Granite Falls Bridge No. 102 Replacement Project

Prepared for
Snohomish County
Everett, Washington

Prepared by
BergerABAM

August 2018
Design Memorandum

Granite Falls Bridge No. 102 Replacement Project
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Submitted to

Snohomish County Department of Public Works
Everett, Washington

July 2019

Submitted by
WSP
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Federal Way, Washington 98003-2600

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DESIGN MEMORANDUM
GRANITE FALLS BRIDGE NO. 102 REPLACEMENT PROJECT

PREPARED FOR
SNOHOMISH COUNTY DEPARTMENT OF PUBLIC WORKS
EVERETT, WASHINGTON

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7/19/2019
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Date
FACT SHEET

• **Proposal:** The Granite Falls Bridge No. 102 is located on the Mountain Loop Highway and spans across the Stillaguamish River gorge. The existing bridge is an 83-year-old fracture-critical structure classified as functionally obsolete. The new bridge will be wider and longer than the existing structure to meet current bridge standards. The bridge section will have two 12-foot vehicle lanes, two 5-foot bike lanes, and two 5.5-foot sidewalks on either side of the bridge, yielding a usable bridge width of 45 feet.

• **Proponent:** Snohomish County

• **Date of Construction:** Subject to availability of construction funds.

• **Agency:**
  Snohomish County Public Works
  3000 Rockefeller
  Admin East Building, Fifth Floor
  Everett, WA 98201

• **Contact Person(s):**
  Project Manager: Jim Weelborg
  Traffic Engineer: Mohammad Uddin

• **Required Permits:** TBD by Snohomish County

• **Authors:** WSP

• **Date of Issue:** 4 September 2018

• **Cost:** $18.3 Million Construction Cost, $24.9 Million Total Program Cost (Note Phase 1A preliminary engineering and County costs not included)
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EXECUTIVE SUMMARY

Purpose and Need
The Granite Falls Bridge No. 102 is located on the Mountain Loop Highway and spans across the Stillaguamish River gorge. The existing bridge is an 83-year-old fracture-critical structure classified as functionally obsolete. The existing bridge serves as the primary crossing for residents and businesses, including gravel quarries north of Granite Falls.

Alternatives Evaluated

Roadway Alignment: One major component of the roadway alignment alternative layouts was that it is necessary to maintain traffic on the existing bridge because no viable detours exist. The preferred alignment shifts the bridge downstream of the existing bridge, which allows the existing bridge to be used during construction, provides a drivable alignment, and accommodates a new bridge width of 45 feet that increases safety to public users whether they are traveling by vehicle, bicycle, or walking.

Bridge Design: Four different bridge types were evaluated; two concrete bridges, one steel bridge, and one hybrid steel/concrete bridge. The bridges considered were all girder bridges consisting of either two or three spans. In selecting the span configuration, a primary objective was to minimize the amount of work that needed to be conducted on or near the steep rock bluffs of the river gorge, while still using conventional bridge, wall, and roadway construction practices. Girder types included standard precast, prestressed and post-tensioned concrete girders, steel plate girders, and/or cast-in-place post-tensioned box girders. The bridge types were evaluated based on various factors, including constructability, environmental impacts, maintenance, and cost. The bridge type selected and approved by the Snohomish County is a two-span steel plate girder bridge.

The recommended two-span steel plate girder bridge accommodates the horizontal curvature of the roadway using straight girders, which minimized the girder pick weights. This was pivotal in terms of increasing the length of the main span of the bridge to avoid the rock bluffs of the river gorge and allowing for conventional girder erection practices.

Aesthetics
The existing Granite Falls Bridge No. 102 is an aesthetically pleasing decked arched steel truss bridge spanning the scenic Stillaguamish River gorge. Pedestrians commonly use the Washington Department of Fish and Wildlife access road to go down to the fish ladders, which also offer great views of the bridge. The new bridge will incorporate a pedestrian overlook.
centered on the river on each side of the bridge and incorporate an aesthetically pleasing concrete parapet railing.

