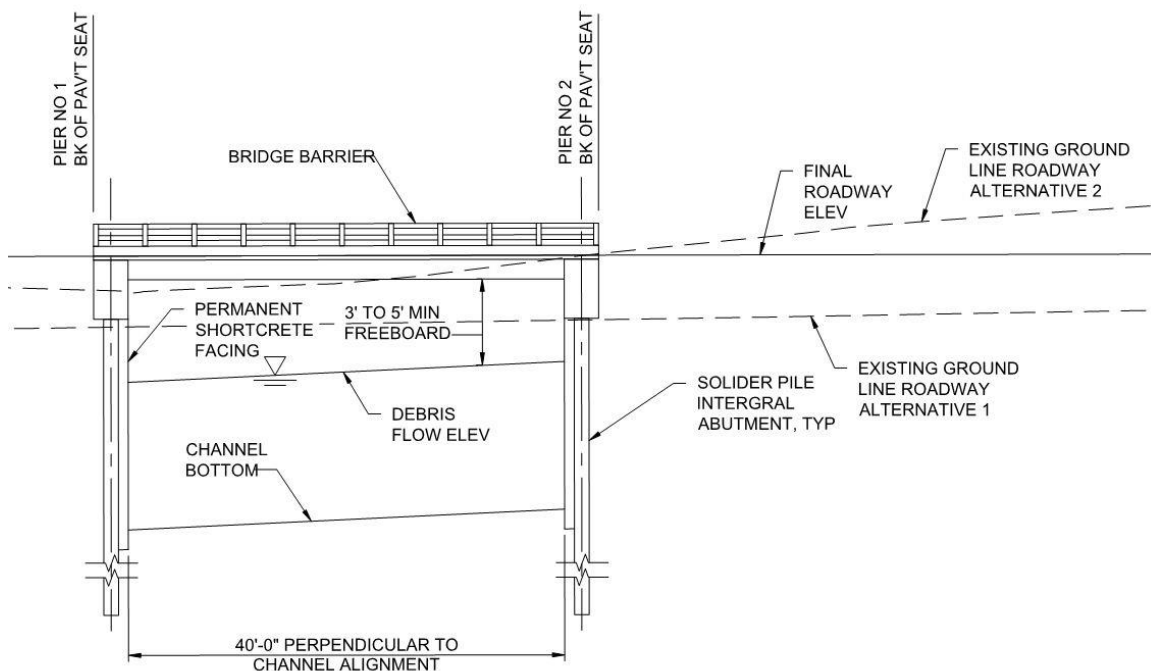
The bridge span would be approximately 52 feet. Two (2)-foot-deep prestressed precast voided slab girders are the most economical girder type for this short span. Deck bulb tee girders and a solid cast-in-place slab were also considered, but would add unnecessary structure depth. Voided slab girders have a shallow depth that improves vertical clearance and has the feature of serving as the final driving surface or can be overlaid with HMA or cast-in-place concrete. An HMA overlay is assumed in the cost estimate for this alternative.

Similar to the deck bulb tee girders of Bridge Alternative 1, it is recommended that the voided slab girders of Bridge Alternative 2 include consideration for boulder collision. Girder protection options considered include filling in the voids of the upstream girder, reinforcing the voids of the upstream girder, or providing a sacrificial member upstream of the bridge. These options are discussed in detail in a later section.

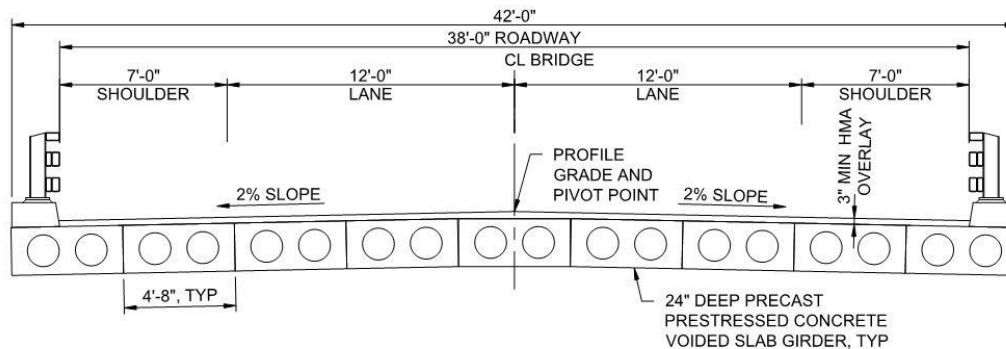
Sediment and boulder impact may mar the shotcrete surface of the abutment walls and wingwalls. Use of higher strength concrete and increased concrete cover would improve the long term durability of the structural elements. Larger chamfered edges would also aid in preventing large chips and spalls on corners. Given the geometry and areas of surfacing that could be abraded by the debris flow, it would not be economical or practical to armor the surfaces with steel.

Construction of the abutments for this alternative is expected to be difficult, given the presence of large boulders and cobbles encountered during geotechnical exploration. Encountering obstructions that need to be removed to install piles is a risk. Contract documents will need to clearly indicate this risk and this risk should be considered in selecting the preferred alternative.

If this option is considered, the geotechnical report recommends driven H-piles with driving shoes, as they have the largest chance of successfully penetrating boulders and cobbles among driven piles. Drilled shafts were not selected due to the increased cost and specialized contractor skills required at this rural site to deal with obstructions.



**Figure 13: Bridge Alternative 2 Elevation**



**Figure 14: Bridge Alternative 2 Typical Section**

**Pros:**

- Shorter span relative to Alternative 1 and less long term bridge deck maintenance.
- Top-down construction method reduces temporary excavations and allows the bridge to be open to traffic sooner.
- Local contractors may self-drive piles.
- Requires less channel excavation than Alternative 1.

**Cons:**

- Potential for pile damage and/or pile refusal during driving, higher risk of encountering obstructions to schedule and costs than spread footings.
- Channel geometry is required to change from sloped to vertical sides under bridge, decreasing the hydraulic efficiency.
- Rectangular channel geometry promotes smaller debris being deposited in channel.

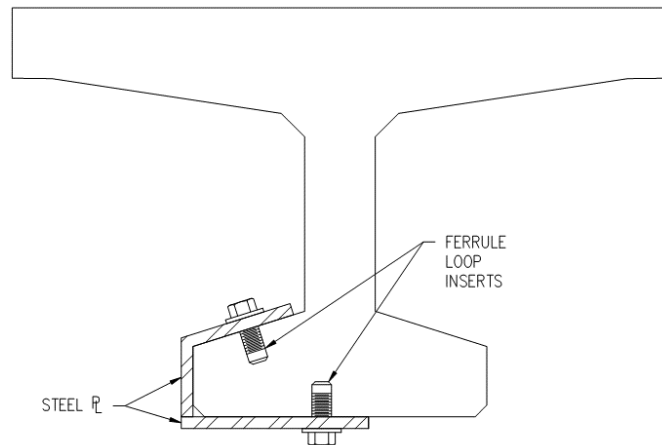
**4.3 BOULDER IMPACT PROTECTION**

The *Slide Ridge Debris Flow – Alternatives Analysis Hydraulic Report - Revised*, dated June 2019, indicates that the snout of a 100-year event debris flow may carry boulders up to 4 feet in diameter at up to 26 feet per second. While the new bridge will be located above the estimated height of the debris flow, it is recommended that both bridge alternatives include girder protection to prevent substantial damage from collisions with boulders. The upstream girder is considered to be at the highest risk of impact.

Possible options to protect the deck bulb tee girders of Bridge Alternative 1 included providing additional full-height intermediate diaphragms along the girder span, adding steel plate armoring around the exterior of the upstream girder, and/or providing a replaceable strike bar in front of the upstream bridge girder. The latter two options are shown in Figures 15 and 16.

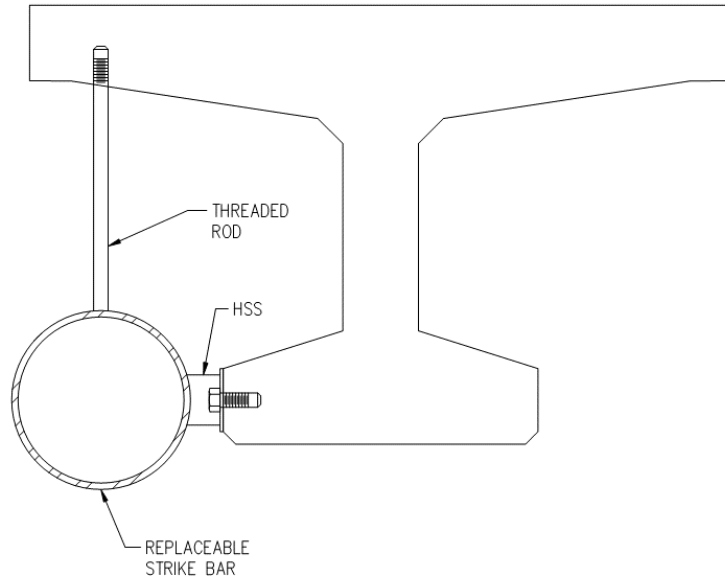
Providing additional full-height intermediate diaphragms along the girder span is recommended to brace the bottom flange of the girder against impact and to better distribute a collision load among multiple girders.

The bottom flange of the upstream girder can also be designed to resist the anticipated collision load. Steel armoring would protect the girder from local concrete damage. The steel plate armoring is shown in Figure 15. Ferrule loop inserts could be installed in the girder bottom flange, allowing the plates to be bolted in the field and replaced or removed for maintenance and inspection. The estimated cost for the inclusion of plates on the bottom flange of the upstream girder would be approximately \$15,000. While this option is substantially less expensive than the strike bar option described below, it requires the girder to resist the full load from a boulder collision at the location of impact, this could lead to some damage to the girder concrete beneath the steel plates.



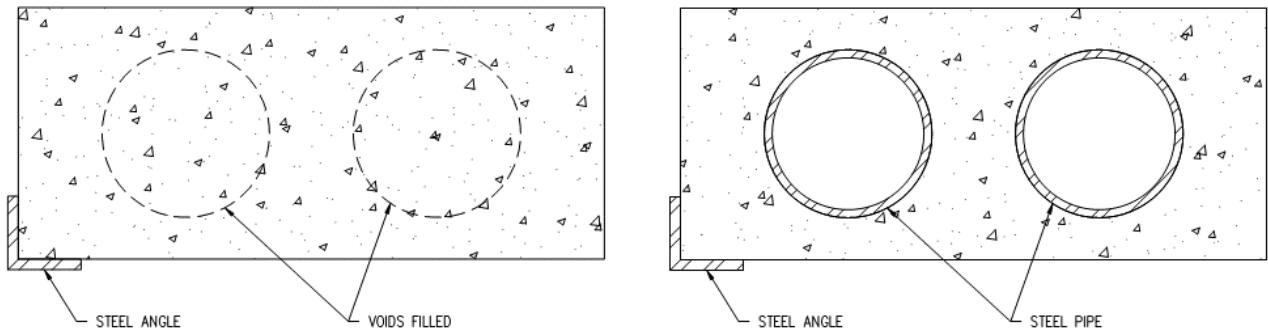
**Figure 15: Deck Bulb Tee Steel Plate Girder Protection**

The replaceable steel strike bar option is shown in Figure 16. It provides its own set of advantages and disadvantages. The strike bar would be connected to the girder at intermediate diaphragm locations, sharing any impact load across multiple girders. Additionally, deformation of a strike bar pipe from boulder impact would absorb the energy otherwise resisted by the girders. The replaceable strike bar is expected to be more expensive than the steel plate armoring, however the risk of damage to the girders is significantly reduced. It is estimated to cost approximately \$60,000 for initial installation and an estimated \$560 per linear foot of strike bar replacement if damaged.



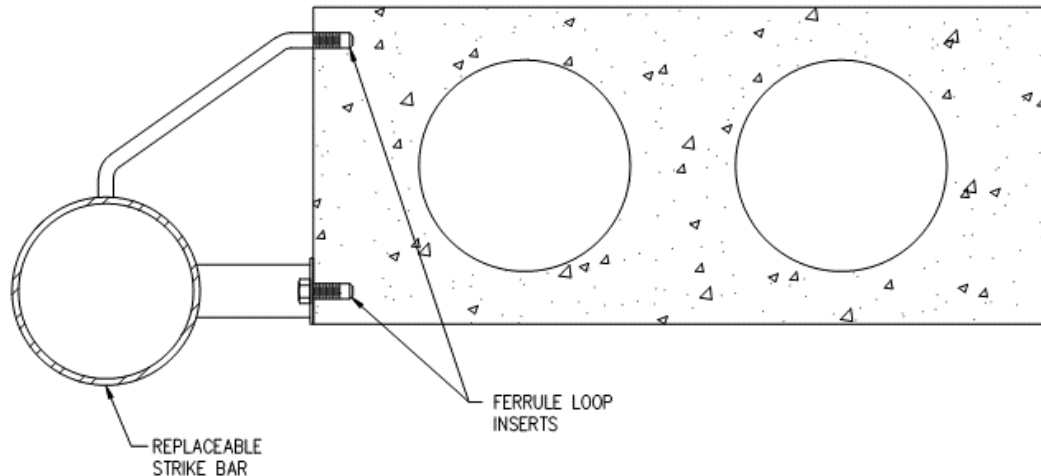
**Figure 16: Deck Bulb Tee Replaceable Strike Bar Girder Protection**

The options for protecting the voided slab girders of Bridge Alternative 2 include filling the voids of the upstream girder or reinforcing the void with steel pipe, and providing a steel angle armor and/or replaceable strike bar on the upstream girder. Constructing a solid void and reinforcing the void are shown in Figure 17. They reduce the risk of failure and collapsing the void in the event of a collision. Reinforcing the void with a pipe is expected to be the more expensive option of the two at approximately \$45,000, compared to \$15,000 for constructing a solid slab. This is mainly due to the price of steel in comparison to the cost of concrete required to fill the void.



**Figure 17: Slab Void Reinforcement Options**

The voided slab girders could also utilize a steel angle armor and/or replaceable strike bar on the upstream girder of the bridge, similar to the method described above for the deck bulb tee girders. These options are shown in Figures 17 and 18. This option would still present potential disadvantages concerning its ability to only protect the upstream girder. However, a replaceable member would mostly eliminate direct contact between the girder and boulders. The steel angle armor is anticipated to be the least expensive option for the voided slab protection, estimated to be approximately \$30,000.



**Figure 18: Voided Slab Replaceable Strike Bar Girder Protection**

Similar to the strike bar girder protection of the deck bulb tee girders, the impact load would be distributed to the adjacent voided slab girders through the joint connection. For this reason, grouted shear keys paired with the conventional welded tab connections are recommended. The strike bar would still decrease the potential of direct collision between the girder and a boulder, which would cause local damage to the concrete.

The girder protection system included in the cost estimate for Bridge Alternative 1 is the sacrificial steel strike bar member; the estimated cost is \$60,000. The concrete-filled void protection system is included in the cost estimate for Bridge Alternative 2; its estimated cost is \$15,000. The girder protection system estimated cost is included in the superstructure cost in Table 6.

## 5.0 Alternatives Cost Analysis

This section presents roadway and bridge cost estimates for each alternative and provides discussion of the key cost differences and cost drivers. Planning level cost for channel construction are also provided.

### 5.1 PRELIMINARY COST ESTIMATES

Planning-level cost estimates for each alternative are provided in Appendix C and are summarized below in Tables 5 and 6.

All cost estimates are in 2019 dollars. Cost estimates do not include the following costs:

- Sales Tax
- Costs associated with construction administration, management, engineering, and for inspection
- Costs associated with permitting
- Costs associated with replanting disturbed areas, applying geologic mitigation measures, or any other costs associated with meeting the requirements of Chapter 11.86 of the Chelan County Code.

## 5.2 ROADWAY COST COMPARISON

Construction costs alone for Roadway Alternatives 1 and 2 are similar, within \$57,000 or 7% of each other after including contingency and mobilization. When looking at total cost, right-of-way cost is the main cost driver that differentiates the two and roadway Alternative 2 becomes the more costly of the two.

**Table 5: Estimated Construction Cost for Roadway Alternatives**

	Roadway Alternative 1	Roadway Alternative 2
Roadway Construction	\$645,250	\$657,750
Right-of-Way	\$0	\$30,000
Mobilization (10% of Roadway)	\$64,525	\$65,775
Contingency (30%)	\$212,933	\$226,058
<b>Total</b>	<b>\$922,708</b>	<b>\$979,583</b>

## 5.3 BRIDGE ALTERNATIVES COST COMPARISON

In comparing the cost of the bridge alternatives, the following are cost drivers and key differences between different components.

### Substructure

- Bridge Alternative 1 substructure is mainly composed of footing concrete and excavation costs.
- Bridge Alternative 2 substructure is mainly composed of pile furnishing and driving.
- The cost of furnishing and driving a sufficient amount of steel piles is significantly higher than the shallow excavation costs required for a spread footing. Additional costs intended to address the removal of pile obstructions also contributes to the more expensive substructure cost of Bridge Alternative 2. This cost could be uncertain.

### Superstructure

- Bridge Alternative 1 has a span approximately twice as long as Bridge Alternative 2. This approximately doubles the amount and cost of the superstructure, bridge barrier, and boulder impact protection system in Bridge Alternative 1.
- Despite the overall superstructure cost being more expensive in Bridge Alternative 1, the superstructure costs per square foot are comparable with Bridge Alternative 1 at \$158 of superstructure per square foot while the superstructure cost per square foot of Bridge Alternative 2 is \$147.

### Channel Lining

- Concrete slope protection in the channel is only required in Bridge Alternative 1. The cost of the channel lining is included under Bridge Alternative 1 only.

**Table 6: Estimated Construction Cost for Bridge Alternatives**

	Bridge Alternative 1	Bridge Alternative 2
Substructure	\$225,749	\$775,981
Superstructure	\$712,200	\$317,835
Concrete Channel Lining Beneath Bridge	\$148,470	\$0.00
Mobilization (10%)	\$108,642	\$109,382
Contingency (30%)	\$358,518	\$360,959
<b>Total</b>	<b>\$1,553,580</b>	<b>\$1,564,158</b>
Bridge Square Footage	4,512 square feet	2,156 square feet
Bridge Cost per Square Foot (not including contingency)	\$344	\$558

#### 5.4 CHANNEL CONSTRUCTION COSTS

Planning level costs for construction associated with the channel are estimated at \$771,012, including 10% mobilization and a 30% contingency. These costs are summarized in Table 7. This construction cost may be reduced by up to \$99,025 (not including associated mobilization or contingency cost reduction) by reducing the initial height and freeboard of the downstream berms.

**Table 7: Estimated Channel Construction Cost**

	Channel Construction
Channel	\$539,169
Mobilization (10%)	\$53,917
Contingency (30%)	\$177,926
<b>Total</b>	<b>\$771,012</b>

These costs are for a system which conveys debris flow from the upstream channel, through a new bridge crossing of the road, and to the lake. Construction of this system includes the transition channel upstream of the road, an open channel beneath the new bridge, and improvements to the channel downstream of the road. Summaries of these construction items are described below and in the *Slide Ridge Debris Flow – Alternatives Analysis Hydraulic Report - Revised*:

- Transition channel upstream of the road. A conveyance channel will be constructed through the existing debris basin upstream of the road that maintains a relatively steep slope and similar bottom width to the existing upstream channel.
- An open channel with a new road crossing and bridge. Beginning at the upstream face of the bridge a new trapezoidal conveyance channel will be constructed through the road crossing. The latest design concept uses a constant channel slope from the upstream bridge face to the lake of 10 to 11%. The design uses a wider channel bottom to maintain peak depth of the debris flow.



- Improve the channel downstream of the road. Improvements to the channel downstream of the road are proposed to contain the debris flows. Channel grading will be performed to create a constant slope and section. Channel banks would be raised to contain the calculated 100-year debris flow.

The channel construction costs consist of cut and fill quantities for regrading about 1000 feet of channel. Specifics concerning the presented channel cost and potential cost reductions are included in the *Slide Ridge Debris Flow – Alternatives Analysis Hydraulic Report - Revised* and below:

The grading volumes are calculated to be 7,750 CY of cut and 22,500 CY of fill. The majority of the earthwork (82%) is downstream of the road, with 13% upstream and 5% under the road. The net result is a cut-fill volume imbalance requiring about 14,750 CY of imported material for channel construction. As a cost saving measure, the design assumes the existing debris basin is not filled with imported material during initial construction. Instead, it could be filled over time with material excavated from the channel from maintenance operations.

It is recommended to construct the upstream channel, the channel under the road, and the downstream channel improvements to the lake with 3.0-ft of freeboard. The existing channel downstream of the road does not have adequate capacity to convey the 100-year debris flow and would be likely overtopped, threatening adjacent properties. However, as a cost saving measure for initial construction, shorter berms can be constructed with no additional freeboard downstream of the road. This would reduce the fill volume (and corresponding import material) by about 4,000 CY. The berms may be raised to provide 3.0-ft of freeboard in the future using non-imported material from maintenance operations.

Import material costs may be further reduced by sourcing the import material for the channel grading from elsewhere at the Slide Ridge site. There are prior debris spoil sites along the upstream channel which could be used for borrow, while maintaining the capacity in the upstream channel.

The downstream channel grading may be further refined in the next phase of the design to account for interaction with road, bridge, and driveway grading. In particular, the driveway on the property to the south of the channel is within the easement and may require relocation or regrading to maintain its function. The downstream channel section assumes 1 to 1 side slopes, which is flatter than many sections of the upstream channel. Fill volumes would be further reduced if a steeper side slope is feasible for the downstream channel banks.

## 6.0 Alternative Analysis

This section describes the alternative analysis metrics used to arrive at the preferred project alternative. *[This section will be revised in the final report after meeting with the County and presenting the project to key stakeholders].*

### 6.1 ALTERNATIVE ANALYSIS METRICS

The alternatives analysis evaluates the alternatives by considering impacts and benefits to the community, together with project cost and risk. Table 8 (below) was developed as a comparison tool to show objectives and metrics and to identify a preferred alternative.

The metric for the evaluation of the criteria is described either qualitatively or quantitatively (cost). Each criterion is evaluated to assess whether the alternative has a positive impact, neutral impact, or negative impact with respect to the other alternatives. Positive impacts score +1 for the criteria, neutral impacts score 0, and negative impacts score -1.

The scores of each criterion are weighted such that each objective has the same weight. *[The County and Stakeholders may wish to weight certain objectives higher than others]*. Scores for each objective are added together to determine the total score used for finding an optimal balance for achieving all objectives.