**Cost Estimate**

The estimated construction costs for the recommended bridge alternative is $18.3 million in 2018 dollars. The total program cost, including approach roads, right-of-way acquisition, design and engineering, County project administration, and construction engineering and administration, is $24.9 million. See Section 10.0 of this memo for additional details.
1.0 INTRODUCTION

WSP (formerly BergerABAM) conducted a review of the November 2012 Granite Falls Bridge No. 102 type, size, and location (TS&L) report for constructability, cost-effectiveness, and conformance with the latest American Association of State Highway and Transportation Officials (AASHTO), Washington State Department of Transportation (WSDOT), and Snohomish County Engineering and Design Development Standards design standards.

The review, documented in the Final Design Review Memorandum submitted on 26 May 2016 prepared by WSP (included in Appendix H for convenience), discussed recommendations or modifications to the TS&L design. This report serves to document the additional studies conducted by WSP to evaluate the agreed upon recommended modifications to the design concept and to present the preferred alternative. The main modifications incorporated into the 30 percent design include

- Revising the original drilled shaft foundations to spread footings.
- Moving the pier locations back slightly from the steep bank to the top of bank.
- Revising the bridge section from two 15-foot lanes and 5-foot sidewalks to two 12-foot lanes, two 5-foot shoulders, and two 5-foot-6-inch sidewalks.
- Addition of pedestrian lookouts on both sides of the bridge over the river. The east lookout may need to be constructed after southbound traffic is switched over to the new bridge based on proximity to the existing southbound lane.
- Not including storm drainage on the bridge.

The preferred alternative is presented in the form of 30 percent construction documents in the appendices of this document.

2.0 EXISTING CONDITIONS

The Granite Falls Bridge No. 102 is located on the Mountain Loop Highway approximately 1.75 miles north of Granite Falls (see Figure 1). The existing structure spans the scenic Stillaguamish River gorge. The bridge and roadway are within a 100-foot Snohomish County right-of-way. The entire bridge and its approaches are within unincorporated Snohomish County. Land south (towards Granite Falls) and west of the existing bridge is owned by the Washington State Department of Fish and Wildlife (WDFW). Land east (upstream) of the bridge is privately owned and is zoned residential. To the east are gravel quarries that contribute much of the truck traffic across the bridge. Future development is expected at these quarries along with an associated increase in truck traffic.
The existing 336-foot-long deck arch truss structure spans the river gorge and serves as the primary crossing for residents and businesses, including gravel quarries east of Granite Falls (see Figure 2).

The structure has two 10-foot lanes providing a 20-foot crowned traveled way, 6-inch curb and gutter, and 3.5-foot sidewalk along both sides of the bridge. Pedestrians are separated from traffic by thrie-mounted guardrail with steel posts that are mounted offset from the face of the curb. The edge of the structure is protected by the original bridge railing. The posted speed limit within the project limits is 35 miles per hour (mph), and a double-yellow stripe delineates a no-passing zone across the bridge. The existing roadway approaches consist of one 11-foot lane in each direction. A wide shoulder is located on either side of the roadway south of the bridge. Visitors to the Stillaguamish River often use this area for parking, while WDFW personnel use it to access a fish ladder west (downstream) of the existing bridge.

The bridge approaches are protected by precast concrete barrier and guardrail. The south approach has a grade of approximately -6.0 percent sloping down toward the bridge, while the north approach rises from the bridge at approximately 5.5 percent. The existing grade across the bridge is approximately -0.5 percent with the low point being near the north abutment.

The existing bridge is 83 years old and functionally obsolete. Concerns with the existing structure include the following:

- Narrow lanes that result in safety concerns
- A limit of one truck at a time on the bridge
- High volume of truck traffic, about 1,900 trucks per day
- Fracture-critical nature of the structure
- High salt content in bridge deck

3.0 PROJECT GOALS AND OBJECTIVES

The original November 2012 TS&L report defined the following Snohomish County project goals and objectives.

- Develop a bridge and roadway solution that meets County standards and is constructible for a reasonable project cost
- Design a new bridge that provides full and independent transportation service capability
- Develop a solution that maintains traffic on the existing bridge throughout bridge construction

The November 2012 TS&L stated that the bridge solution must comply with the following general criteria.
Figure 2. Existing Bridge Location
• No fracture-critical structures.

• No part of the bridge or support members, whether permanent or temporary, shall be located within the high water mark of the river.

• Right-of-way impacts to adjacent property owners shall be identified.

• A TS&L study and preliminary design (30 percent basis of design) will be prepared for a new bridge to be constructed parallel to and completely separate from existing bridge.

• Future funding and technical design consideration will determine the next stage(s) of work.

• The bridge and roadway alternative must be practical to construct.

In addition to the goals and objectives identified in the November 2012 TS&L, WSP focused on minimizing construction risks on an already complicated site. The objective was to minimize the amount of work that needed to be conducted on or near the steep rock bluffs located on each side of the river, while still using conventional bridge, wall, and roadway construction practices.

4.0 PROJECT SITE ISSUES
The project site has significant issues to be addressed during design. These issues include construction over a deep gorge, bridge foundations in rock, large cut walls, right-of-way acquisition, environmental impacts (e.g., wetlands, stream buffers), drainage, utility relocations, and maintenance of traffic during construction. Private properties are located along the upstream side of the existing bridge, and WDFW property is located along the downstream side. Wetland locations east of the existing bridge have been flagged, and environmental impacts will need to be addressed. Utilities in the vicinity include a water line, overhead power lines, and future underground cable and phone lines. Geotechnical challenges include foundations in or on rock formations, existing slide areas (downstream and across river from Granite Falls, see Geotechnical Report), and existing loose fill in the northeast quadrant (upstream and across the river from Granite Falls).

5.0 COMMON DESIGN PARAMETERS
Detailed Roadway Design Parameters are listed in the Final Design Review Memorandum, which is included as Appendix H. The structural design criteria are included in Section 7.0 of this report.

The basic roadway criteria include the following.
- Snohomish County Engineering Design and Development Standards (EDDS), January 2016 Edition
- AASHTO’s A Policy on Geometric Design of Highway and Streets, 2011
- Roadway access during construction shall maintain a single lane of traffic in both directions during non-working hours and allow limited closures during working hours.

### Table 1. Roadway Design Criteria

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| Functional Classi| Minor Arterial (Urban)  
Rural Standards will be followed for the cross section due to the Rural nature of the project | TS&L                                                                      |
| Design Speed      | 45 mph                                                                   | TS&L                                                                      |
| Posted Speed      | 35 mph                                                                   |                                                                           |
| Truck Turning     | WB-67 Truck                                                              |                                                                           |
| Minimum Vertical Curve | 135 ft                  | EDDS 3-110                                                                |
| Minimum Horizontal Curve | 1,050 ft                 | AASHTO for Low-Speed Urban Streets                                         |
| Stopping Sight Distance | 360 ft                  | Table 3-6 EDDS                                                            |
| Object Height     | 3.5 ft                                                                   | EDDS                                                                      |
| Driver Eye Height | 1.5 ft                                                                   | EDDS                                                                      |
| Maximum Grade     | 10%                                                                      | Table 3-5 EDDS                                                            |
| Minimum Grade     | 0.05%                                                                    | Table 3-07                                                                |
| Design Clear Distance | 10 ft                    | EDDS Section 4-15                                                         |
| Roadway Section   | Two 12-ft lanes with 8-ft shoulders                                      |                                                                           |
| Bridge Section    | Two 12-ft lanes with 5-ft shoulder and 5.5-ft sidewalk on each side      |                                                                           |
| Curb Height       | 12 in.                                                                   | TS&L, Requested by County                                                 |
| Roadway Cross Slope | 2.0%                      | TS&L, EDDS Standard Plan 3-010                                             |
| Bridge Railing    | None                                                                     | NA                                                                        |
| Superelevation    | Normal Crown                                                             | TS&L                                                                      |
| Bridge Drainage   | None                                                                     | Low point off of the bridge                                               |
| Clearance from Exiting Bridge | 5 ft minimum              |                                                                           |
| Design Traffic Year | 2040                      | TS&L                                                                      |
### Category | Criteria | Reference
--- | --- | ---
2040 ADT | 6,134 (two-way) | TS&L
2040 Truck % | 30.5% | TS&L
Driveway Intersection Sight Distance | 360 ft | EDDS Table 3-8
Downhill Stopping Distance | 392 ft | Interpolated from Table 3-7 EDDS
Uphill Stopping Distance | 336 ft | Interpolated from Table 3-7 EDDS