**Table 8: Objectives for Alternative Analysis**

Objective	Criteria	Performance Parameter	Metric
Maintain good public relations	Minimize right-of-way impacts	Area of right-of-way impacts	Qualitative analysis
	Minimize traffic and access impacts	Access, traffic disruption, length of construction	Qualitative analysis
Improve channel function	Channel shape conducive to better debris flow	Channel cross-section shape	Qualitative analysis
	Channel profile conducive to better debris flow	Grade of downstream channel	Qualitative analysis
Constructability	Minimize construction risk	Ease of construction	Qualitative analysis
Minimize costs	Roadway construction costs		Total estimated construction dollars
	Bridge construction costs, incl. channel beneath bridge		Total estimated construction dollars

## 6.2 ALTERNATIVE ANALYSIS RESULTS

The results for each objective and criteria are presented in Table 9.

**Table 9: Alternative Analysis Results**

Evaluation Area			Alternative			
			Roadway Alternative 1 (Exist. Align.)		Roadway Alternative 2 (East of Exist. Align.)	
Objective	Criteria	Weight	Bridge Alt 1 (Deck Bulb Tee, Spread Footing)	Bridge Alt 2 (Slab Girder, Soldier Pile)	Bridge Alt 1 (Deck Bulb Tee, Spread Footing)	Bridge Alt 2 (Slab Girder, Soldier Pile)
Maintain good public relations	Minimize right-of-way impacts	0.5	+1	+1	-1	-1
	Minimize traffic and access impacts	0.5	0	0	+1	+1
Improve channel function	Channel shape	0.5	+1	-1	+1	-1
	Channel profile	0.5	0	0	0	0
Constructability	Minimize construction risk	1.0	+1	-1	+1	-1
Minimize costs	Roadway constr. costs	0.5	+1	+1	-1	-1
	Bridge constr. costs, incl. channel beneath bridge	0.5	0	0	0	0
<b>Weighted Total Score:</b>			<b>+2.5</b>	<b>-0.5</b>	<b>+1.0</b>	<b>-2.0</b>

### **6.2.1 Roadway Alignment Alternative Comparison**

The following three criteria apply to the roadway alignment alternatives:

- Minimize right-of-way impacts: Roadway Alternative 1 does not require permanent impacts to driveways and/or construction easements, while Roadway Alternative 2 may require one or both. Roadway Alternative 2 would also remove more trees. Alternative 1 is scored positively and Alternative 2 is scored negatively.
- Minimize traffic and access impacts: Roadway Alternative 1 requires closing the road and opening a one lane detour onto the existing unpaved bypass road during construction of the bridge, while Roadway Alternative 2 maintains traffic on the existing paved road. However, due to the low ADT of South Lakeshore Road, using the existing unpaved bypass is not considered a major impact to traffic. As such, Alternative 1 is scored neutrally and Alternative 2 is scored positively.
- Channel profile: Both alternatives provide the same channel profile and vertical clearance. Both are scored neutral.
- Roadway construction costs: Roadway Alternative 1 is less costly than Roadway Alternative 2. They are scored positively and negatively, respectively.

### **6.2.2 Bridge Alternative Comparison**

The following three criteria apply to the bridge alternatives:

- Minimize traffic and access impacts: Bridge Alternative 2 can be constructed in a top-down fashion that allows the bridge to be open to traffic sooner, while channel excavation occurs below. However, as mentioned earlier, the use of the existing unpaved bypass is not a major impact to traffic due to the low ADT of the road. This leads to the traffic and impacts score not being affected by the bridge alternative selected.
- Channel shape: A trapezoidal channel shape with lining is expected to perform better than a rectangular section. A rectangular channel shape promotes the settlement of finer debris in the channel, leading to more debris being deposited at the bridge after a flow event. Bridge Alternative 1 is scored positively and Bridge Alternative 2 is scored negatively.
- Minimize construction risk: Spread footing foundations are anticipated to be more constructible at this site and pose less of a risk. The presence of large boulders and cobbles may damage the piles or refuse the piles at shallower construction depths. Bridge Alternative 1 (spread footings) is scored positively. Bridge Alternative 2 (piles) is scored negatively.
- Bridge construction costs, including channel beneath bridge: The estimated cost of Bridge Alternative 1, including the concrete channel lining beneath the bridge, is comparable to the estimated cost of Bridge Alternative 2 (within \$11,000 of each other). As such, Bridge Alternative 1 and Bridge Alternative 2 are both scored neutrally.

## **6.3 PREFERRED ALTERNATIVE**

The combination of Roadway Alternative 1 and Bridge Alternative 1 is the recommended solution. Elevating the road on the current alignment and constructing a longer-span bridge founded on spread footings best achieves the project objective's.

## **6.4 ENVIRONMENTAL DOCUMENTATION**

Chelan County Staff will be responsible for preparation of any environmental permits required for this project. To support these efforts, the design team reviewed the previously prepared (1991) Environmental Impact Statement to determine how to proceed to obtain approval from the permitting agencies.

Step 1: Consult with the appropriate FEMA Environmental Planning Program Manager who will assist in determining the environmental process that Chelan will need to go through for environmental approval. In general, federal agencies are interested in using existing environmental documentation so a previous process is not duplicated.

There are three paths to comply with NEPA:

1. Categorical Exclusion – activities that do not have significant impact on the environment
2. Environmental Assessment – concise document to determine if there is a significant impact (requiring Environmental Impact Statement) or Finding of No Significant Impact (FONSI)
3. Environmental Impact Statement (EIS) – FEMA determined there would be a significant environmental impact

When the County consults with FEMA on the environmental process, there are a few options to be aware of:

- To use the Final Environmental Impact Statement (FEIS) determination for this project, FEMA must determine the FEIS:
  - Adequately assesses potential impacts of a proposed action and reasonable alternatives
  - FEMA concurs that the same or substantially the same impacts would result from the proposed action
- If FEMA determines there are new circumstances or additional environmental concerns, the County could prepare a Supplemental EIS that addresses the current project and updated environmental information. This could include incorporating the FEIS by reference and only presenting the new info.
- When consulting with FEMA, another option could be qualifying for a Categorical Exclusion, such as:
  - D5 – Maintenance dredging within floodplains, waterways
  - E2 – new construction in previously disturbed site and consistent with surrounding scale
  - M13 – Install structures and facilities to ensure continuity of operations during emergencies, flooding. Ground disturbing activities less than one acre
  - N4 – Modifications to floodways with ground disturbance less than ½-acre

KPFF staff contacted staff in the FEMA environmental program to develop recommendations for moving forward. Based on these discussions, we expect that Categorical Exclusions would likely cover this work.

## 7.0 Summary of Recommendations

Based on the results of the alternatives analysis, a combination of Roadway Alternative 1 and Bridge Alternative 1 achieves an optimal balance in satisfying the proposed objectives. This combination would construct a 108-foot-span bridge founded on spread footing abutments. South Lakeshore Road would remain in its current alignment with traffic being routed through the bypass road during construction.

A channel with 1H:1V sloped banks and a 30-ft bottom width would be constructed beneath the bridge and lined with concrete to maintain the integrity of the slopes. Channel regrading and construction would occur upstream and downstream to tie in the existing channel, fill the sedimentation basin, reconstruct portions of channel berms, and grade the channel.

The estimated combined cost of the preferred alternative and channel construction is approximately \$3,247,300.

This combination proves to be the most cost effective; offers the least construction risk when installing foundations; minimizes construction impacts; maintains traffic during construction; and best facilitates hydraulic conveyance of debris under the road crossing.

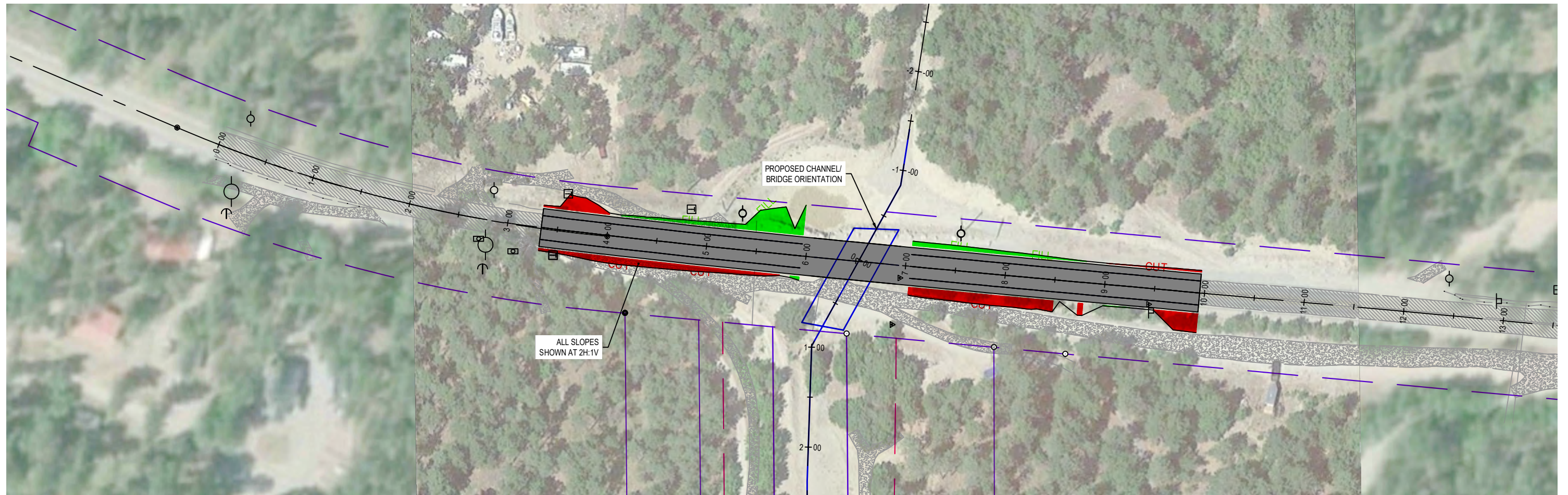


# Appendix A

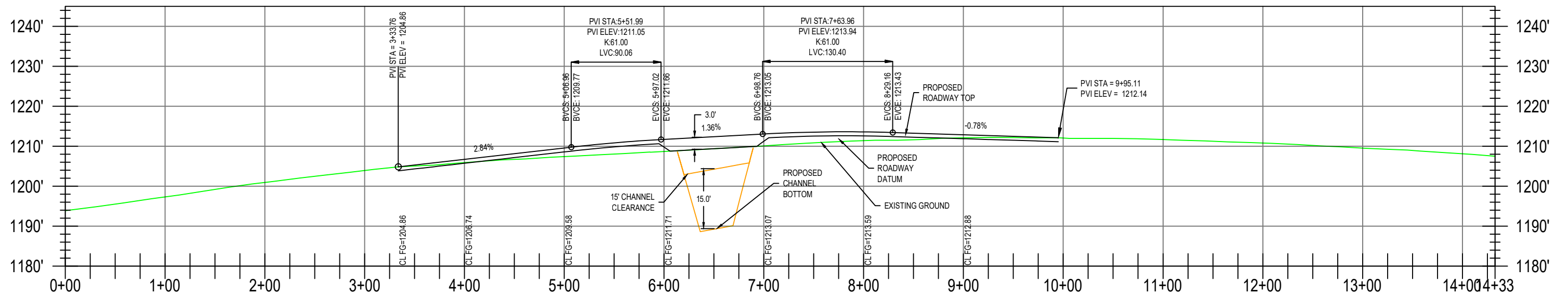
Preliminary Roadway and Channel Alignment Drawings







**PLAN VIEW**  
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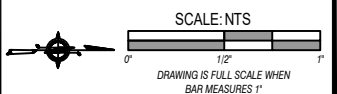


**S LAKE SHORE ROAD CL PROFILE**  
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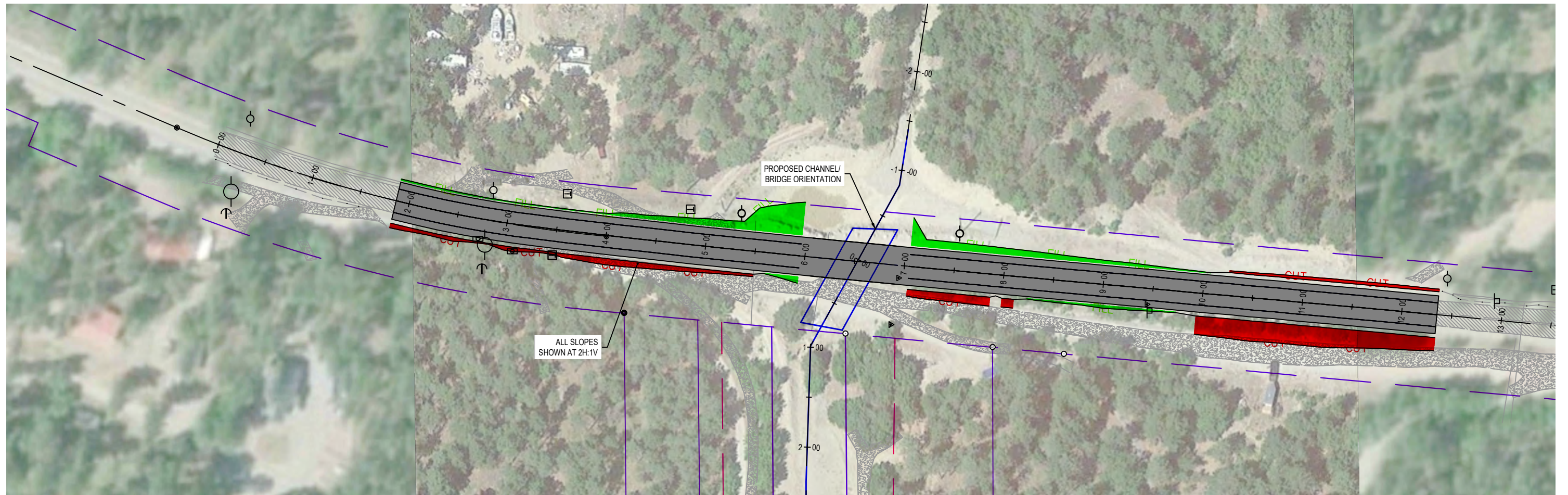
ROADWAY ALTERNATIVE 1

**PROPOSED CONVEYANCE GEOMETRY**  
**EXISTING ALIGNMENT**

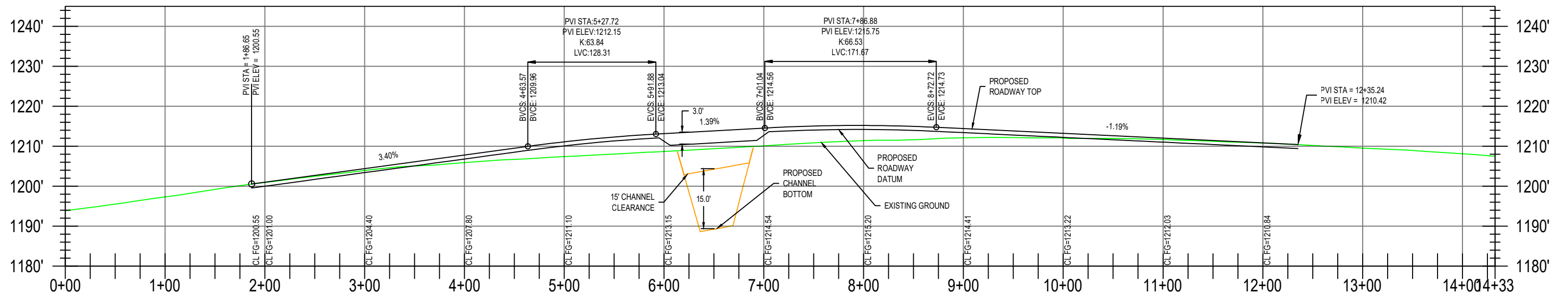
SLIDE RIDGE CULVERT REPLACEMENT



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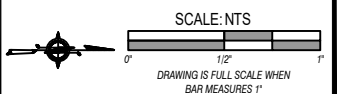


**S LAKE SHORE ROAD CL PROFILE**  
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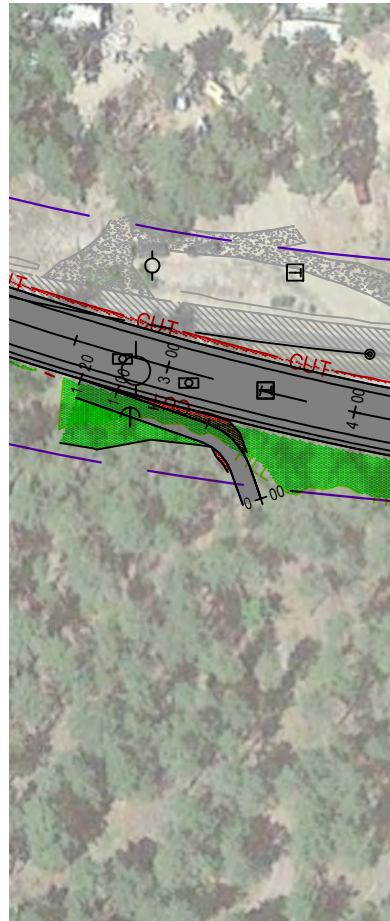
ROADWAY ALTERNATIVE 2

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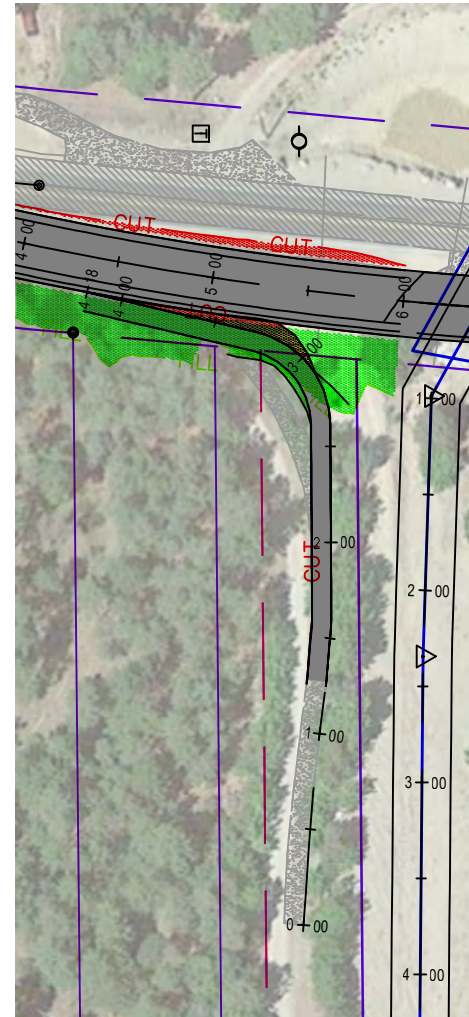
SLIDE RIDGE CULVERT REPLACEMENT



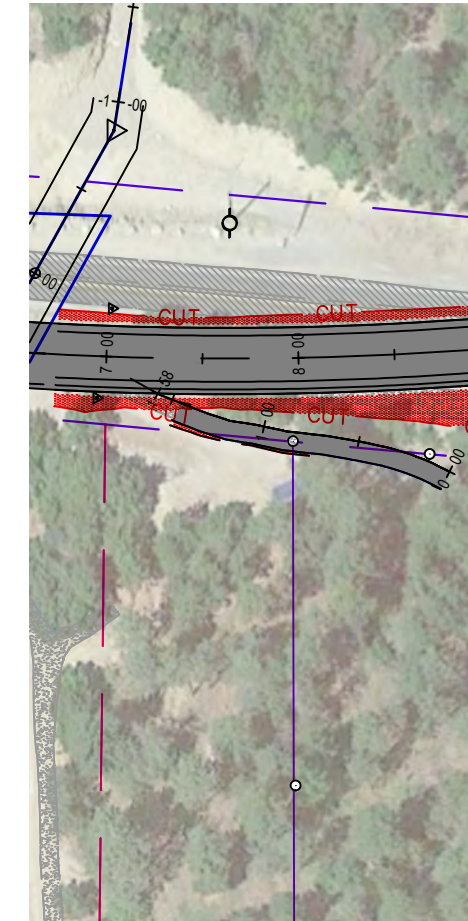
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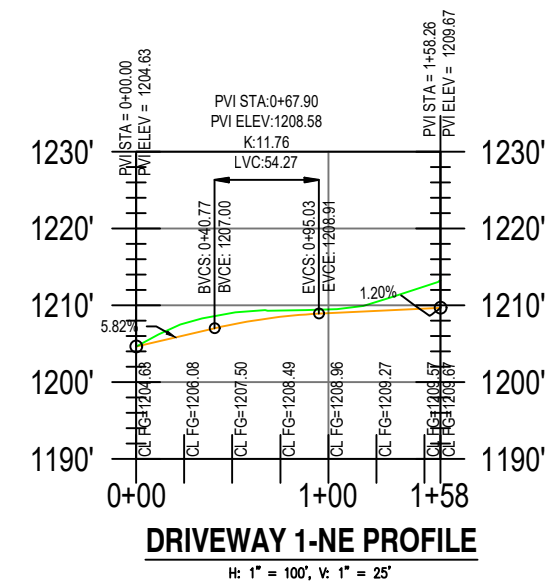
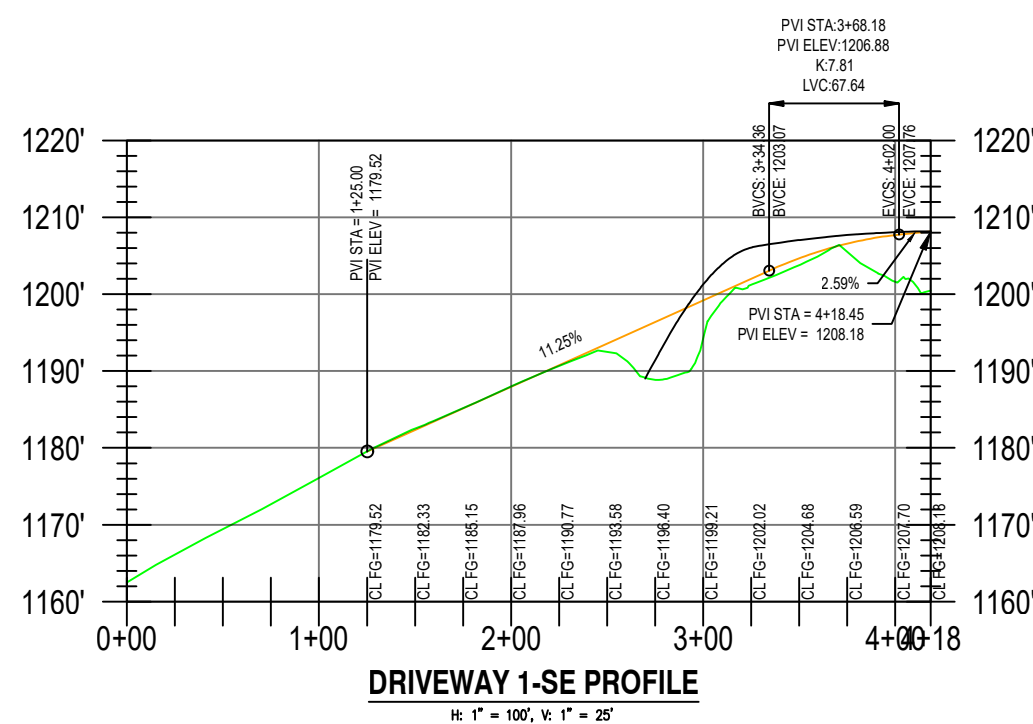
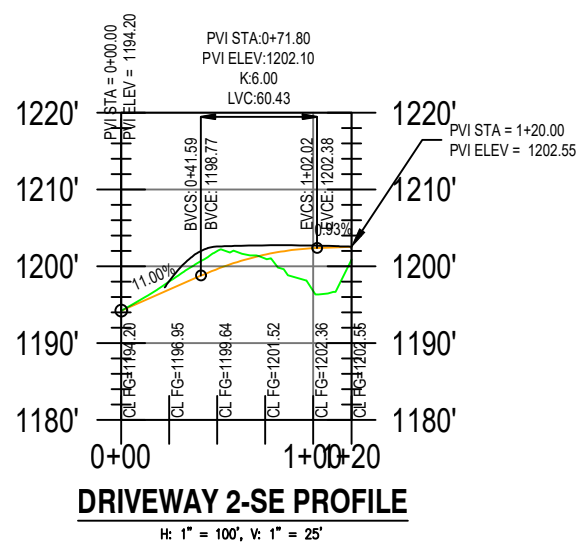
**DRIVEWAY 2-SE PLAN VIEW**  
1" = 100'