### 5.1 Typical Section
The design team was tasked with determining the feasibility of providing a wider bridge roadway cross section than that shown in the November 2012 TS&L report in order to increase safety to public users whether they are traveling by vehicle, bicycle, or walking. The TS&L typical bridge section was designed with two 15-foot lanes with 5-foot sidewalks on either side of the bridge, yielding a usable bridge width of 40 feet. It was determined with coordination with the County that the preferred bridge section would have two 12-foot vehicle lanes, two 5-foot bike lanes, and two 5.5-foot sidewalks on either side of the bridge, yielding a usable bridge width of 45 feet. By adjusting the roadway alignment and geometrics as discussed in Section 5.2 below, the increased bridge width was achieved. Including the width of the bridge barriers, this yielded an out-to-out width of the bridge of 48 feet 2 inches.

The proposed roadway section off the bridge includes two 12-foot lanes with 8-foot paved shoulders per Snohomish County Standard Plan 3-030B. This is the standard section for two-lane major collector rural arterials with average daily traffic (ADT) greater than 2,000. No sidewalk or bike lines will be provided on the approach roadway to the bridge. Directly after the traffic barrier ends, the roadway and guardrail taper from the 45-foot usable bridge width to the standard 40-foot roadway approach section.

At the south end of the project, the 12-foot lanes will taper into the existing 11-foot lanes in 50 feet. This is more than the minimum of 35 feet, but we believe the 50-foot transition edge stripes will be less noticeable to the driver. The guardrail on the right side will end at the project limits because there is no existing guardrail to tie into. However, our review indicates a guardrail may be warranted for the existing conditions south of the project limits. On the left side, the guardrail will be tied into the existing guardrail.

At the north end of the project, the 12-foot lanes will taper into the 11-foot lanes just south of the existing culvert. This should allow for no road widening at the culvert. The lane lines transition over the length of the horizontal curve that tie the proposed alignment back into the existing alignment. Although the roadway width tappers back to existing prior to the existing culvert, the exiting culvert and stream channel may pose a roadside hazard. In the existing condition, there are no guardrails on either side of the
roadway at the location of the culvert; but as the design moves forward, the County needs to consider if a guardrail is warranted.

5.2 Roadway Geometrics and Alignment

The alignment of the TS&L was based on a roadway cross section of 40 feet as defined in Section 5.1 above. This allowed for the WDFW fish ladder access road to be constructed on the downstream side of the bridge close to its current (existing) location.

As discussed in Section 5.1, in order to increase the usable width across the bridge, the design team made adjustments to the alignment that was developed in the original November 2012 TS&L report. The TS&L report used a 413-foot-long, 1,500-foot radius curve at the south end of the project; a 667-foot-long, 6,900-foot radius curve over the bridge; and a 218-foot-long, 2,500-foot radius curve to tie back into the existing alignment at the north end of the project. The minimum curve used in the TS&L study was 1,500 feet, which is more than what is required. It was determined that the larger radii added longer transitions lengths between the existing and proposed roadway, thus, lengthening the project limits.

WSP reduced the curve radius at both ends of the project to 1,050 feet (the minimum allowed), which shortened the project limits. The curve radius across the bridge was also reduced from 6,900 feet in the TS&L report to 4,500 feet on the 30 percent plan. There are two cross road culverts near the limits of the project, and shortening the project limits eliminated the need to replace or modify the existing roadway culverts. The 1,050-foot radius still allows for the entire roadway to remain as a normal crowned section throughout, which eliminates superelevation transitions and helps drainage flow away from the roadway centerline.

There were minimal changes made to the roadway profile from the TS&L design. The profile was adjusted to remove the 1 percent grade break at the north of the project. The vertical curves near the project ends were also adjusted to accommodate the shortened project limits caused by the change in horizontal alignment.

The vertical profile on the proposed alignment includes two sag vertical curves and one crest vertical curve. These curves include the following.

1. 450-foot-long vertical sag curve from Stations 11+15 to 15+65. Negative 4.57 percent in and positive 1.1 percent out.

2. 290-foot-long vertical sag curve from Stations 21+75 to 24+65. Positive 1.1 percent in and positive 4.9 percent out.

3. 135-foot-long vertical crest curve from Stations 25+12.5 to 26+47.5. Positive 4.9 percent in and positive 2.92 percent out.
The low point of the roadway is located at Station 14+78, which is well off the bridge and located at a desirable location where stormwater can be collected.

5.3 Right-of-Way and Easements
After completion of the November 2012 TS&L study and prior to the onset of Phase 1B, the County developed the right-of-way exhibit included in Appendix L. As part of the studies conducted under Phase 1B, it was determined that additional right-of-way and permanent easements are required. The additional right-of-way and/or permanent easements required are associated with rock bolts in the vicinity of Pier 3, and the tieback anchors needed for Walls 5 and 7; reference Appendix B, 30 percent plans for wall locations.

5.4 Hydraulics
The proposed bridge does not touch or impact the waters of the Stillaguamish River. The Snohomish County Surface Water Management group prepared a hydraulic model for the bridge site. The proposed bridge deck and foundations are well above the 100-year flood elevation of 335 feet. Reference the TS&L Appendix K for additional details on the hydraulic analysis.

Tetra Tech performed a bathymetric survey of the Stillaguamish River bed to determine if there was any underscour of the rock cliffs below the bridge. The design of the rock bolts of the bridge foundations will take into account the rock underscour as recorded in the bathymetric survey.

5.5 Surface Water Management
Basic runoff treatment is required for runoff on all new and replaced pollution-generating surfaces on the project because the project is considered a road-related redevelopment project, and the new hard surface total is more than 5,000 square feet.

The site is located downstream of the confluence of Cranberry Creek on the South Fork of the Stillaguamish River. According to Appendix I-E of the Snohomish County Drainage Manual, the project is flow control exempt.

Because of the steep slopes of the project and known shallow rock layer below the ground, it is not feasible to treat stormwater via infiltration on the project. Much of the proposed roadway is located adjacent to either steep slopes or site walls, which makes it difficult to treat runoff via roadside features like media filter drains, vegetated filter strips, or roadside dispersion. There are viable locations to place bioswales for treatment on both the north and the south sides of the bridge.

Stormwater on the north approach of the bridge can be curbed, collected, and conveyed to the bioswale on the north. Stormwater from the south side of the bridge and stormwater from the bridge can be curbed, collected, and conveyed to a bioswale on the south side of the bridge. After stormwater is treated in the bioswales, it can be conveyed to a dispersion trench on the bank on the river.
A gutter analysis was performed, and it is necessary to place any drainage structures on the bridge. Bridge drains will be needed. Stormwater can be collected directly off the bridge and conveyed south to the bioswale.

One item that will need to be addressed in the Temporary Erosion and Sediment Control (TESC) plan is the timing of the bioswale installation. Due to the location of the bioswales, they cannot be constructed until after the bridge demolition is complete, which is at the end of the project. The contract will need to address treatment of stormwater prior to the bioswales becoming operational. Drainage layout is shown on the 30 percent plans in Appendix B.

Additional details about stormwater can be found in the future storm drainage report.