**DRIVEWAY 1-SE PLAN VIEW**  
1" = 100'



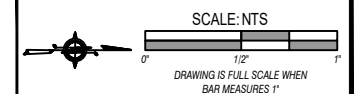
**DRIVEWAY 1-NE PLAN VIEW**  
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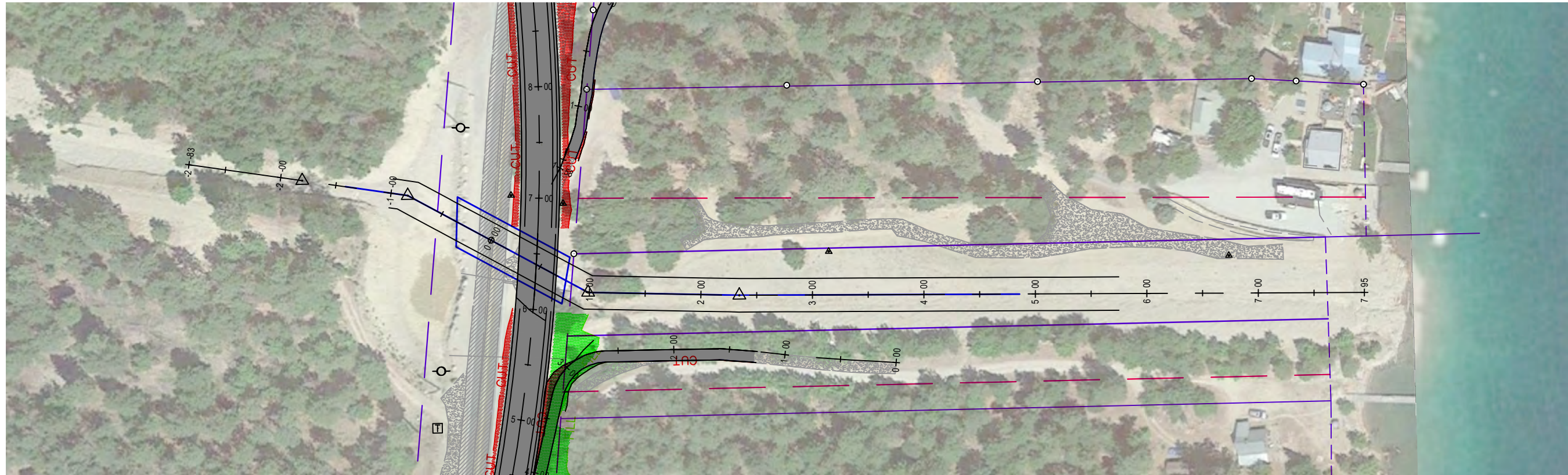
LEGEND	
	PROPOSED DRIVEWAY
	PROPOSED ROADWAY
	EXISTING GROUND

**PROPOSED DRIVEWAY PLAN AND PROFILE**

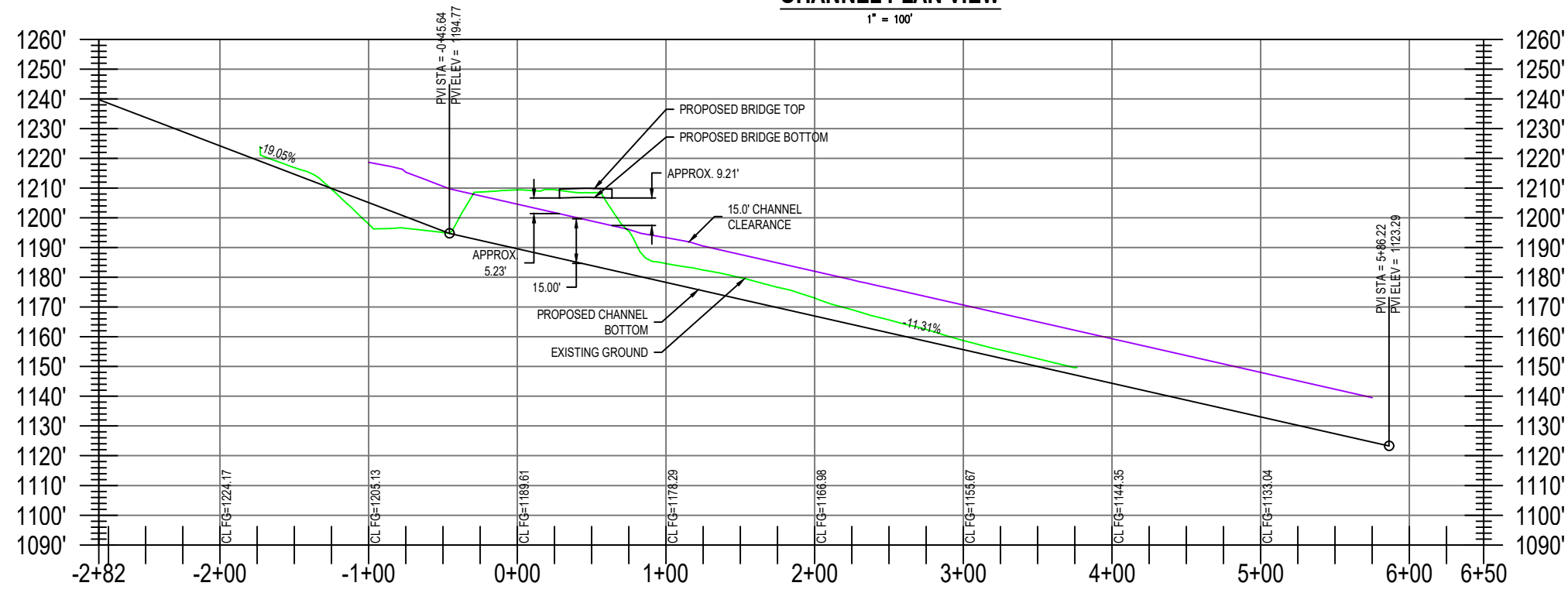
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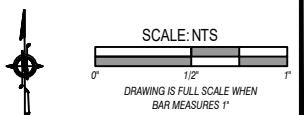
**CHANNEL PLAN VIEW**  
1" = 100'



**CHANNEL PROFILE**  
H: 1" = 100', V: 1" = 50'

**PROPOSED CHANNEL PLAN AND PROFILE**

**SLIDE RIDGE CULVERT REPLACEMENT**

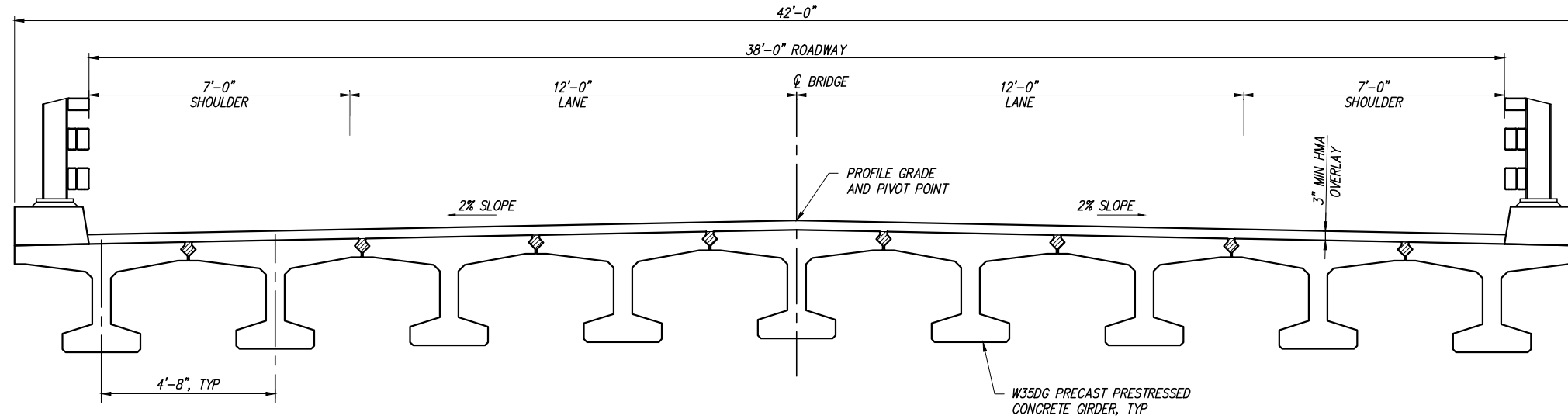


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# Appendix B

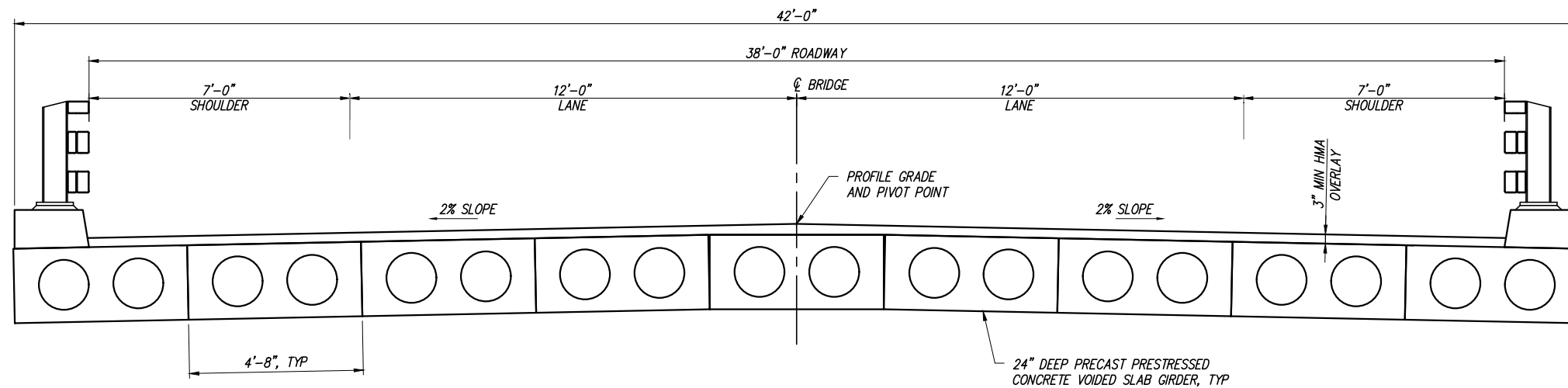
Preliminary Bridge Drawings





**ALTERNATIVE 1 TYPICAL SECTION**

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



**ALTERNATIVE 2 TYPICAL SECTION**

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

CAD USER: saming PLOT DATE: Mar 27, 2019-01: 44pm  
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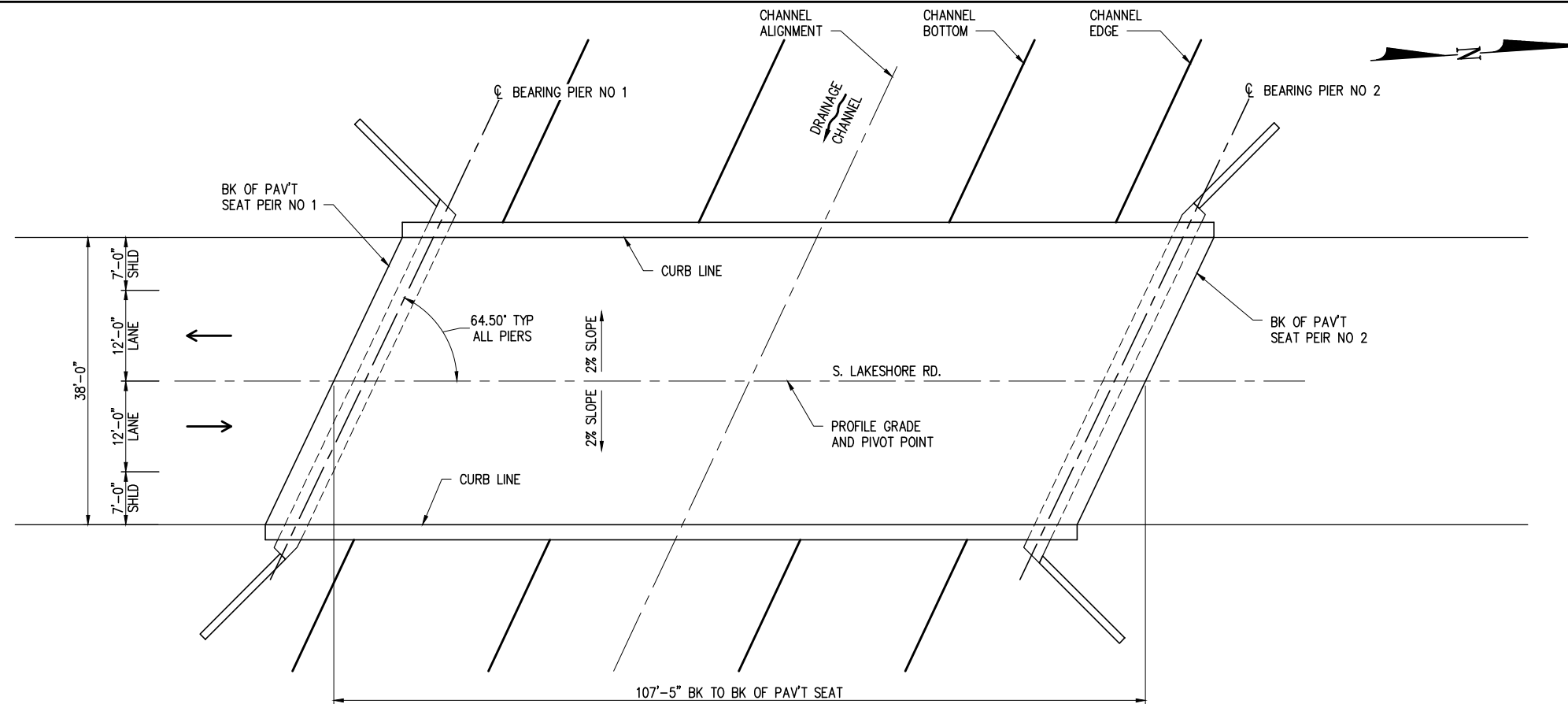
NO.	DATE	BY	REVISION

**SLIDE RIDGE CULVERT REPLACEMENT**  
 CHELAN COUNTY, WA

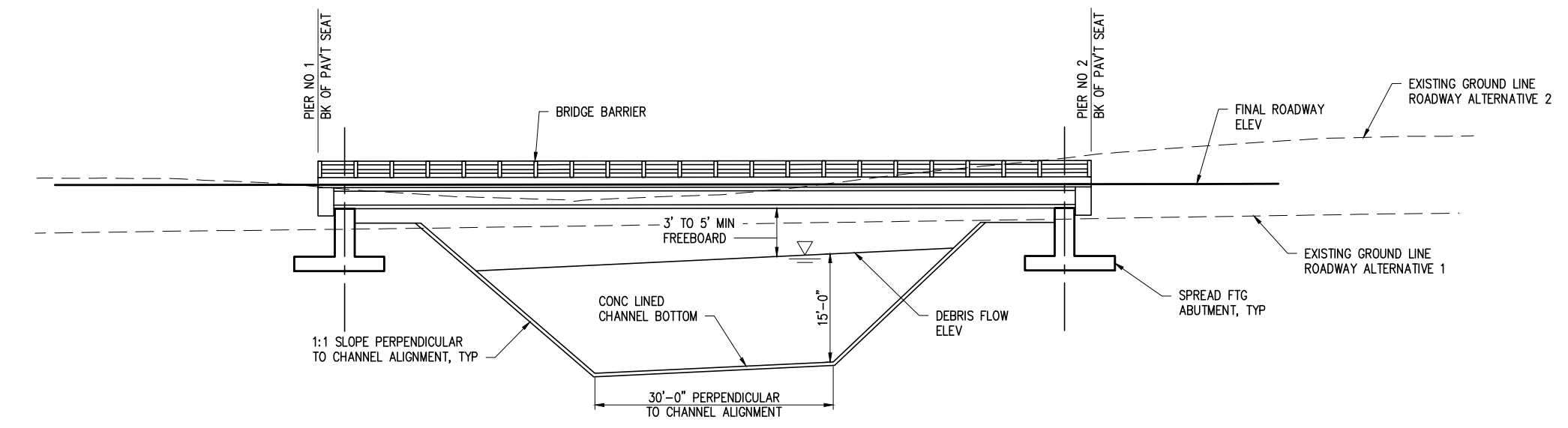
TYPICAL SECTIONS

DRAWN: xxx	PROJECT NO.: XXXXXXXX
DESIGN: xxx	SCALE: AS SHOWN
CHECKED: xxx	DATE: 00-00-2019
DRAWING NO.	
SHEET NO.	SHT OF

SUBMITTAL



**ALTERNATIVE 1 PLAN**  
SCALE: 1" = 10'-0"



**ALTERNATIVE 1 ELEVATION**  
SCALE: 1" = 10'-0"

CAD USER: soming PLOT DATE: Mar 27, 2019-11:57am  
PATH: V:\1700691 (Slide Ridge Replacement)\Design\SRR-Figure-02.dwg



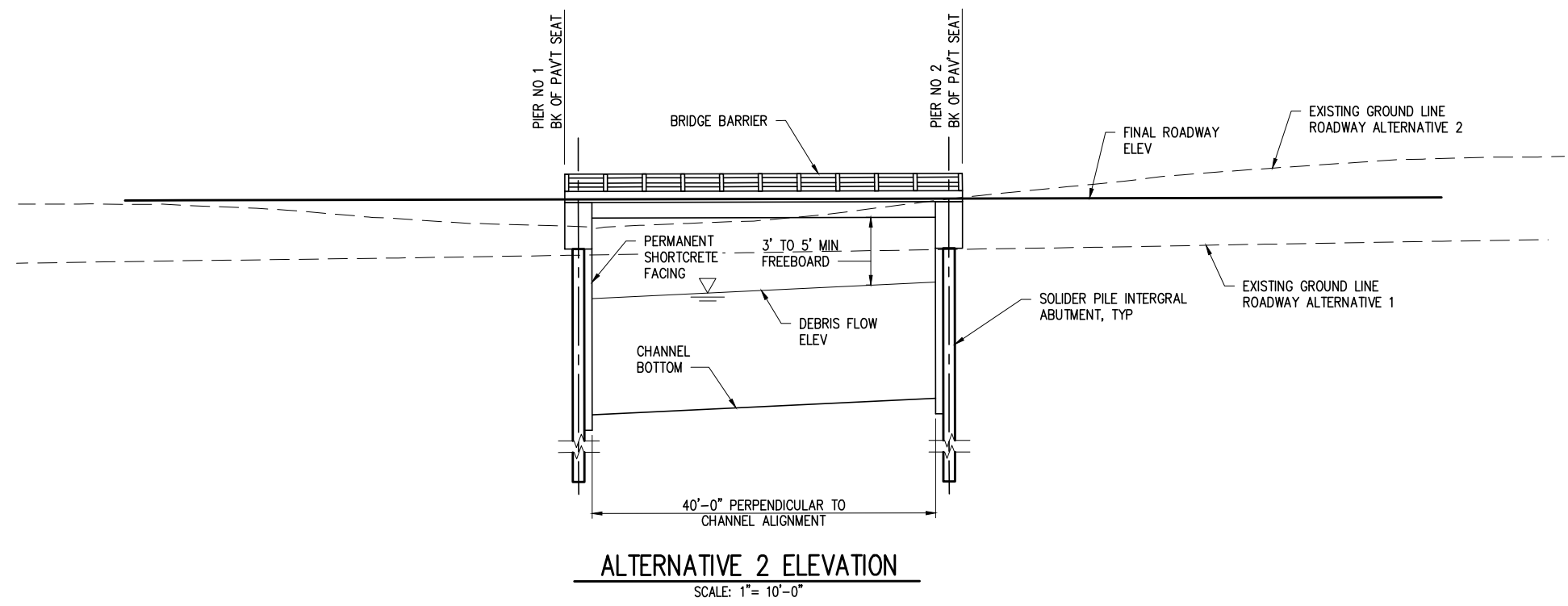
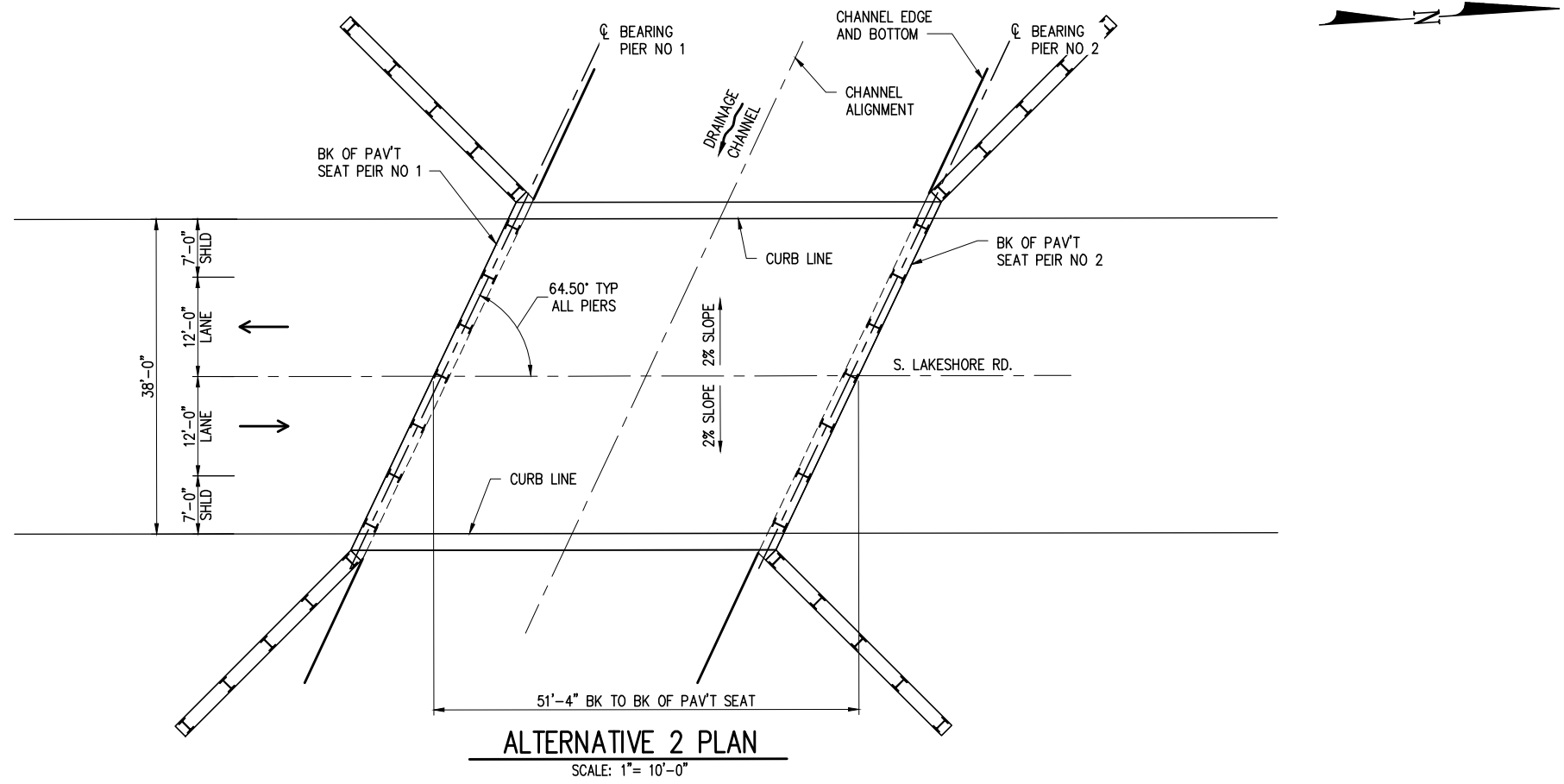
NO.	DATE	BY	REVISION