5.6 Environmental

Snohomish County is procuring all permits for the project. During the kickoff meeting, excavation means and methods were discussed. It is anticipated that controlled rock blasting will be needed when constructing excavation for the bridge substructure and foundation elements. Concerns were expressed by the County as to the potential environmental constraints regarding blasting. WSP is currently working on the Index-Galena project with Snohomish County and Shannon & Wilson, and it is our understanding that blasting is allowable on that project. It is our assumption that the November 2012 TS&L schedule and cost estimate included blasting as a means and method for excavating rock cuts. Other means and methods are more time consuming and will add duration and cost to the TS&L schedule and budget. The 30 percent construction schedule and cost estimate assume blasting is allowable.

Additional environmental considerations shall include

- Tree Removal: The roadway alignment is being adjusted as part of the project. The adjacent land is forested, and tree removal will be required to build the project. Consideration should be given to the excavation needed to facilitate construction of the walls and/or bridge/roadway adjacent to the walls, and its impact on the root structure of the tree.

- Consideration for the rock bolting operation will need to be evaluated for clearing, contractor access, and debris containment. The 30 percent rock bolting plan shows rock anchors being installed down to Elevation 320 on the north slope and Elevation 300 on the south slope. The TS&L report bridge plans show the Ordinary High Water Mark (OHWM) at Elevation 325. The lower limits of the rock bolts on the north slope are near the OHWM and on the south slope three rows appear to be below the OHWM. The lower elevation of the rock bolts will need to be evaluated as the design moves forward. The slopes will also need to be selectively cleared of any vegetation and soil covering the rock face within the rock bolting area. It is our assumption that a debris catchment fence will need to be installed along the lower
slope limits on both the north and south banks to prevent debris from falling into the river. Localized containment at each anchor will need to be evaluated for containing drill spoils, excess grout, and concrete. Additional evaluation for containment will be needed as the design moves forward.

- Catchment Fence: Excavation for Piers 2 and 3 on the downbank side along with the southern end Wall 5 will be on steep slopes. Something more robust than silt fence will be needed to prevent debris from falling into the river. It is anticipated that this will be more of a catchment fence than a silt fence. One concept that has been used in the past is a geotextile lined chain link fence with post anchored into the rock slope. As the design progresses, we will evaluate options with Shannon & Wilson.

5.7 Utilities

5.7.1 Power Lines
The existing overhead power is on the west side of the bridge and will be in the way during construction work. The power company prefers to move their lines only once rather than temporarily relocate the lines during construction and move them into final location at the end of the project. This may be possible by relocating the existing power lines from the west side of the bridge to a permanent location on the east side of the existing bridge early on in construction. This will allow bridge construction and bridge demolition to proceed unimpeded by the power lines and save costs for the power company by relocating only once. A schematic of the proposed power line relocation is shown in the 30 percent design plans in Appendix B. The proposed relocation will need to be discussed with Snohomish County Public Utility District because the design makes assumptions based on pole spacing.

5.7.2 Water
The existing 8-inch water line will be replaced as a result of this project. The 8-inch line is inside a 16-inch casing across the existing bridge. There are fire hydrants on the north and south sides of the bridge within project limits that will have to be replaced with the new roadway alignment. The TS&L plans show the new water line shares a bay with a storm line and makes a fairly rapid vertical and horizontal transition off the bridge to tie into the existing water line. The proposed water line will be installed on the downstream side of the bridge suspended under the bridge. The existing water line will remain operational throughout construction until the proposed line is tied in and completed. We have evaluated the water line tie-in locations during the 30 percent design and have extended them from the locations shown in the TS&L. On the Granite Falls side of the bridge, the tie-in is located approximately 400 feet south of the abutment, and on the quarry side, the tie-in is located approximately 500 feet north of the abutment. The existing water line will need to be temporarily relocated in the vicinity of new Bridge Pier 1. A fairly extensive shoring wall is needed on the west side of the existing Pier 1 to allow construction of new Bridge Pier 1. To avoid conflicts during construction of the
shoring wall, approximately 120 feet of existing water line will need to be relocated. The conceptual relocation layout is shown in the 30 percent plans in Appendix B.

5.7.3 Communication
From our field review, Frontier Communications has fiber-optic cable that runs on the east side of the road and bridge. At the existing bridge, it is overhead. The underground segment at the south approach may need to be relocated to allow for final grading and bioswale construction. It is recommended that this facility be potholed in the vicinity of Wall 4 bioswales and in the vicinity between Stations 27+00 and 30+00 to ensure there are no conflicts with proposed construction. If power is relocated to the east side of the bridge, the cable could go aerial on the new power poles. The cable could also be placed in conduits running under the new bridge if preferred.

5.8 Key Project Features
During preparation of the 30 percent plans, several items were identified as key project features. These included certain work elements that involved items on the bridge and off the bridge. Examples include rock bolts, roadway lighting, bridge lighting, bridge barrier type, sidewalk height on bridge, utility relocations, guardrail needs, existing culverts, and other items. A listing of these items, including along with details of the item and if a design exception is needed, is included in Appendix G.

6.0 GEOTECHNICAL
Shannon & Wilson conducted additional geotechnical investigations as part of Phase 1B. The additional investigations, combined with prior investigations, provided the basis for the draft geotechnical report included in Appendix A.

7.0 BRIDGE ALTERNATIVES
The Granite Falls Bridge replacement project began with a multi-phase study that evaluated different structure types and span configurations. A summary of all the structure types and span configurations considered is included in Appendix J. This design report discusses the four structure types chosen for further evaluation. It should be noted that the descriptions of the four bridge alternatives were written at a point in time for comparison purposes only and may not be consistent with the description of the preferred alternative described in Section 12 and shown in Appendix B.

The four structure types, and the sections that describe them, are as follows.

- Alternative 1: Three-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Precast Concrete Girder Main Span Drop-In
- Alternative 2: Two-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Precast Concrete Girder Main Span Drop-In
- Alternative 3: Two-Span Bridge: Steel Plate Girder
Alternative 4: Two-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Steel Plate Girder Main Span Drop-In

The four different bridge alternatives evaluated follow state-of-the-art construction methods. The alternatives were evaluated based on superstructure constructability, schedule impacts, construction costs, and inspection and maintenance (see Appendix I for the Bridge Alternative Evaluation Matrix).

7.1 Design Criteria and Assumptions
The proposed bridge designs conform to the following standards and criteria.

- WSDOT, Bridge Design Manual (BDM) (WSDOT Manual M 23-50.16), June 2016, and applicable design memoranda
- Operational Importance Classification “Typical” (for BDS §1.3.5)
- Design Live Load: AASHTO HL-93
- Seismic Design: The seismic design will comply with the Guide Specifications and modifications adopted by WSDOT; the seismic provisions of the BDS are not applicable.
- Utilities: The new bridge will carry a water line. Final design will account for the appropriate load effects. For this study, the preliminary design was conducted using a utility dead load allowance of 53 pounds per linear foot (total).

Notable assumptions made during the bridge study are as follows.

1. The existing bridge columns will be removed. It is assumed that the existing bridge footings will remain.

2. The new bridge will have a typical concrete deck width equal to 46 feet 9 inches measured from edge-to-edge of deck.

3. All new work activities will be a minimum of 5 feet offset from the existing bridge at all times the existing bridge is carrying live traffic.

4. A single set alignment was used for the bridge study. Details of the roadway alignment can be found in Section 5.2 of this document and in Appendix B. The
alignment has a radius of 4,500 feet throughout the bridge extents; however, the girders are assumed to be straight from pier-to-pier.

5. The vertical profile of the proposed roadway is linear, sloping down to the east.

### 7.2 Superstructure

#### 7.2.1 Alternative 1: Three-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Precast Concrete Girder Main Span Drop-In

Alternative 1 (see Figure 3) features a three-span, 416-foot-long bridge with span lengths of 88 feet, 260 feet, and 68 feet, measured south to north. The superstructure consists of cast-in-place post-tensioned box girder end spans with standard WSDOT precast, prestressed WF95PTG concrete girders for the drop-in span. The cast-in-place post-tensioned box girder end spans start at the abutments and cantilever out 35 feet past the intermediate piers into the main span. The precast concrete drop-in girders are 190 feet long and use sand lightweight aggregate concrete to minimize the pick weight of the girders. Due to the inability to ship a single 190-foot precast girder to the project site, the precast drop-in girder needed to be field spliced from two 35-foot end segments and a 120-foot middle segment at an off-site splicing yard.