**SLIDE RIDGE CULVERT REPLACEMENT**  
CHELAN COUNTY, WA  
**ALTERNATIVE 1**  
PLAN AND ELEVATION

DRAWN: xxx	PROJECT NO.: XXXXXXXX
DESIGN: xxx	SCALE: AS SHOWN
CHECKED: xxx	DATE: 00-00-2019
DRAWING NO.	
SHEET NO.	SHT OF

SUBMITTAL





CAD USER: soming PLOT DATE: Mar 27, 2019--12:09pm  
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NO.	DATE	BY	REVISION

SLIDE RIDGE CULVERT REPLACEMENT  
 CHELAN COUNTY, WA  
 ALTERNATIVE 2  
 PLAN AND ELEVATION

DRAWN: xxx	PROJECT NO.: XXXXXXXX
DESIGN: xxx	SCALE: AS SHOWN
CHECKED: xxx	DATE: 00-00-2019
DRAWING NO.	
SHEET NO.	SHT OF

SUBMITTAL

# Appendix C

## Alternatives Cost Estimates





Client: Chelan County  
 Project: Slide Ridge Culvert Replacement  
 KPFF#: 1700691  
 By: RH2 Engineering  
 Date: 2/28/2019

**Engineer's Estimate of Probable Cost**

TS&L Design: Slide Ridge Culvert Replacement - Roadway Alternative 1

STD. ITEM NO.	SPECS REF.	ITEM	UNIT	UNIT PRICE	QTY	COST
Slide Ridge Culvert Replacement - Civil						
0001	1-09	MOBILIZATION	L.S.	\$64,525.00	1	\$64,525.00
		PULVERIZING EXISTING ROADWAY	L.S.	\$5,000.00	1	\$5,000.00
0025	2-01	CLEARING AND GRUBBING	AC.	\$5,000.00	1	\$2,500.00
0310	2-03	ROADWAY EXCAVATION INCL. HAUL	C.Y.	\$20.00	1,200	\$24,000.00
0470	2-03	EMBANKMENT COMPACTION	C.Y.	\$10.00	2,300	\$23,000.00
	2-03	DETOUR GRADING	L.S.	\$40,000.00	1	\$40,000.00
	2-03	DRIVEWAY GRADING	L.S.	\$10,000.00	1	\$10,000.00
		RETAINING WALL	S.F.	\$50.00	1,000	\$50,000.00
5100	4-04	CRUSHED SURFACING BASE COURSE	TON	\$30.00	2,350	\$70,500.00
5120	4-04	CRUSHED SURFACING TOP COURSE	TON	\$40.00	760	\$30,400.00
	5-04	HMA CL. 1/2" PG 64-28	TON	\$100.00	1,590	\$159,000.00
	4-04	DETOUR SURFACING	TON	\$30.00	1,000	\$30,000.00
	5-04	DETOUR PAVING	TON	\$100.00	500	\$50,000.00
6414	8-02	SEEDING, FERTILIZING AND MULCHING	AC.	\$5,000.00	1	\$2,500.00
6490	8-01	EROSION/WATER POLLUTION CONTROL	EST.	\$1.00	2,000	\$2,000.00
6468	8-01	STABILIZED CONSTRUCTION ENTRANCE	S.Y.	\$15.00	130	\$1,950.00
6373	8-01	SILT FENCE	L.F.	\$5.00	500.0	\$2,500.00
6719	8-11	BEAM GUARDRAIL TYPE 31 NON-FLARED TERMINAL	EACH	\$5,000.00	4	\$20,000.00
6757	8-11	BEAM GUARDRAIL TYPE 31	L.F.	\$50.00	800	\$40,000.00
	8-11	BEAM GUARDRAIL TRANSITION SECTION	EACH	\$3,000.00	4	\$12,000.00
6806	8-22	PAINT LINE	L.F.	\$1.00	3,900	\$3,900.00
6890	8-21	PERMANENT SIGNING	L.S.	\$1,000.00	1	\$1,000.00
6971	1-10	PROJECT TEMPORARY TRAFFIC CONTROL	L.S.	\$30,000.00	1	\$30,000.00
	1-05	ROADWAY SURVEYING	L.S.	\$20,000.00	1	\$20,000.00
		AS CONSTRUCTED DRAWINGS	L.S.	\$5,000.00	1	\$5,000.00
7480	2-01	ROADSIDE CLEANUP	DOL	\$1.00	10,000	\$10,000.00

CIVIL CONSTRUCTION COST SUBTOTAL =	\$709,775.00
CONTINGENCY = 30%	\$212,932.50
ESTIMATED TOTAL CONSTRUCTION COST IN 2019 DOLLARS =	<b>\$922,707.50</b>

**NOTES:**

- (1) Unit costs are in 2019 dollars
- (2) Costs do not include Sales Tax, Engineering, Construction Administration or costs associated with permitting



Client: Chelan County  
 Project: Slide Ridge Culvert Replacement  
 KPFF#: 1700691  
 By: RH2 Engineering  
 Date: 2/28/2019

**Engineer's Estimate of Probable Cost**

TS&L Design: Slide Ridge Culvert Replacement - Roadway Alternative 2

STD. ITEM NO.	SPECS REF.	ITEM	UNIT	UNIT PRICE	QTY	COST
Slide Ridge Culvert Replacement - Civil						
0001	1-09	MOBILIZATION	L.S.	\$65,775.00	1	\$65,775.00
		REMOVAL OF EXISTING ROADWAY	L.S.	\$10,000.00	1	\$10,000.00
0025	2-01	CLEARING AND GRUBBING	AC.	\$10,000.00	1	\$5,000.00
0310	2-03	ROADWAY EXCAVATION INCL. HAUL	C.Y.	\$20.00	4,300	\$86,000.00
0470	2-03	EMBANKMENT COMPACTION	C.Y.	\$10.00	2,000	\$20,000.00
	2-03	DRIVEWAY GRADING	C.Y.	\$20.00	500	\$10,000.00
		RETAINING WALL	S.F.	\$50.00	2,000	\$100,000.00
5100	4-04	CRUSHED SURFACING BASE COURSE	TON	\$30.00	2,510	\$75,300.00
5120	4-04	CRUSHED SURFACING TOP COURSE	TON	\$40.00	810	\$32,400.00
	5-04	HMA CL. 1/2" PG 64-28	TON	\$100.00	1,710	\$171,000.00
6414	8-02	SEEDING, FERTILIZING AND MULCHING	AC.	\$5,000.00	1	\$2,500.00
6490	8-01	EROSION/WATER POLLUTION CONTROL	DOL	\$1.00	2,000	\$2,000.00
6468	8-01	STABILIZED CONSTRUCTION ENTRANCE	S.Y.	\$15.00	130	\$1,950.00
6373	8-01	SILT FENCE	L.F.	\$5.00	500.0	\$2,500.00
6719	8-11	BEAM GUARDRAIL TYPE 31 NON-FLARED TERMINAL	EACH	\$5,000.00	4	\$20,000.00
6757	8-11	BEAM GUARDRAIL TYPE 31	L.F.	\$50.00	750	\$37,500.00
	8-11	BEAM GUARDRAIL TRANSITION SECTION	EACH	\$3,000.00	4	\$12,000.00
6806	8-22	PAINT LINE	L.F.	\$1.00	3,600	\$3,600.00
6890	8-21	PERMANENT SIGNING	L.S.	\$1,000.00	1	\$1,000.00
6971	1-10	PROJECT TEMPORARY TRAFFIC CONTROL	L.S.	\$20,000.00	1	\$20,000.00
	1-05	ROADWAY SURVEYING	L.S.	\$30,000.00	1	\$30,000.00
		AS CONSTRUCTED DRAWINGS	L.S.	\$5,000.00	1	\$5,000.00
7480	2-01	ROADSIDE CLEANUP	DOL	\$1.00	10,000	\$10,000.00

CIVIL CONSTRUCTION COST SUBTOTAL =	\$723,525.00
PROPERTY RIGHTS FOR RECONSTRUCTING DRIVEWAYS =	\$30,000.00
CONTINGENCY = 30%	\$226,057.50
ESTIMATED TOTAL CONSTRUCTION AND RIGHT-OF-WAY COST IN 2019 DOLLARS =	<b>\$979,582.50</b>

**NOTES:**

- (1) Unit costs are in 2019 dollars
- (2) Costs do not include Sales Tax, Engineering, Construction Administration or costs associated with permitting



Client: Chelan County  
 Project: Slide Ridge Culvert Replacement  
 KPFF#: 1700691  
 By: D. Kozak  
 Date: 7/8/2019

**Engineer's Estimate of Probable Cost**

TS&L Design: Slide Ridge Culvert Replacement - Bridge Alternative 1

STD. ITEM NO.	SPECS REF.	ITEM	UNIT	UNIT PRICE	QTY	COST
Slide Ridge Culvert Replacement - Structural						
0001		MOBILIZATION	LS	\$108,641.93	1	\$108,641.93
4006	2-09	STRUCTURE EXCAVATION CL A BR NO 90/589	CY	\$33.00	428	\$14,124.00
4013	2-09	SHORING EXTRA EXCAVATION CL A BR NO 90/589	LS	\$19,591.00	1	\$19,591.00
4025	6-02	GRAVEL BACKFILL FOR WALL	CY	\$82.50	189	\$15,592.50
4148	6-02	EPOXY-COATED ST REINF BAR FOR BRIDGE	LB	\$2.50	3,159	\$7,897.50
4149	6-02	ST REINF BAR FOR BRIDGE	LB	\$1.65	7,001	\$11,551.65
4151	6-02	ST REINF FOR ABUTMENT	LB	\$1.65	27,601	\$45,541.65
4202	6-02	CONC CLASS 4000 FOR ABUTMENT	CY	\$935.00	140	\$130,900.00
4269	6-02	PRESTRESSED CONC GIRDER WF36G	LF	\$490.00	967	\$473,830.00
4322	6-02	CONC CLASS 4000 FOR BRIDGE	CY	\$935.00	36	\$33,660.00
4380	6-02	CONC CLASS 4000D	CY	\$1,800.00	16	\$28,800.00
4410	6-06	BRIDGE RAILING TYPE 3-TUBE	LF	\$235.00	215	\$50,525.00
5766		HMA CL 3/8 IN PG 64H-28	TON	\$385.00	72	\$27,720.00
	6-02	CEMENT CONC PAVEMENT - CHANNEL LINING	CY	\$505.00	294	\$148,470.00
	6-02	ELASTOMERIC BEARING - SUPERSTR	EA	\$800.00	18	\$14,400.00
	6-02	ELASTOMERIC STOP PADS	EA	\$106.00	36	\$3,816.00
		GIRDER IMPACT PROTECTION	LS	\$60,000.00	1	\$60,000.00

STRUCTURAL CONSTRUCTION COST SUBTOTAL =	<u>\$1,195,061.23</u>
CONTINGENCY = 30%	<u>\$358,518.37</u>
ESTIMATED TOTAL CONSTRUCTION COST IN 2019 DOLLARS =	<u>\$1,553,579.60</u>

**NOTES:**

- (1) Unit costs are in 2019 dollars
- (2) Costs do not include Sales Tax, Engineering, Construction Administration or costs associated with permitting



Client: Chelan County  
 Project: Slide Ridge Culvert Replacement  
 KPFF#: 1700691  
 By: D. Kozak  
 Date: 5/21/2019

**Engineer's Estimate of Probable Cost**

TS&L Design: Slide Ridge Culvert Replacement - Bridge Alternative 2

STD. ITEM NO.	SPECS REF.	ITEM	UNIT	UNIT PRICE	QTY	COST
Slide Ridge Culvert Replacement - Structural						
0001		MOBILIZATION	LS	\$109,381.65	1	\$109,381.65
4006	2-09	STRUCTURE EXCAVATION CL A BR NO 90/589	CY	\$33.00	301	\$9,933.00
4013	2-09	SHORING EXTRA EXCAVATION CL A BR NO 90/589	LS	\$17,831.00	1	\$17,831.00
4085	6-05	FURNISHING AND DRIVING STEEL TEST PILE	EA	\$29,000.00	2	\$58,000.00
4090	6-05	FURNISHING ST PILING	LF	\$140.00	1,600	\$224,000.00
4095	6-05	DRIVING ST PILE	EA	\$3,100.00	32	\$99,200.00
4148	6-02	EPOXY-COATED ST REINF BAR FOR BRIDGE	LB	\$2.50	1,510	\$3,775.00
4149	6-02	ST REINF BAR FOR BRIDGE	LB	\$1.65	6,385	\$10,535.25
4151	6-02	ST REINF FOR ABUTMENT	LB	\$1.65	21,925	\$36,176.25
4202	6-02	CONC CLASS 4000 FOR ABUTMENT	CY	\$935.00	111	\$103,785.00
4269	6-02	PRESTRESSED CONC GIRDER WF36G	LF	\$445.00	462	\$205,590.00
4299		LAGGING	SF	\$16.00	3,827	\$61,232.00
4322	6-02	CONC CLASS 4000 FOR BRIDGE	CY	\$935.00	33	\$30,855.00
4380	6-02	CONC CLASS 4000D	CY	\$1,800.00	8	\$14,400.00
4410	6-06	BRIDGE RAILING TYPE 3-TUBE	LF	\$235.00	103	\$24,205.00
5766		HMA CL 3/8 IN PG 64H-28	TON	\$385.00	35	\$13,475.00
7011	6-02	GRAVEL BACKFILL FOR FOUNDATION CLASS A	CY	\$93.00	158	\$14,694.00
7561	6-18	SHOTCRETE FACING	SF	\$30.00	3,827	\$114,810.00
8376	6-05	FURNISHING STEEL PILE TIP	EA	\$960.00	12	\$11,520.00
		FORCE ACCOUNT REMOVE PILE OBSTRUCTIONS (25% OF DRIVING COST)	LS	\$24,800.00	1	\$24,800.00
		GIRDER IMPACT PROTECTION	LS	\$15,000.00	1	\$15,000.00

STRUCTURAL CONSTRUCTION COST SUBTOTAL =	\$1,203,198.15
CONTINGENCY = 30%	\$360,959.45
ESTIMATED TOTAL CONSTRUCTION COST IN 2019 DOLLARS =	<b>\$1,564,157.60</b>

**NOTES:**

- (1) Unit costs are in 2019 dollars
- (2) Costs do not include Sales Tax, Engineering, Construction Administration or costs associated with permitting



Client: Chelan County  
 Project: Slide Ridge Culvert Replacement  
 KPFF#: 1700691  
 By: Indicator Engineering  
 Date: 6/4/2019

**Engineer's Estimate of Probable Cost**

TS&L Design: Slide Ridge Culvert Replacement - Channel Excavation

STD. ITEM NO.	SPECS REF.	ITEM	UNIT	UNIT PRICE	QTY	COST
Slide Ridge Culvert Replacement - Channel Excavation						
		MOBILIZATION	LS	\$53,916.90	1	\$53,916.90
	2-03	CHANNEL EXCAVATION INCL HAUL	CY	\$15.00	7,752	\$116,280.00
	2-03	SELECT BORROW (IMPORT)	CY	\$25.00	14,745	\$368,625.00
	2-03	SELECT BORROW (FROM STOCKPILE)	CY	\$7.00	7,752	\$54,264.00

CIVIL CONSTRUCTION COST SUBTOTAL =						\$593,085.90
CONTINGENCY = 30%						\$177,925.77
ESTIMATED TOTAL CONSTRUCTION AND RIGHT-OF-WAY COST IN 2019 DOLLARS =						<b>\$771,011.67</b>

**NOTES:**

- (1) Unit costs are in 2019 dollars
- (2) Costs do not include Sales Tax, Engineering, Construction Administration or costs associated with permitting
- (3) Additional channel cost reduction can be achieved by reducing import/fill such that there is no freeboard downstream of the road for a 100-year event. This would reduce the SELECT BORROW (IMPORT) quantity by 3961 CY resulting in a cost reduction of \$99,025.00 before contingency





# Appendix D

Alternatives Design Workshop Presentation and Notes





**MEETING NOTES**  
**Slide Ridge Culvert Replacement**  
**Preliminary Design Workshop**

Date: October 26, 2018  
Time: 10:00 am – 1:00 pm  
Location: Chelan County Offices, Wenatchee

Attendees:  
Anne Streufert (KPFF)  
Jason Pang (KPFF)  
Pat Flanagan (Indicator)  
Joanna Crowe Curran (Indicator)  
Robert Kimmerling (PanGeo)  
Angi Waligorski (RH2)  
Paula Cox (Chelan County)  
Jason Detamore (Chelan County)

- 
- Pat indicated the existing drainage channel and culvert arrangement is intended to catch everything at the road and it is working very good at this.

Hydraulic Alternatives Developed:

- Conveyance
- Debris retention
- Modified maintenance

Conveyance:

- Concerns – can the debris get all the way to the lake? There is a high level of uncertainty in that the debris can stop anywhere in the channel.
- Ideal condition is to make the channel as steep as possible and maintain the slope
- Conveyance option would likely need to revise the roadway to get enough freeboard and channel depth
- Also would need to raise the levees downstream of the ROW to not overtop and regrade inside the channel
- Debris flow:
  - About 15-20 ft deep channel (15 ft wide at base with 1:1 slopes)
  - 2017 flow was about 15 ft deep
  - Would want a wider channel at the bridge to get a 15 ft deep flow
  - Have different channel configurations
- Jason D. – If we use the conveyance option, will the maintenance at the upstream section decrease?
  - Pat – No, sediment from smaller events will just build up. If multiple small events build up there and a large event happens there is a higher risk of it jumping the channel
- Paula worried about cleaning the channel when the rainfall records show the channel needs to be cleaned. It would be the wettest/most dangerous time of the year to be there.
- County's goal is to have the least maintenance possible. Maintenance of the channel will still be required along the channel.
- Jason P. – If water helps convey flow, is there a way to get more water to the system?
  - Pat/Joanne – Not really. No creeks/channels nearby to contribute water.

- Rather than add water, the wier/herringbone sediment separator system could be used. See p. 17 of the hydraulic report. This pushes the larger rocks to the sides to be able to be cleared.
  - It is a very good solution for this situation, but it needs to be considered experimental
  - Bob asked if the case studies had a steady state flow to flush the system out between large events?
- Paula asked if we could place the herringbone closer to the road to make maintenance easier?
  - Pat – Yes.
- None of these solutions solves channel capacity issues. Therefore the channel would be at high risk to overtop near the apex.
  - Recommend more inspections/maintenance at the apex.
  - The upper basin is where debris may overtop the channel.

#### Debris Retention Systems:

- Existing channel is very efficient. Could put a basin anywhere between the road and apex.
- Benefits of basin at the apex are much less maintenance of the channel because “everything” is caught up high.
  - Reduces amount of debris needing to be removed at the road.
  - However, access to the apex is challenging with limited access road.
- County would prefer maintenance at one location and reducing the interval of the channel maintenance.
- LA County has some design guidelines dating back 30 years for debris flow retention and storm recurrence.
- If a basin is added at the apex, would still want to replace the current culvert with an about 15 ft wide opening.
- The basins are at the same approximate location as the top parking lot
- Top basins also reduce maintenance of the channel
- Bob – For any of the debris containment options, do we start to approach jurisdiction limits where the containment becomes considered a dam? (Could push us into needing design and maintenance at a different level).
- Bob thinks there could be more issues/design considerations related to the containment systems than the bridge option.
- Pat – No options will be without maintenance.
- Paula/Jason to talk with maintenance regarding which debris containment option would be preferred
- Do we have any bathymetry at the lake?
  - Conveyance could require impacts/containment/maintenance in the water.

#### Civil:

- Anticipate moving the road towards the lake improves the vertical clearance beneath the crossing.
  - Shifting the road could impact a lot of properties and driveways very quickly due to the narrow properties lines.
- Moving the road uphill doesn't really help because it has less freeboard or will require raising the road and far more impact to upstream properties.
- Maintaining existing alignment and raising the road to the top of the temporary road berm looks feasible (about 4-5 ft).

- Amount we can raise the road will be controlled by:
  - Access to driveways
  - ROW
  - Sightlines
  - Want to shoot for 3-5 ft freeboard
    - About 20 ft clear from soffit to bottom of channel at upstream edge
- Angi to send AMS requested survey limits.

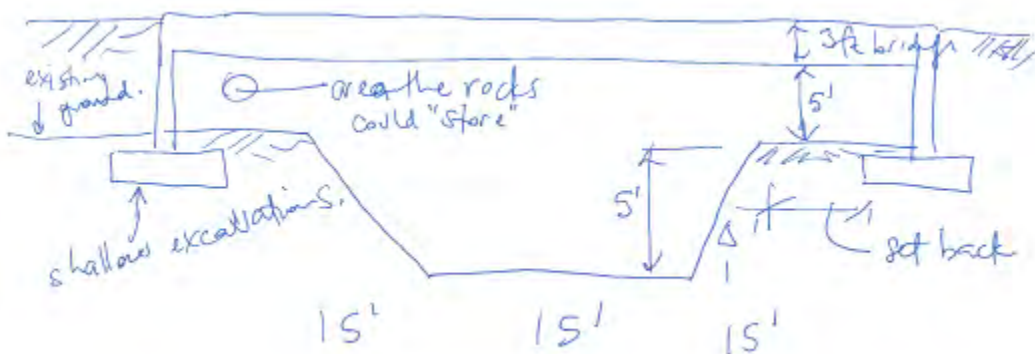
#### Jason P./Bridge:

- Structure options - See slides for more information
- Top-down soldier pile option and longer span spread footing option most preferred and feasible.
- Bob recommended that top-down soldier pile option could eliminate tie backs and utilize superstructure to strut across. Piles can likely be 30" drilled shafts and casing, with 4-5ft spacing.
- 400 ADT - For low ADT could put traffic directly on the deck girders or slabs
  - County would prefer to have at least HMA if not CIP concrete.

#### Geotechnical:

- Sheets would be very difficult to get in the ground.
- Rather than use auger cast piles, would want to use shaft construction (30" diameter would be good, close by).
- Any structure option should address "ejecting boulders".
- Could use 1:1 temporary sloped
- Could use 1:1 permanent slopes at the channel
- Use 1.5:1 permanent road slopes

#### Bob's Idea:



#### Next Steps:

- Paula/Jason D. to talk with maintenance
- Pat to get channel cross-sections to Jason P.
- Jason P. to get a structure depth to Angi
- Angi to see what it will take to provide a road surface
- Bob K. to get a driller lined up
- Anne to send out meeting notes & schedule for next steps.



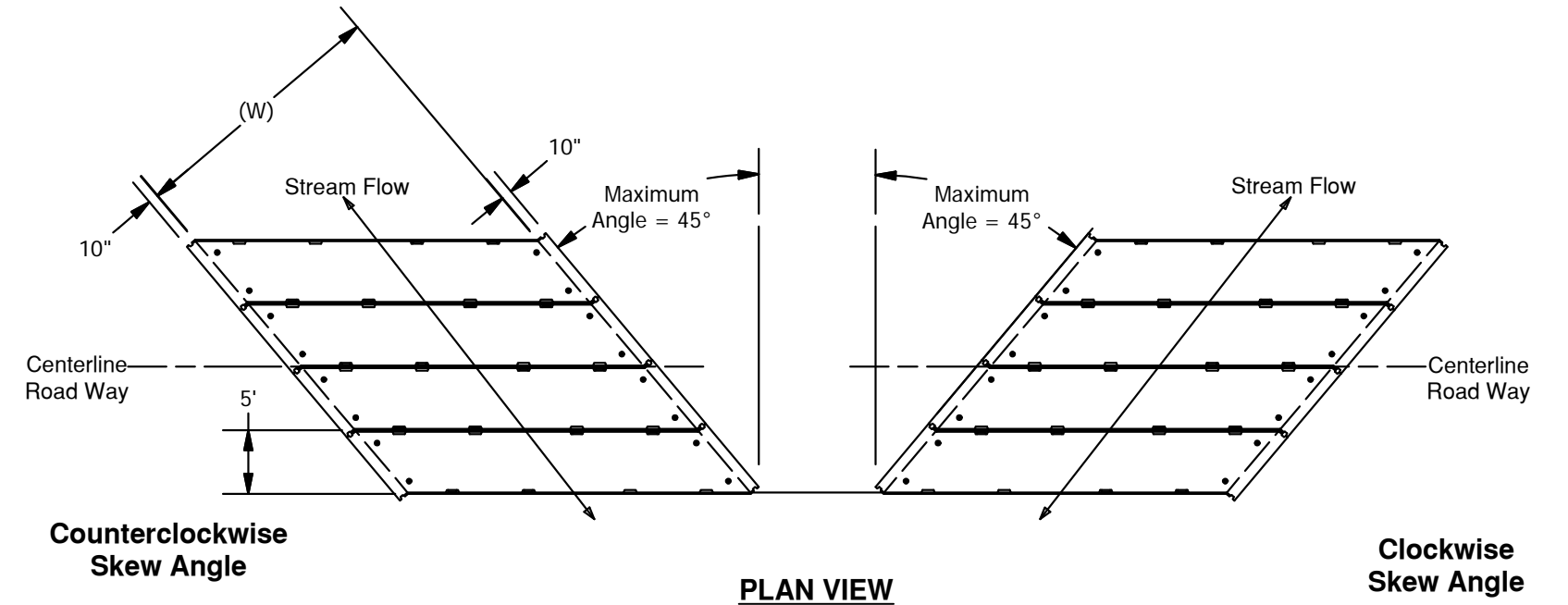
# Slide Ridge Culvert Replacement Project

Design Workshop  
October 2018





# Alternate 1 - Prefabricated 3-Sided Bridges



- (T) VARIABLE DECK THICKNESS
- (H) VARIABLE HEIGHT - (I.D.) - 2' to 10' MAX.
- (W) VARIABLE WIDTH - (I.D.) - 12' to 33' MAX.

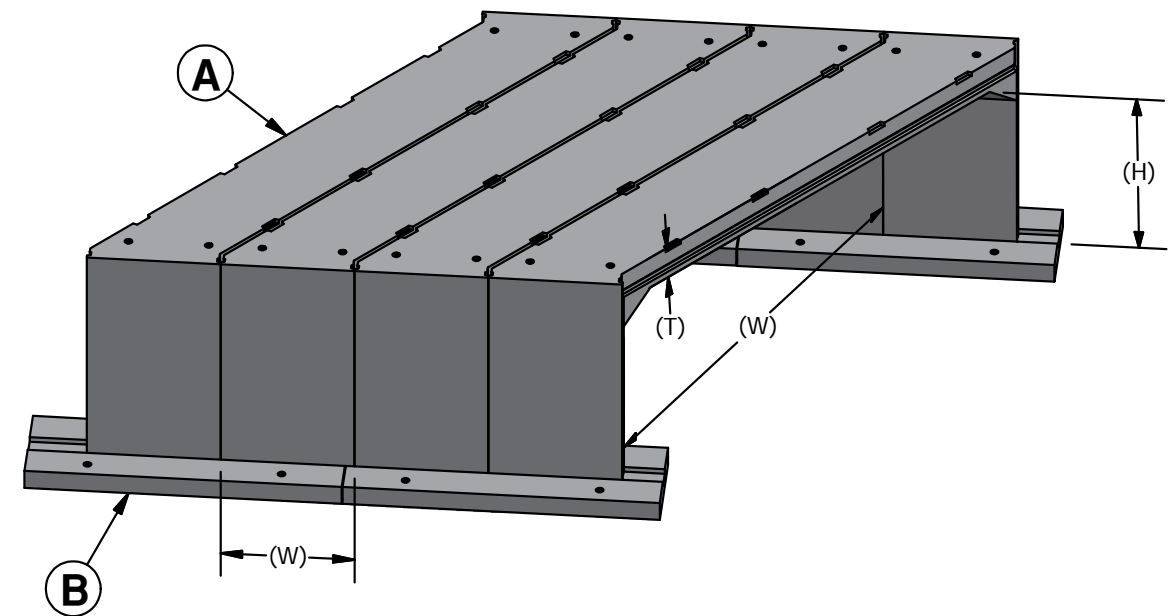
+ Typically very economical

+ Typically straight forward and quick to construct.

- Limitations on sizes and skews

- Requires a large area of open excavation and/or shoring to construct abutment walls

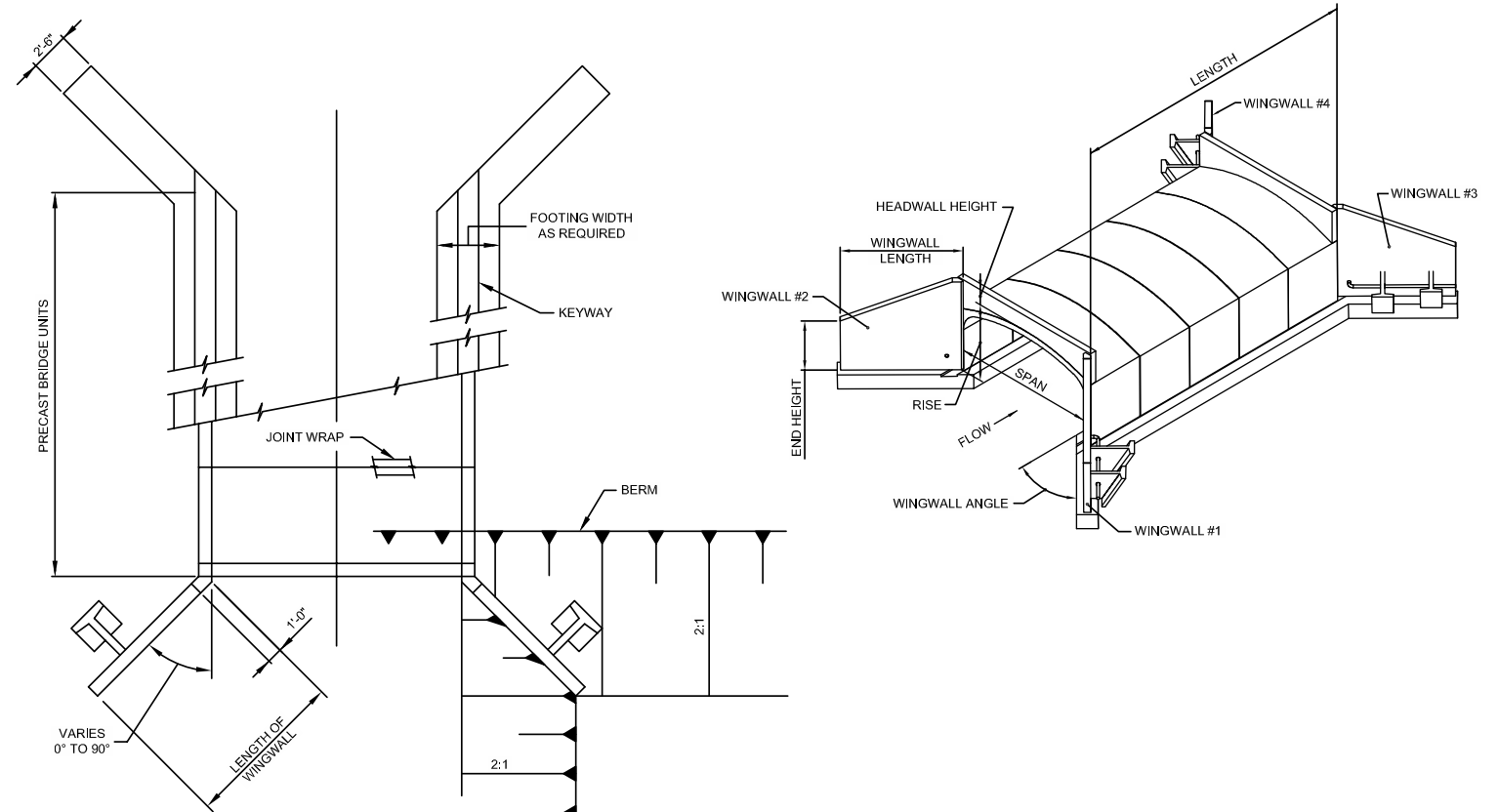
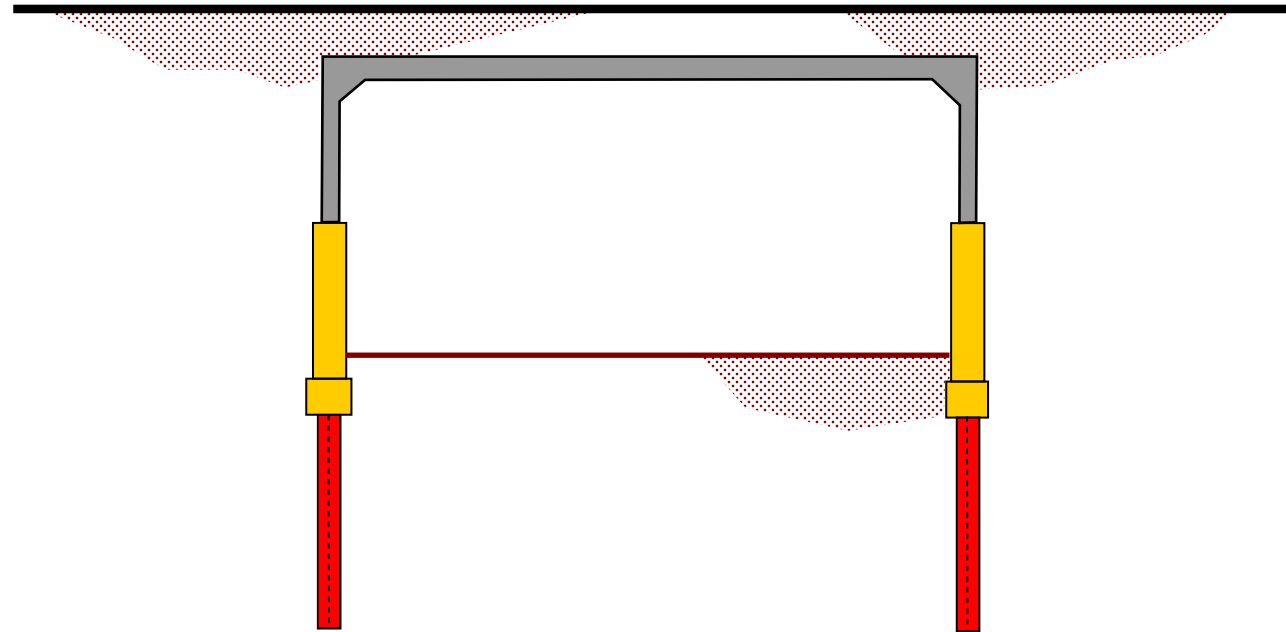
+/- Specialized foundations required for deep scour



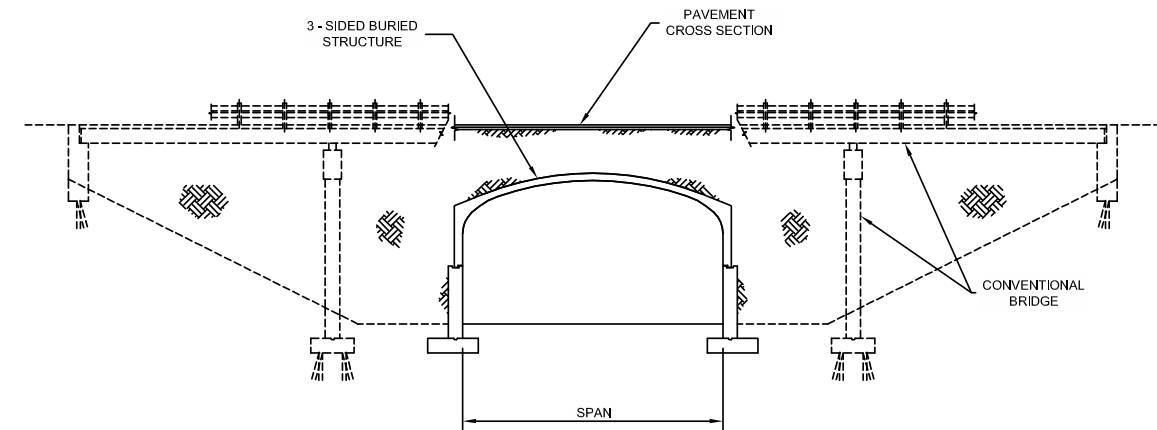
- (A) SKEWED SHORT SPAN BRIDGE
- (B) OPTIONAL FOOTING or FULL BASE SLAB

# Alternate 1 - Prefabricated 3-Sided Bridges (Continued)

## Precast Details



## Buried Structure vs. Bridge-at-Grade



DESIGN SpEc If Ica t lo NS  
 aa SHt o :  
 Ma NUf act URING SpEc If Ica t lo NS  
 ASTM C1504



## Alternate 2 - Cast-in-Place Abutment Wall

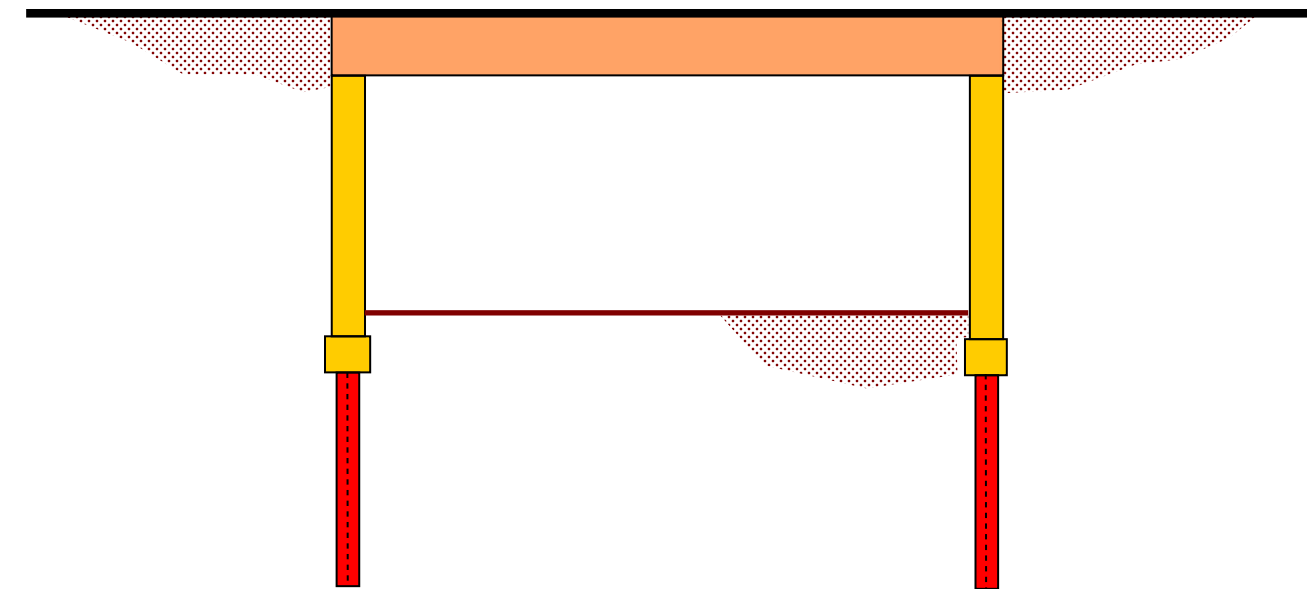
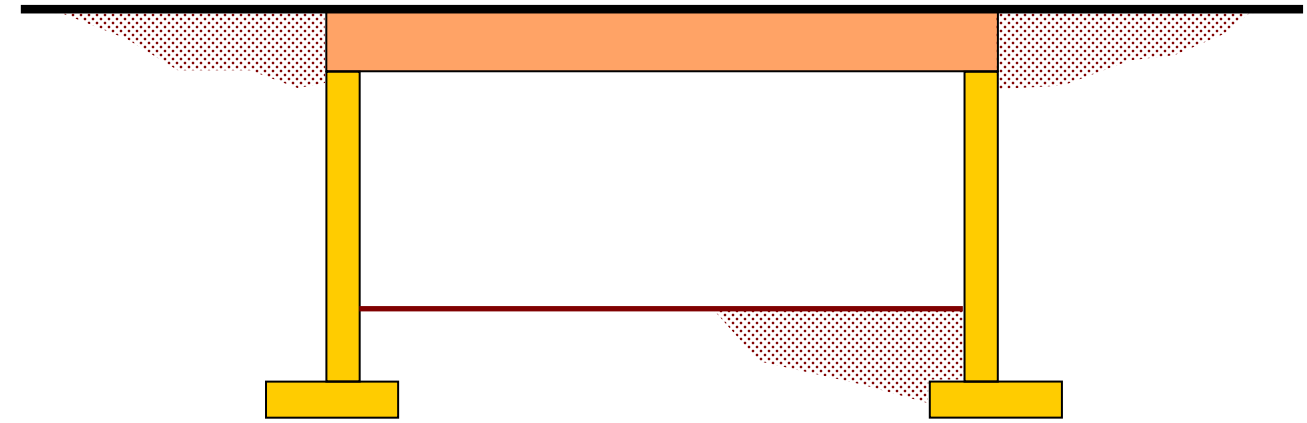
+/- Abutment walls serve as the channel walls to retain embankment.

+. Traditional structure type and construction.

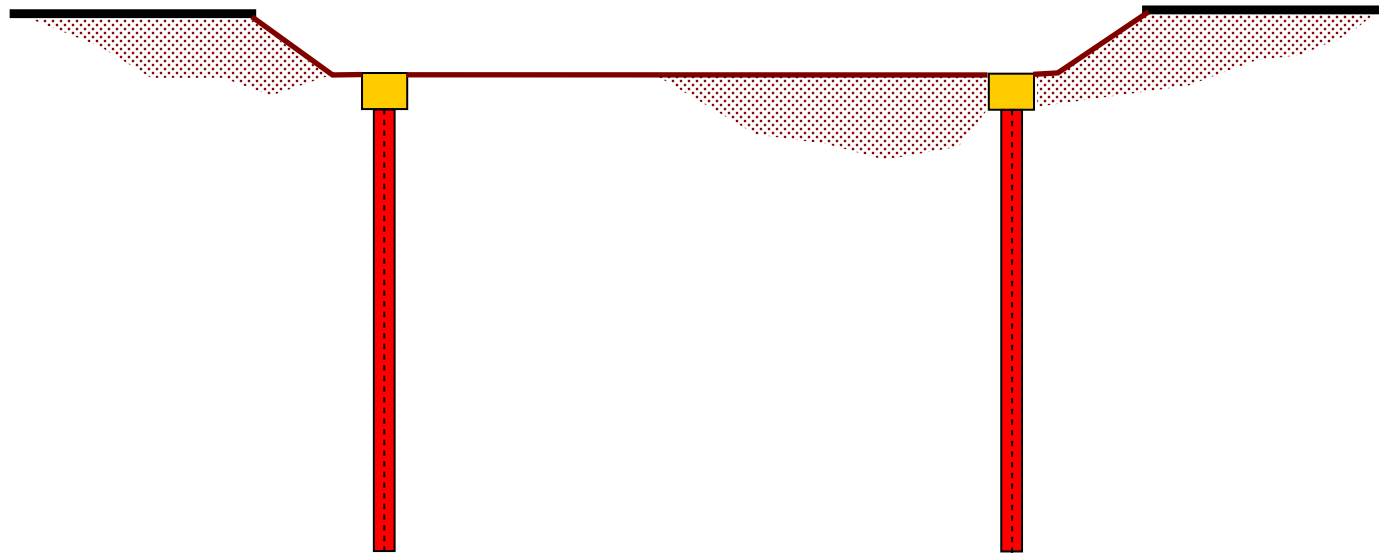
+/- Accommodates both long and short spans

- Requires a large area of open excavation and/or shoring to construct abutment walls

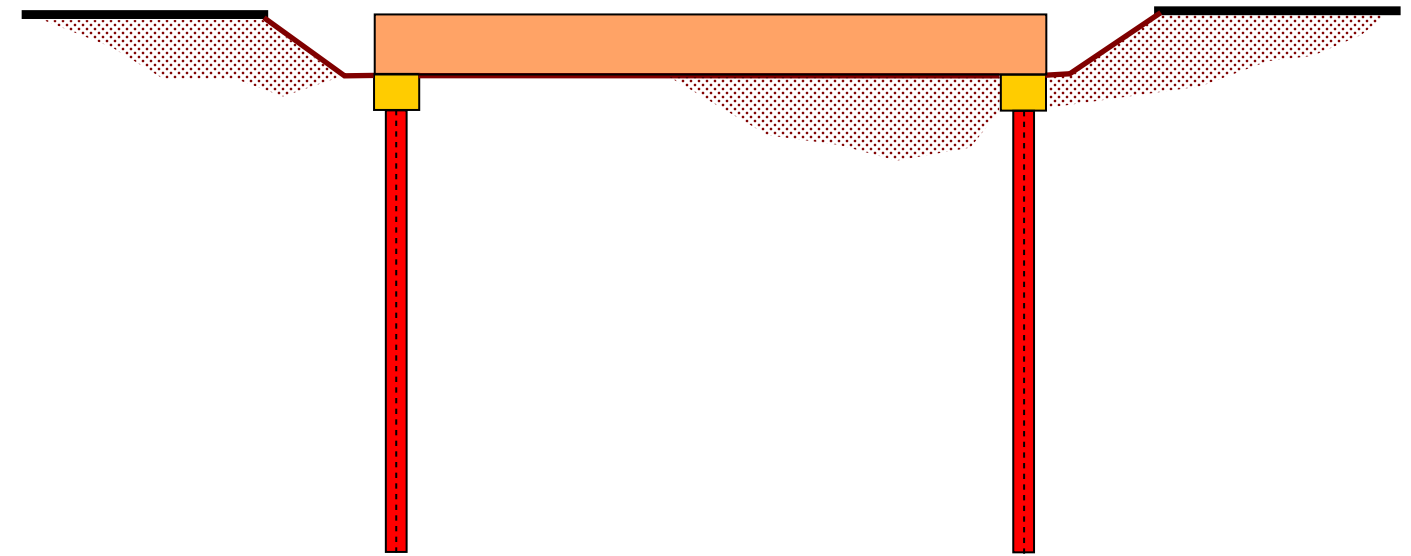
- Foundations can become large as the wall height increase with channel depth.



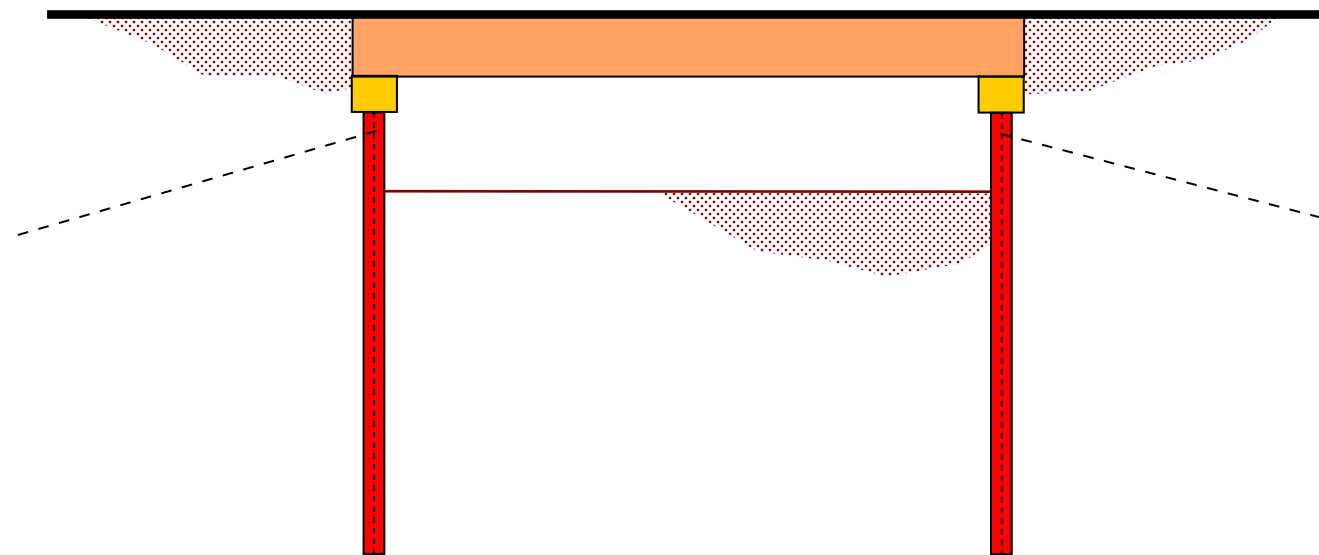
# Alternate 3 - Soldier Pile Abutment - Top Down Construction



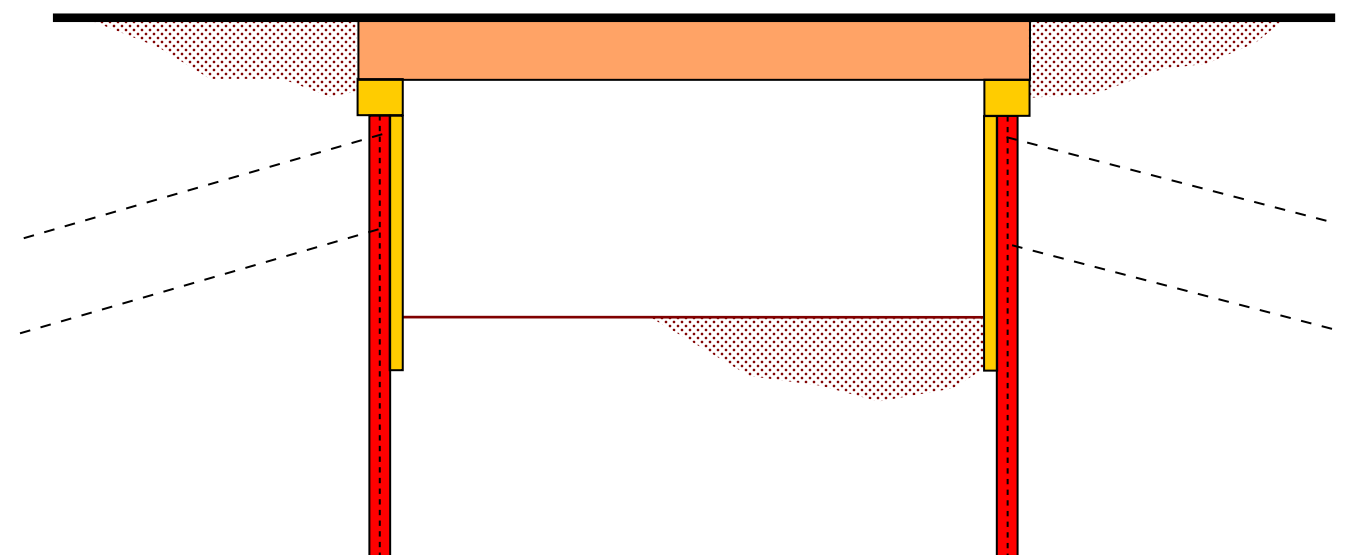
Step 1 - Install Soldier Piles and Caps



Step 2 - Construct Superstructure



Step 3 - Excavate and Install Lagging and Tie Backs



Step 3 - Excavate and Install Shotcrete Facing

# Alternate 3 - Soldier Pile Abutment - Top Down Construction (Continued)



## Alternate 3 - Soldier Pile Abutment - Top Down Construction (Continued)

- + Facilitates phased and accelerated construction to minimize closures of S. Lakeshore.
- + Reduces open excavation, especially as the channel depth increase
- + Deep foundations resistant to scour.
- +/- Soldier pile walls form the final channel walls.
- Construction process is slightly more specialized than earlier alternatives.

## Alternate 4 - SE Wall Perched Abutment

+ SE walls serve as the channel walls to retain embankment. Typically are more cost effective especially with taller walls.

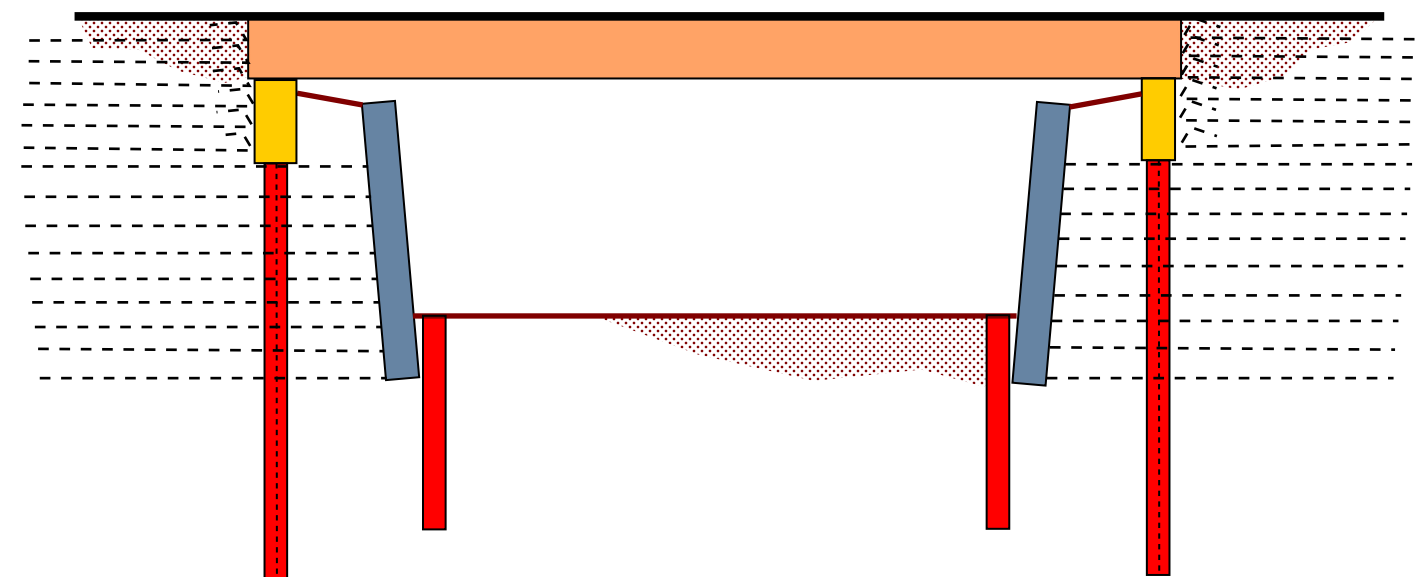
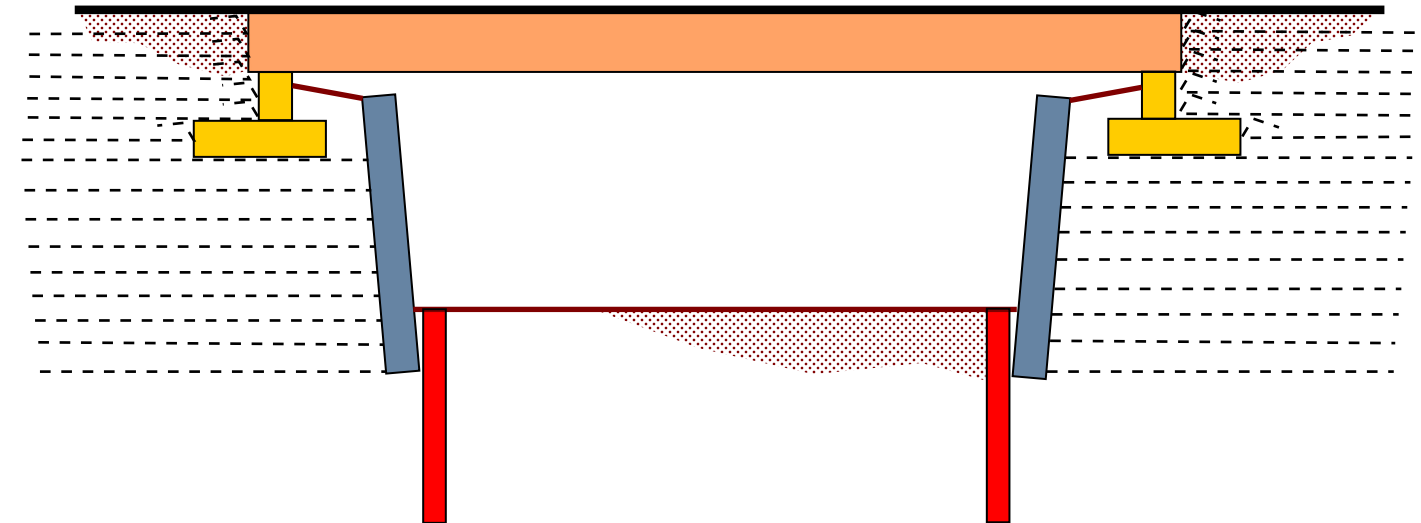
+ Perched spread footings are very economical and easy to construct.

- Requires a large area of open excavation to construct SE walls.

+/- SE walls can be susceptible to scour and would need protection, such as sheet piles

+/- Accommodates both long and short spans

+/- Large block facings or gabions can be resistant to debris collision



# Alternate 4 - SE Wall Perched Abutment (Continued)





## Alternate 5 - Stable Slopes Perched Abutment

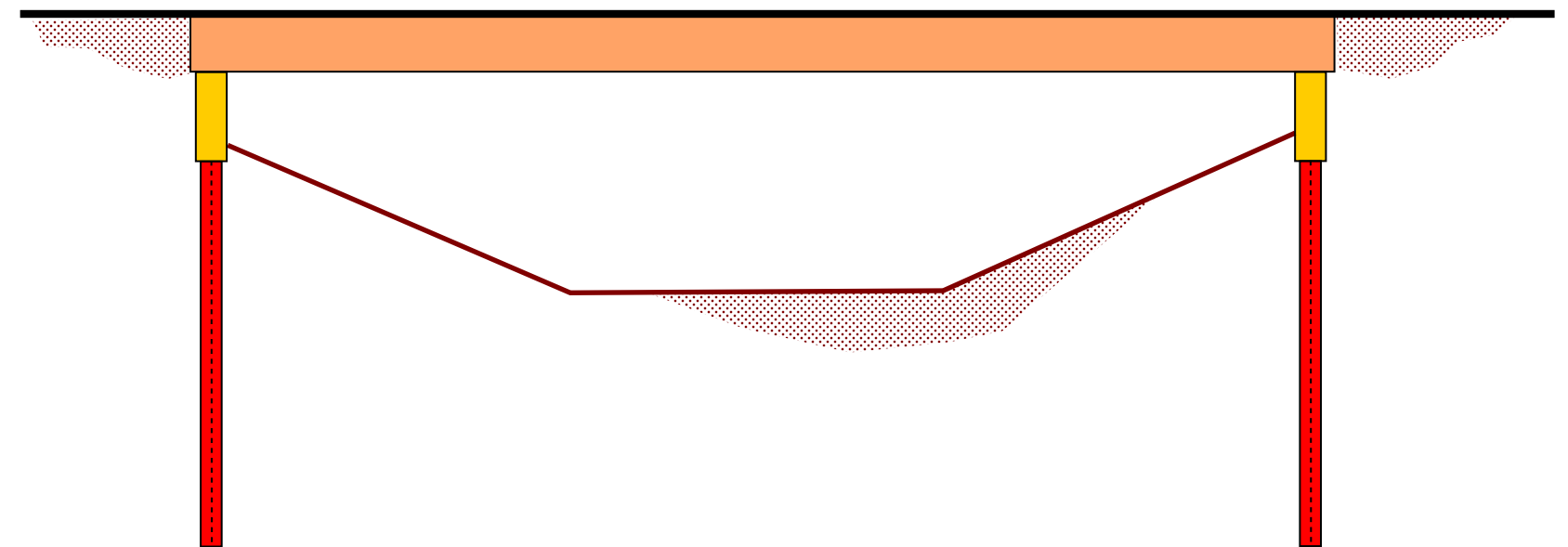
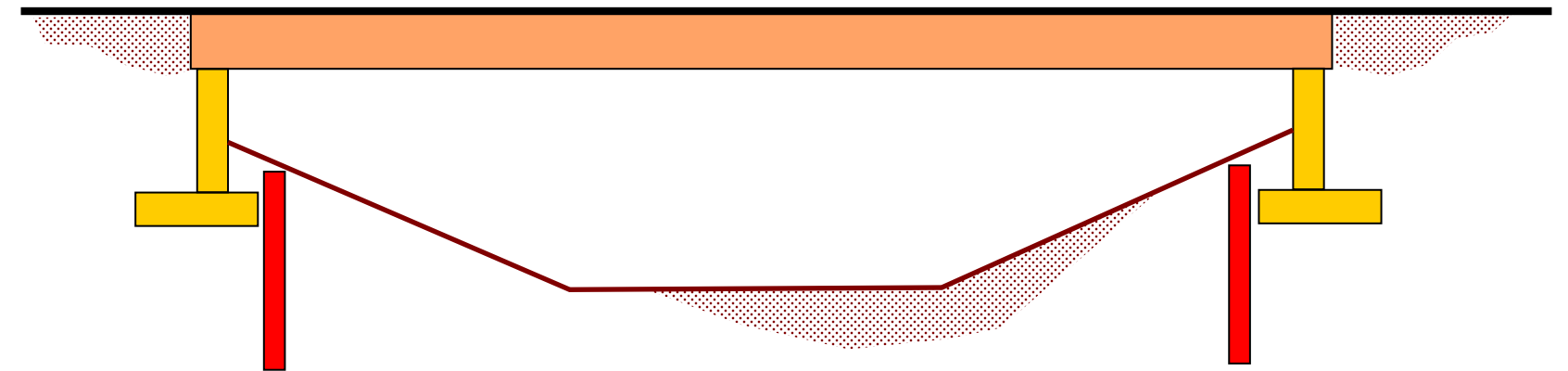
+ Setting back the slope reduces abutment wall height and footing depths.

+ Traditional structure type and construction.

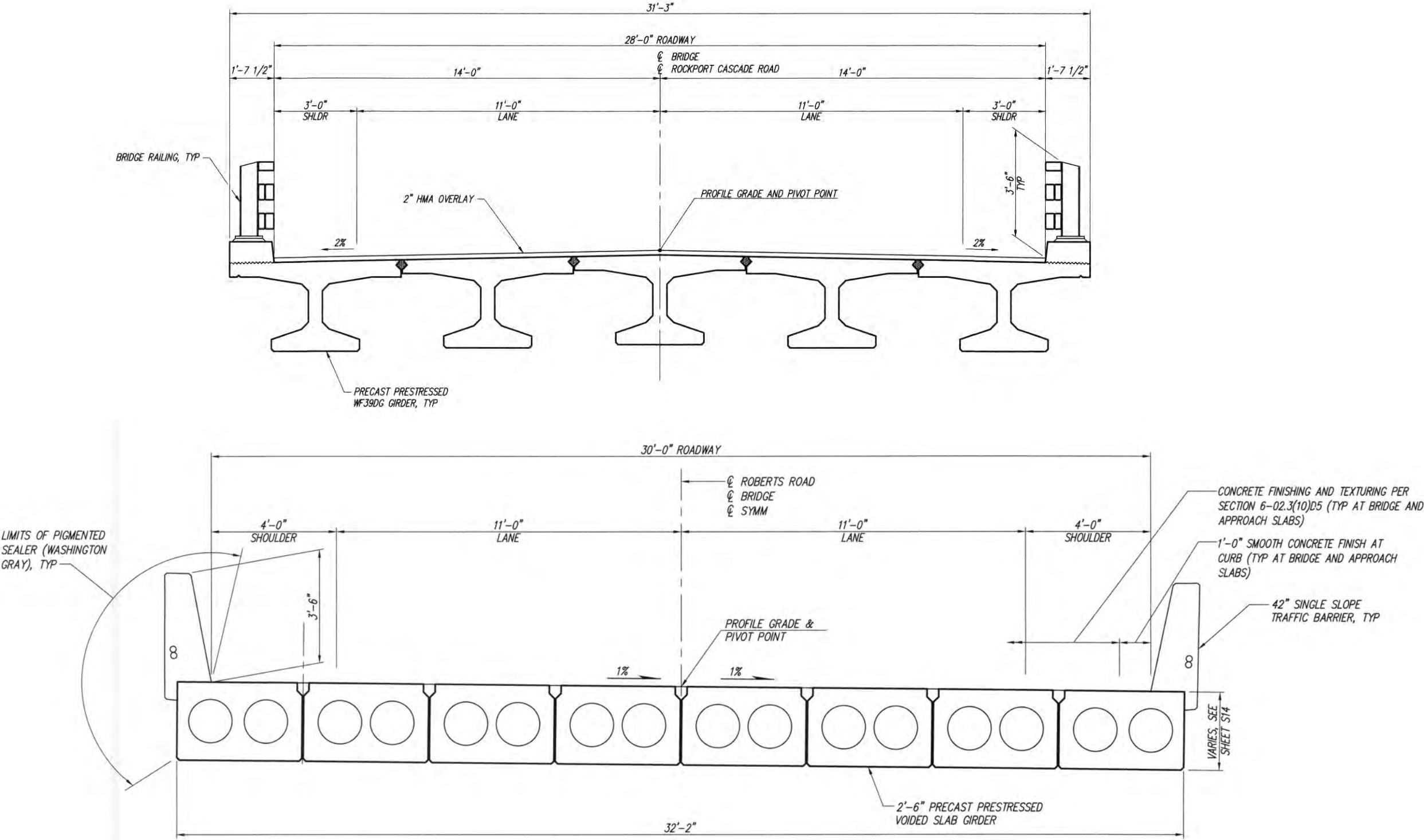
+/- Accommodates wider channels

- Requires a large area excavation

- Exposed slopes may erode. Armoring may be required.



# Superstructure Options



# Appendix E

Preliminary Geotechnical Report



DRAFT GEOTECHNICAL REPORT  
Slide Ridge Culvert Replacement  
South Lakeshore Road, Lake Chelan  
Chelan County, Washington

PROJECT NO. 17-425  
February 2019



Prepared for:



*Geotechnical & Earthquake  
Engineering Consultants*

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Figure 3	Generalized Subsurface Profile
Figure 4	Nominal Bearing Resistance: Spread Footings
Figure 5	Nominal Axial Resistance: 12- and 14-inch H-Piles

## APPENDIX A: FIELD EXPLORATIONS AND BORING LOGS

## APPENDIX B: LABORATORY TEST RESULTS

**DRAFT GEOTECHNICAL REPORT  
SLIDE RIDGE CULVERT REPLACEMENT  
SOUTH LAKESHORE ROAD, LAKE CHELAN  
CHELAN COUNTY, WASHINGTON**

**PROJECT DESCRIPTION**

Chelan County plans to replace the existing culvert that carries South Lakeshore Road over the Slide Ridge drainage channel approximately 3.0 miles north-northwest of the intersection of SR 971, the Navarre Coulee Road (Barrett Grade), and South Lakeshore Road along the south/west side of Lake Chelan, Washington. The Slide Ridge drainage is prone to relatively frequent debris flows that the current culvert is unable to convey which results in over-topping of South Lakeshore Road with debris and road closures. The existing conveyance beneath South Lakeshore Road consists of a main channel culvert of squash-pipe shape and an opening width of approximately 6 feet. A secondary overflow culvert is also present with an inlet elevation approximately 3 feet above the inlet of the main conveyance and consists of an approximately 2 feet diameter pipe.

The proposed replacement structure for the culverts is a single span bridge with sufficient width to accommodate two travel lanes plus shoulders. The length of the replacement structure is yet to be determined by final design, but is likely to be in the range of 45 to 60 feet.

**SITE DESCRIPTION**

The project site is located in the east-central portion of Chelan County, along the west side of Lake Chelan, approximately 12 miles northwest of the town of Chelan, Washington. The site is located in Section 21, Township 28N, Range 21E, W.M. (Figure 1, Vicinity Map). The project area surrounding the existing culvert is dominated by an alluvial fan topographic feature that slopes at approximately twenty percent (20%) toward the east and Lake Chelan. The elevation of South Lakeshore Road is approximately 1210 feet (NAVD88) at the location of the culvert conveyance. Land use in the immediate vicinity of the existing culvert conveyance beneath South Lakeshore Road is undeveloped forested land, beyond which is rural residential including beachfront properties on the downslope side. A berm constructed using debris flow sediments from past events serves as a modified frontage road on the east side of South Lakeshore Road. A sediment collection and containment basin is immediately west of South Lakeshore Road.

**GEOLOGY**

The project site is located on a coalesced alluvial fan formed by three drainages that descend from Slide Ridge west of the project area. These coalesced fans form the shorefront areas known as Hollywood and Shrine Beaches (see Figure 1). The granular constituents of the alluvial fan



are a mix of the numerous parent bedrock lithologies that crop out in the steep slide slopes of Slide Ridge to the west of the fan. Appendix C includes a technical memorandum prepared by PanGEO (dated August 17, 2018) that provides a more detailed description of the existing geologic mapping of the area and these lithologies.

### **FIELD EXPLORATIONS**

The subsurface exploration program consisted of drilling one test boring on either side of the existing culvert at the approximate locations of potential new bridge abutment foundations, as shown on Figure 2, Site and Exploration Plan. Field explorations took place on November 28, 2018. The borings were drilled using truck mounted hollow stem auger drilling equipment provided by Holocene Drilling of Edgewood, Washington. Soil samples were taken in the borings at 5-foot intervals using a standard penetration test (SPT) split spoon sampler, in accordance with ASTM Standard Specification D-1586. The borings were designated PG-1 and PG-2 and were advanced to total depths of approximately 46.3 and 33.0 feet, respectively. Drilling attempts at both locations met with practical drilling refusal in the first attempts to advance the borings to depth. In both cases, the drill rig was moved approximately 5 feet and the drilling was advanced to the previous depth of refusal where split spoon sampling was re-started until drilling was again met with practical refusal at the maximum depths stated above.

An engineering geologist from PanGEO was on site to coordinate drilling activities and log the test borings. The approximate locations of the test borings are indicated on Figure 2, Site and Exploration Plan. Appendix A contains summary logs of the test borings and describes the field exploration methodology in greater detail.

### **LABORATORY TESTING**

Laboratory tests consisting of grain size and moisture content were performed on selected samples collected with the SPT sampling. Laboratory testing was performed by Harold L Benny & Associates, LLC of Poulsbo, WA according to the appropriate ASTM test procedure. The results of laboratory testing are presented in Appendix B.

### **SUBSURFACE CONDITIONS**

An interpretive subsurface profile is included as Figure 3, Generalized Subsurface Profile, with a more detailed description of the soils encountered during the field explorations provided below.

## **SOILS**

Based on the soil samples recovered from the test borings, the subsurface soil profile at the site consists predominantly of silty gravel with sand to poorly graded gravel with silt and sand. Based on the drilling action and surface exposures of the alluvial fan soils surrounding the project area, cobble and boulders are also plentiful in the soil profile. The petrology of the granular clasts are representative of the source bedrock materials in Slide Ridge above the alluvial fan, as discussed above under Geology and in Appendix C. At the time of drilling the soil samples were moist and based on the SPT blowcounts the relative density of the soils was medium dense to very dense. It should be noted that the blowcounts may be over-stated due to the presence of coarse-grained clasts of gravel, cobbles and boulders in the soil profile.

## **GROUNDWATER**

Groundwater was not encountered in either boring drilled as part of the field explorations for this project. The drainage channels in the alluvial fan below Slide Ridge are likely “losing streams” under normal flow conditions and the majority of surface water flow moves rapidly underground once the flow reaches the apex of the alluvial fan. Static groundwater levels are therefore not expected within the construction depths for new structure foundations. Transient groundwater at higher levels within the alluvial fan may be concurrent with rapid surface flow in the channel associated with thunderstorm or rapid snowmelt conditions.

## **CONCLUSIONS AND RECOMMENDATIONS**

### **SITE SEISMICITY**

The project site is located within the uplifted bedrock complex of the Cascade Range. This area is not as seismically active as is the area west of the Cascades but does experience seismic activity. The nearest fault to the site that is thought to be potentially active is the Class B Straight Creek/Evergreen Fault system. This is a north-south trending feature mapped about 50 miles west of the site (Lidke, 2016; Tabor et al., 1993). No faults currently thought to be active intersect with the project site.

At a distance of approximately 65 miles south and southeast of the project site is the Yakima Fold Belt, an area of roughly east-west trending folds along the west margin of the Columbia Plateau. The folds began to develop originally in the late Miocene and deformation may continue into the present day. Seismicity on the Columbia Plateau tends to be generally shallow and associated with thrust faults along the north limbs of the anticlinal structures. Seismicity in the fold belt is generally limited to micro-earthquake swarms that may contain up to 100

individual events in a limited time frame. These occur at shallow depths, normally 3 to 5 kilometers, and rarely exceed 3.5 in magnitude.

The largest historical earthquake observed to date in Washington, with an estimated magnitude of approximately 6.5 to 7.0, occurred on December 14, 1872 in the northern Cascade Mountains. Some recent research and thinking suggests that this event may have taken place on a postulated Chelan Seismic Zone (Crider, et al., 2003), which is located about 10 to 15 miles to the southeast of the project site within a prolific zone of micro-earthquakes referred to as the Entiat cluster.

### ***Seismic Design Parameters***

For seismic design, an acceleration coefficient of 0.139g is recommended per the current acceleration map in AASHTO (2017). The recommended acceleration coefficient is based on expected ground motion at the project site that has a 7 percent probability of exceedance in a 75-year period (approximately 1000-year return period).

Design response spectra presented in AASHTO (2017) are considered appropriate for seismic design of the bridge. A horizontal response spectral acceleration coefficient at a period of 0.2 seconds ( $S_S$ ) is 0.306 and the horizontal response spectral acceleration coefficient at a period of 1.0 seconds ( $S_1$ ) is 0.099.

Based on understanding of the regional geology, the soils at the site are preliminarily considered Site Class D. The associated site factors,  $F_{pga}$ ,  $F_a$  and  $F_v$ , are 1.522, 1.555 and 2.40, respectively, from which values for  $A_S$ ,  $S_{DS}$  and  $S_{D1}$  of 0.212, 0.477 and 0.237, respectively, are obtained. The site is therefore in Seismic Performance Zone 2. The site class may be re-evaluated based on site-specific field explorations and test borings.

### ***Liquefaction Potential***

Simplified screening was used to assess the liquefaction susceptibility of the site soils in accordance with 6.4.2.1 of the Geotechnical Design Manual (WSDOT, 2015). Based on our analyses, liquefaction is not expected to develop at the site under the design earthquake conditions due to the dense nature of the alluvial fan deposits and the absence of saturated ground conditions within 45 feet of the ground surface. Therefore, no special design considerations are recommended due to the potential for liquefaction.

### **LATERAL EARTH PRESSURES ON STRUCTURE SIDEWALLS**

If a joint is provided at the abutment so that the abutment wall is free to deflect slightly, active pressures can be used in design. An equivalent fluid pressure of 35 pounds per cubic foot (pcf) may be used to calculate lateral earth pressures on the abutments. This equivalent fluid pressure

does not include live load surcharge. A lateral earth pressure coefficient,  $K_A$ , of 0.25 may be used to calculate the lateral load due to surcharge.

If abutment walls are fixed against lateral deflection, at-rest pressures will be appropriate for design. An equivalent, at-rest fluid pressure of 55 pcf may be used to calculate at-rest passive earth pressures on the abutments. This equivalent fluid pressure does not include live load surcharge. An at-rest lateral earth pressure coefficient,  $K_o$ , of 0.40 may be used to calculate the lateral load due to surcharge.

The seismic earth pressure is computed according to the Mononobe-Okabe method described in the LRFD Bridge Design Specifications (AASHTO, 2017). The walls are assumed free to move and to develop the active earth pressure conditions during a seismic event. For this project we recommend that the seismic earth pressure increment be taken as  $6H$  psf, where  $H$  is the height of the soil behind the abutment. The seismic earth pressure increment is in addition to the active static earth pressure, and is in a trapezoidal distribution, applied at  $0.6H$  from the bottom of the pressure distribution.

The above lateral earth pressures assume that the new structure is backfilled with good quality, granular material such as Gravel Borrow or Gravel Backfill for Walls per the *Standard Specifications* (WSDOT, 2018). Native alluvial fan materials may also be used for abutment wall backfill provided the abutments are detailed with drainage as recommended in the Bridge Design Manual (WSDOT, 2018).

## **STRUCTURE FOUNDATION RECOMMENDATIONS**

### ***Foundation Alternatives***

From a geotechnical engineering perspective, both deep and shallow foundations are conceptually feasible for support of the replacement structure for the existing culvert. However, the test borings encountered refusal on boulders in both drilling locations. There is therefore a relatively high risk of constructability issues with either drilled shafts or driven piles. Spread footings are considered the best foundation option from both constructability and cost perspectives, provided the scour risk is mitigated.

Deep foundations alternatives consisting of either driven piles or drilled shafts may be considered if a spread option is ruled out. In general, drilled shafts are expected to be a higher cost option relative to driven piles. However, drilled shafts have the advantage of being able to penetrate obstructions such as boulders that could cause difficulties for driven piles. Due to the presence of cobbles and boulders in the alluvial fan soil profile, high displacement piles such as cast-in-

driven shell (WSDOT) piles or pre-cast, pre-stressed concrete piles are not recommended as these types of piles may refuse on shallow obstructions or be difficult to drive within location tolerances. Low-displacement piles such as heavy H-pile sections with driving shoes have been found to obtain penetration with less location control difficulties in these types of soil profiles. Micropiles are also geotechnically feasible, but the slenderness of these types of elements make them more vulnerable to scour damage and less effective in resisting lateral load effects.

Recommendations for both shallow spread footing and driven H-piles are provided below.

### ***Spread Footing Foundations***

Cast-in-place spread footings may be used for support of the new structure abutments provided scour protection is included as part of the channel reconfiguration. Shallow foundations should bear in the medium dense to dense gravel and sand at or below approximately 10 feet below the existing road surface elevation. The bearing elevation and ground conditions should be verified in the field during construction by a representative of PanGEO.

For footings bearing as described above, Figure 4 may be used to proportion footings under LRFD service, strength and extreme load combinations. The service limit state nominal bearing resistance presented in Figure 4 was developed to limit settlement of footings to less than 1 inch. The nominal bearing resistance for strength and extreme limit states plotted in Figure 4 may be used to compare to strength limit state load combinations using a resistance factor of 0.45 and extreme limit state load combinations with a resistance factor of 1.0.

Resistance to sliding will be provided by the friction interface between cast-in-place concrete placed directly on native subgrade soils or on an unreinforced “rat slab”. For these construction conditions a friction coefficient ( $\tan\phi$ ) of 0.70 may be used in conjunction with a resistance factor of 0.8 for determining the strength limit state sliding resistance of the footing.

### ***Driven Pile Foundations***

If a shallow foundation option is not feasible, H-pile sections with tip protection are recommended for driven piling. Heavier sections are preferred to provide better resistance to damage while driving. Large diameter cobbles and boulders may cause piling to refuse or deflect from plumb and plan locations.

Figure 5 presents axial compressive and tensile resistances for 12-inch and 14-inch H-pile sections as a function of elevation. Elevations obtained from Figure 5 should be considered the estimated tip elevation. Provided piles are driven to the required ultimate driving resistance

service limit state resistance is expected to be less than ½-inch. Extreme event axial loads may be evaluated using the nominal (ultimate) resistance on Figure 5 and a resistance factor of 1.0.

Please note that the axial resistances provided on Figure 5 are predominantly for the purpose of estimating pile quantities. Actual pile lengths should be based on verification of axial resistance of driven test piles during construction using the dynamic formula provided in Section 6-05 of the Standard Specifications (WSDOT, 2018). Pile installation should be performed in accordance with Section 6-05 of the Standard Specifications (WSDOT, 2018). The LRFD resistance factors recommended for design are summarized in Table 1.

**Table 1**  
**Recommended Driven Pile Resistance Factors (LRFD)**

Limit State	Resistance Factors, $\phi$ for Driven Piles
Strength	0.55 <sup>1</sup>
Extreme	1.0

Notes: <sup>1</sup> – Pile resistance determination using dynamic formula in Article 6-05.3(12) of the Standard Specifications (WSDOT, 2018)

***Lateral Pile Resistance & Group Reduction Factors***

For pile groups consisting of a single row of piles, the group reduction factors for lateral analysis presented in AASHTO (2017) should be used. Group effects for axial loads will not be significant so long as piles are spaced at least 2.5D.

Recommended parameters for analysis of lateral pile resistance using the program LPILE™ or COM624 are presented in Table 2. Note that the soil layer is referenced to an assumed bottom of pile cap elevation that is approximately 10 feet below the existing road grade. The layer depths should be adjusted accordingly for the actual bottom of pile cap elevation. The soil conditions and layering are sufficiently similar between both test borings drilled as part of this study to generalize the p-y response for both abutments by using the parameters provided in Table 2.

**Table 2**  
**Recommended p-y Curve Parameters**

Reference Elevation: +1200 feet <sup>1</sup>		STATIC ANALYSIS								
Soil Layer	Bottom of Layer Elevation	Soil Type	Soil Type (KSOIL)	Effective Unit Weight of Soil		Cohesion		Axial Strain $\epsilon_{50}$	Friction Angle $\phi$	Modulus of Subgrade Reaction
	(ft)			(pci)	(pcf)	(psi)	(psf)		(deg)	(pci)
1	1160	Sand	4	0.072	125	--	--	--	36	160

Notes: <sup>1</sup> – Adjust as necessary for bottom of pile cap elevation or ground line, as appropriate.

### **CONSTRUCTION CONSIDERATIONS**

The following items should be considered during the foundation design and development of the contract specifications.

1. Temporary shoring and/or slopes will be required during construction of the new abutment foundations. The design of temporary shoring or slopes should be the responsibility of the Contractor.
2. Spread footings should bear on native, undisturbed granular soils. A representative of PanGEO should observe and verify the spread footing subgrade prior to placement of reinforcing steel and concrete. A rat slab may facilitate forming and placement of the footing. Scour protection for spread footings should be included as part of the channel reconfiguration.
3. Piles should be driven in accordance with Section 6-05 of the Standard Specifications (WSDOT, 2018). Nominal resistance of driven piles should be verified during construction using the dynamic formula in Section 6-05 in order to be consistent with the Strength Limit state resistance factor of 0.55 recommended for design. At least one test pile should be driven in accordance with Section 6-05.3(10) of the Standard Specifications (WSDOT, 2018).
4. H-pile sections should be provided with pile tip protection. Pile tips should be selected from the current items approved in the WSDOT Qualified Product List.

### **ADDITIONAL SERVICES**

Construction support services, including review of pile driving submittals and field observation of pile installation, or observation of spread footing subgrades, are beyond the scope of geotechnical design services under which this report was prepared. A supplemental scope and budget would be required for PanGEO to provide construction support services and is recommended in order to confirm that construction is consistent with the design and construction recommendations provided herein.

### **LIMITATIONS AND UNIFORMITY OF CONDITIONS**

PanGEO, Inc. (PanGEO) prepared this report for KPFF and Chelan County. The recommendations contained in this report are based on a site reconnaissance, a subsurface

exploration program, review of pertinent subsurface information, and our understanding of the project.

Variations in soil conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, PanGEO should be immediately notified to review the applicability of the recommendations presented herein. Additionally, PanGEO should also be notified to review the applicability of these recommendations if there are any changes in the project scope.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 36 months from its issuance. PanGEO should be notified if the project is delayed by more than 36 months from the date of this report so that the applicability of the conclusions and recommendations presented herein may be evaluated considering the time lapse.

Within the limitations of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time this report and/or its contents was prepared. No warranty, express or implied, is made. The scope of PanGEO's work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water or groundwater at this site. PanGEO does not practice or consult in the field of safety engineering. PanGEO does not direct the contractor's operations, and cannot be held responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes shall be at the contractor's sole option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.



February 27, 2019  
Project No. 17-425

**CLOSURE**

PanGEO is pleased to support the KPFF design team and Chelan County with geotechnical engineering recommendations. If you have any questions regarding this report, please call (206) 262-0370.

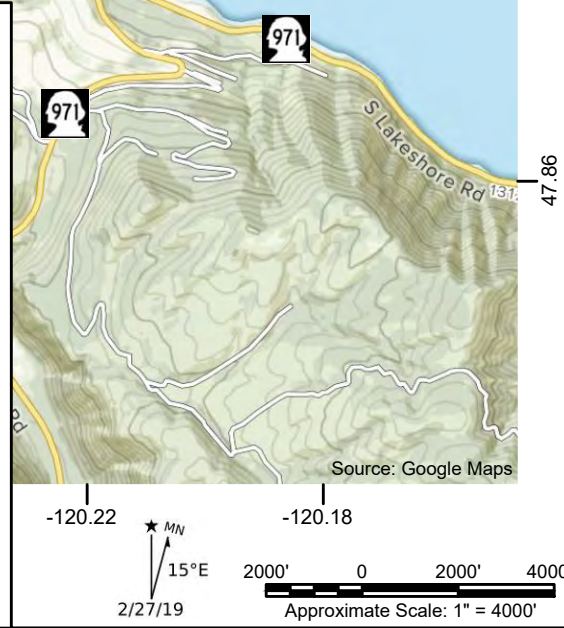
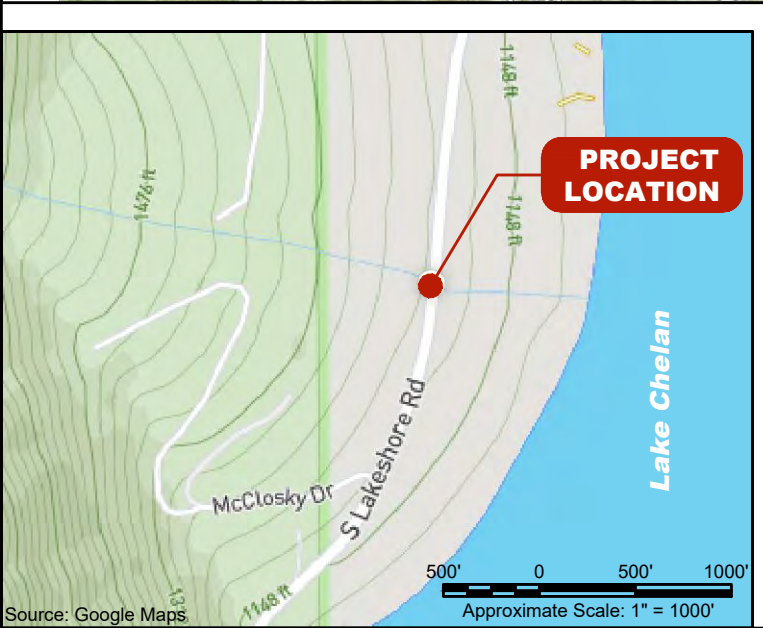
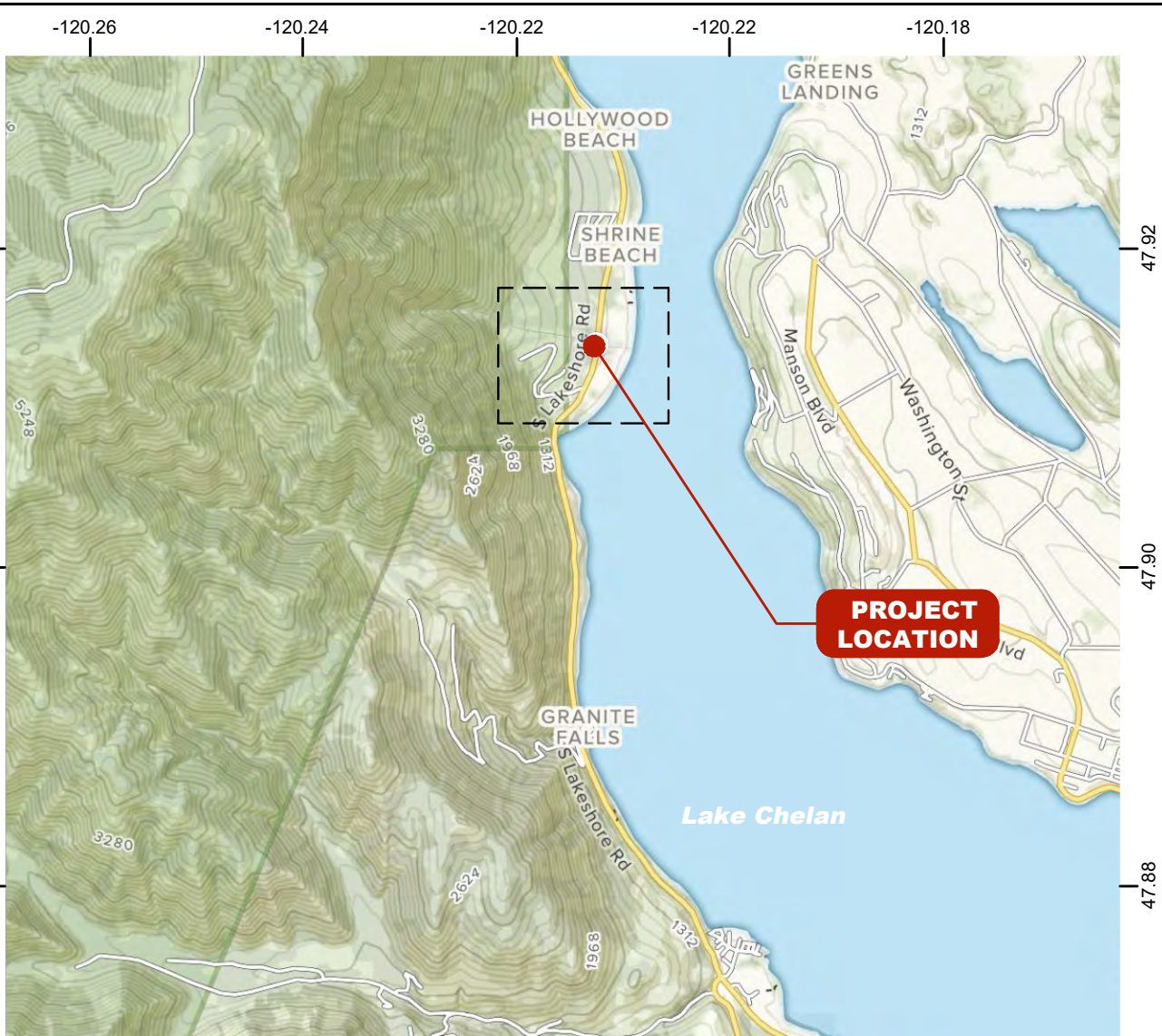
Robert E. Kimmerling, P.E.  
Principal Geotechnical Engineer

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- Washington State Department of Natural Resources Interactive Geologic Map, 2013  
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- WSDOT, 2018. Bridge Design Manual LRFD (BDM), M 23-50, Washington State Department of Transportation
- WSDOT, 2015. Geotechnical Design Manual (GDM), M 46-03, Washington State Department of Transportation
- WSDOT, 2018. Standard Specifications for Road, Bridge and Municipal Construction, M 41-10, Washington State Department of Transportation

## **FIGURES**

CHECKED BY: REK    DATE: 2/27/2019    Z:\Projects\2017 Projects\17-425 Side Ridge Culvert Replacement\Report\Draft Report\Figures\CADD\Vicinity Map.dwg



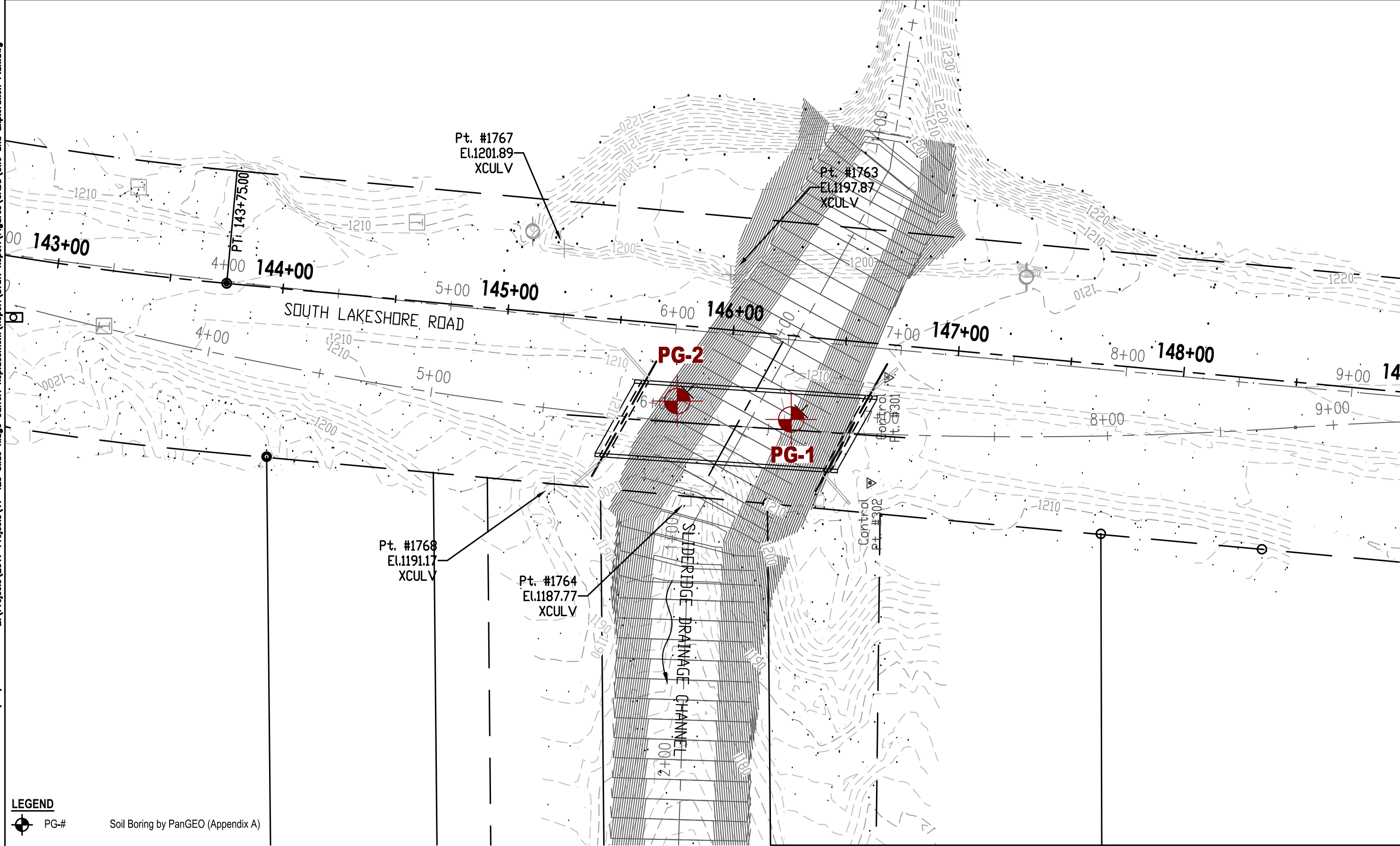
DRAWN BY: NTW  


Slide Ridge Culvert Replacement  
 South Lakeshore Road  
 Lake Chelan  
 Chelan County, Washington

VICINITY MAP

PROJECT NO. 17-425	FIGURE NO. 1
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Z:\Projects\2017 Projects\17-425 Slide Ridge Culvert Replacement\Report\Draft Report\Figures\CADD\Site and Exploration Plan.dwg  
 DATE: 2/26/2019  
 CHECKED BY: REK  
 DRAWN BY: NTW

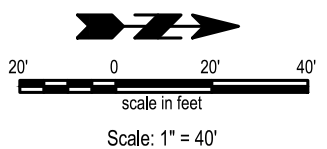


**LEGEND**

PG-# Soil Boring by PanGEO (Appendix A)

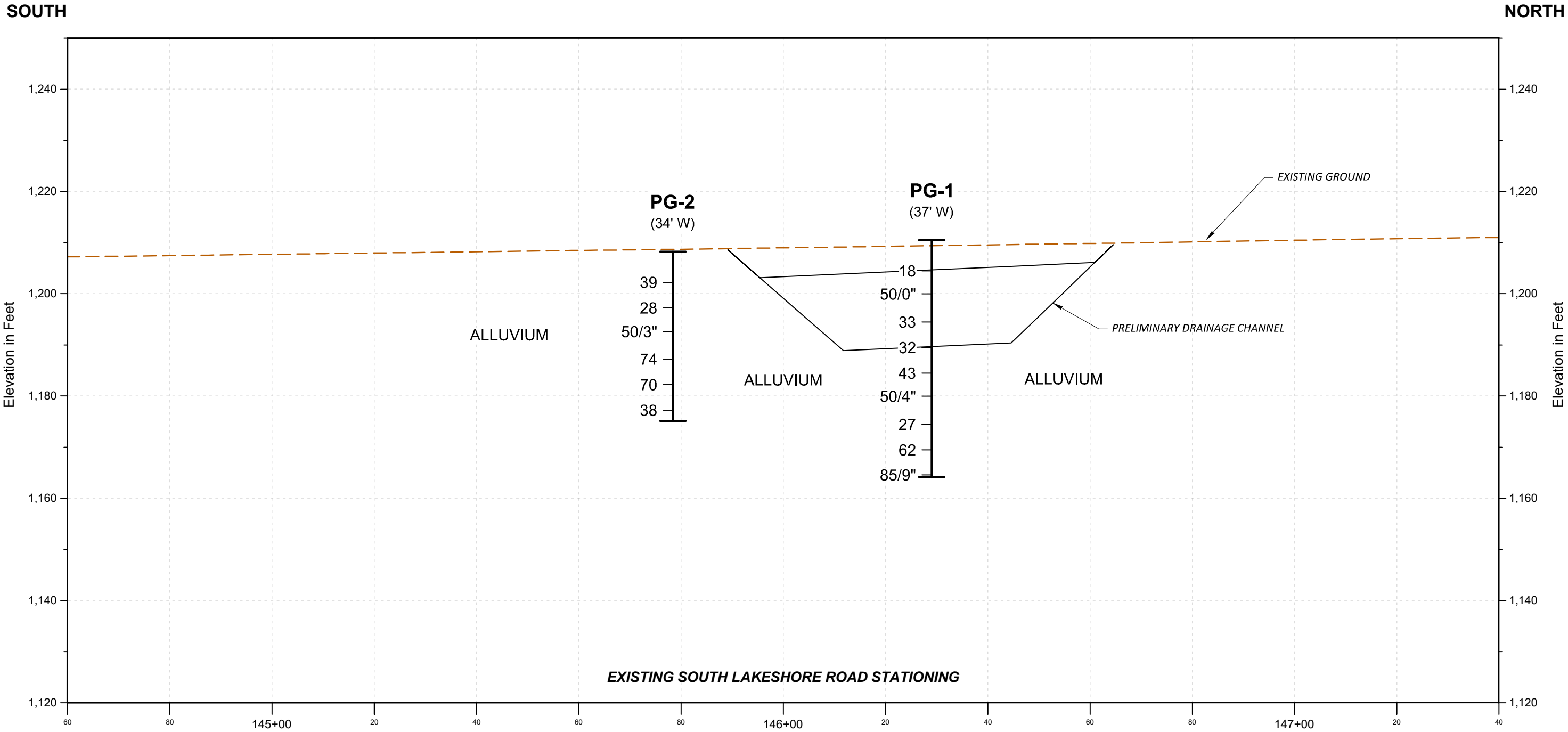
**NOTES**

1. Base map provided by KPFF on February 19, 2019.
2. All PanGEO boring locations were located on the current base mapping relative to existing and known site features. These locations should therefore be considered approximate and not a substitute for accuracy of field surveys.
3. All elevations referenced to North American Vertical Datum of 1988 (NAVD'88).

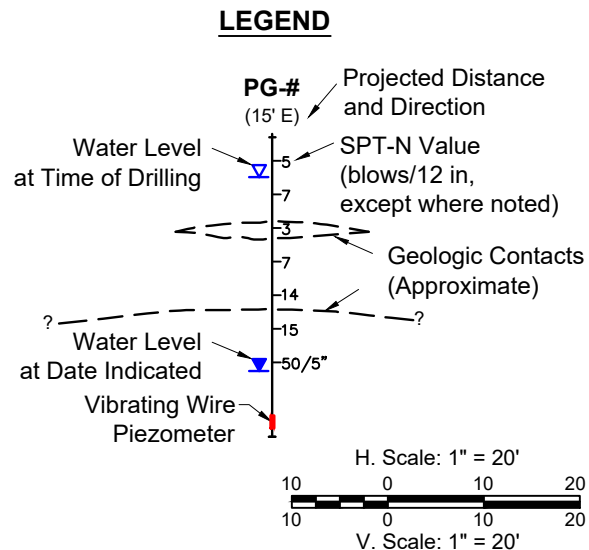


	Slide Ridge Culvert Replacement South Lakeshore Road Lake Chelan Chelan County, Washington		<b>SITE AND EXPLORATION PLAN</b>	
	<small>PROJECT NO.</small> 17-425	<small>FIGURE NO.</small> 2		

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 DATE: 2/27/2019  
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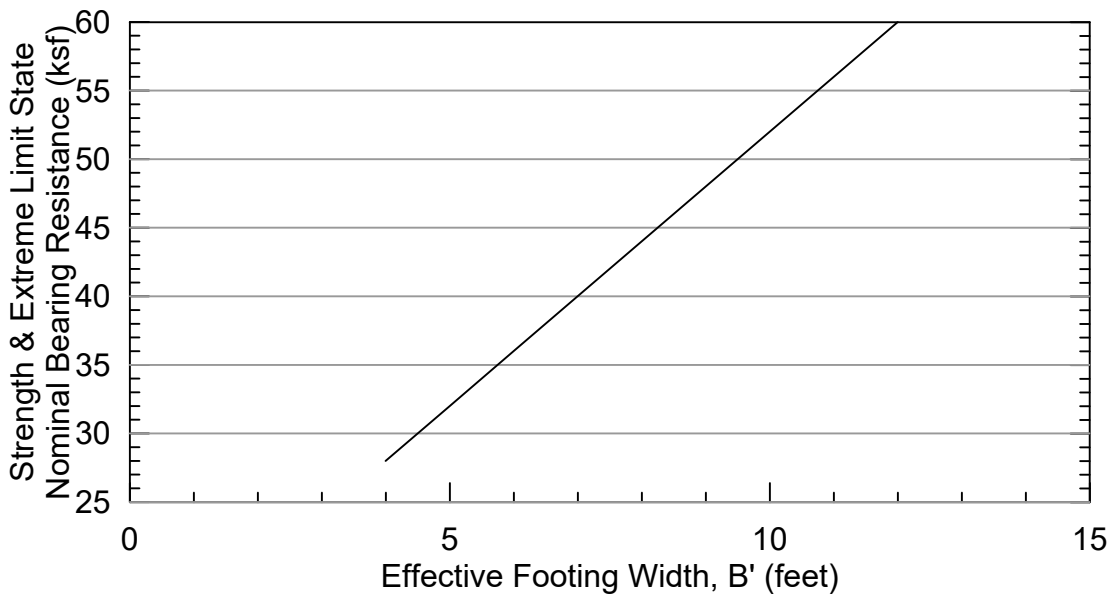
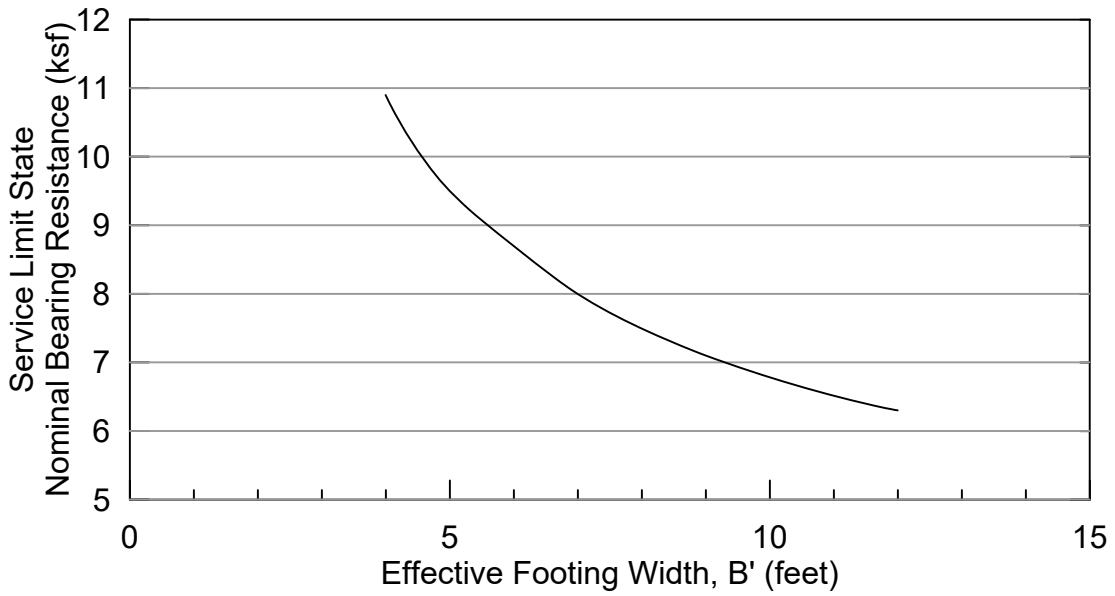
- NOTES**
- All PanGEO boring locations were located on the current base mapping relative to existing and known site features. These locations should therefore be considered approximate and not a substitute for the accuracy of field surveys.
  - Refer to Figure 2 for approximate boring locations.
  - Summary boring logs are presented in Appendix A.
  - Existing ground surface and profiles provided by KPFF on February 19, 2019.
  - All elevations referenced to North American Vertical Datum of 1988 (NAVD'88).
  - Geologic contacts between borings are inferred.



**ENGINEERING SOIL UNITS**  
See text of report for soil unit descriptions.

Alluvium (Ha): Silty gravel with sand to poorly graded gravel with silt and sand. See individual logs of test borings for information at specific locations.

	Slide Ridge Culvert Replacement South Lakeshore Road Lake Chelan Chelan County, Washington		<b>GENERALIZED SUBSURFACE PROFILE</b>	
	PROJECT NO. 17-425			FIGURE NO. 3



Notes:

- 1) Service limit state nominal resistance developed for settlement of 1-inch or less.
- 2) Resistance factor for strength limit state load combinations may be taken as  $\phi_b = 0.45$  per AASHTO LRFD 10.5.5.2.2.
- 3) Resistance factor for extreme limit state load combinations may be taken as 1.0.

17-425\_SpreadFigLRFD.grf w/17-425 B vs Q.xls 2/26/19 (09:05) REK



**Slide Ridge Culvert Replacement**  
 South Lakeshore Road  
 Lake Chelan  
 Chelan County, Washington

**NOMINAL BEARING RESISTANCE  
 SPREAD FOOTINGS**

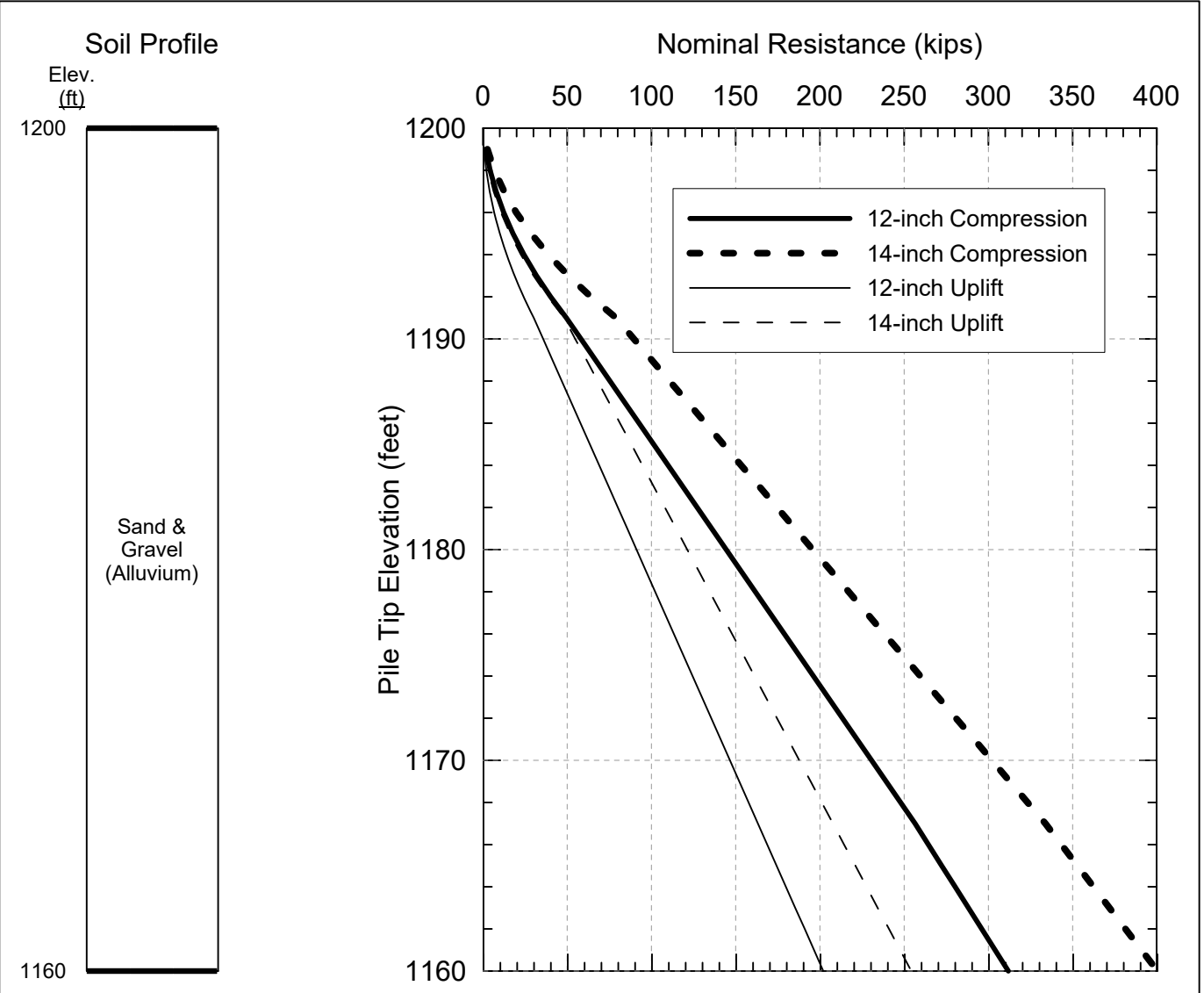
Project No.

17-425

Figure No.

4

17-425 H-Pile Capacity EI.gcf w/ 17-425 Pile Capacity\_HP\_12-inch.xls & 17-425 Pile Capacity\_HP\_18-inch.xls 2/26/19 (10:52) REK



**Notes:**

- 1) Axial resistance values are for 12- & 14-inch H-piles.
- 2) Axial resistance values are nominal (ultimate) values.
- 3) LRFD resistance factor:  
 $\phi_{dyn} = 0.55$  (axial resistance verified using the formula in Section 6-05.3(12) of the WSDOT Standard Specifications)
- 4) For piles driven to the required axial compressive resistance service limit state settlement is expected to be less than 1/2-inch.
- 5) Pile tip elevations provided in this chart for piles in compression are for pile quantity estimation purposes only. Actual pile penetrations should be determined on the basis of test piles and field verification using the dynamic method associated with the strength limit state resistance factor from Note 3, above.
- 6) Estimated penetration assumes a bottom of pile cap elevation of approximately 1200 ft.