![Figure 3. Alternative 1 – Three-Span Bridge; Cast-in-Place Post-Tensioned Box Girder End Spans with Lightweight Precast Concrete Girder Main Span Drop-In](image)

The cast-in-place box girders contain a two-staged post-tensioning system. The first stage of post-tensioning is installed prior to removing the falsework and installing the drop-in precast girder span. The second stage of post-tensioning is installed continuously through the cast-in-place box girder end spans and the precast drop-in girders (i.e., continuous from end-to-end of the bridge) after the closure pour between them has cured, but prior to pouring the main span deck.
The precast girders have pre-tensioning and a two-stage post-tensioning system. The pre-tensioning is required to haul the individual segments to the splicing yard. Available haul routes and horizontal clearances limit the maximum girder length to 135 feet (see Appendix F for Ott-Sakai Constructability Review). The first stage of post-tensioning, performed at the off-site splicing yard, provides continuity between the individual girder segments to create the 190-foot-long girder. The girders will then be transported to the site and set into place. The second stage of post-tensioning is the continuity post-tension described above.

The precast girders will be made to act composite with a cast-in-place concrete bridge deck. The cast-in-place box girders will be continuous for dead loads, superimposed dead loads, and live loads over the pier. The precast girders will only be continuous for superimposed dead loads and live loads.

7.2.2 Alternative 2: Two-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Precast Concrete Girder Main Span Drop-In

Alternative 2 (see Figure 4) features a two-span, 351-foot-long bridge with span lengths of 88 feet and 263 feet, measured south to north. The superstructure consists of a cast-in-place post-tensioned box girder end span with standard WSDOT precast, prestressed WF95PTG concrete girders for the drop-in span. The cast-in-place post-tensioned box girder end spans start at the abutments and cantilever out 35 feet past the intermediate pier into the main span. The precast concrete drop-in girders are 225 feet long and use sand lightweight aggregate concrete to minimize the pick weight of the girders. Due to the inability to ship a single 225-foot precast girder to the project site, the precast drop-in girder needed to be field spliced from two 45-foot end segments and a 135-foot middle segment at an off-site splicing yard.

![Figure 4. Alternative 2 – Two-Span Bridge; Cast-in-Place Post-Tensioned Box Girder End Span with Lightweight Precast Concrete Girder Main Span Drop-In](image-url)
Similar to Alternative 1, the cast-in-place box girders have a two-stage post-tensioning system, and the precast girders have pre-tensioning and a two-stage post-tensioning system. Once the drop-in precast girders are in place and the closure pour has cured, the second stage of post-tensioning is installed to create continuity along the length of the bridge.

The precast girders will be made to act composite with a cast-in-place concrete bridge deck. The cast-in-place box girders will be continuous for dead loads, superimposed dead loads, and live loads over the pier. The precast girders will only be continuous for superimposed dead loads and live loads.

7.2.3 Alternative 3: Two-Span Bridge: Steel Plate Girder
Alternative 3 (see Figure 5) features a two-span, 351-foot-long bridge with span lengths of 88 feet and 263 feet, measured south to north. The superstructure uses steel plate girders along the entire length of the bridge. The end span steel plate girder segment will start at Pier 1 and cantilever out past the intermediate pier (Pier 2) 48 feet. The main span (Span 2) drop-in girder consists of two segments that are at-grade field spliced on-site just prior to erection. The superstructure depth, measured from top of bridge deck to the top of bottom flange, is 10 feet 6 inches.

Steel plate girders are conventionally fabricated with a horizontal curve matching the horizontal curvature of the bridge deck. Doing so results in a constant overhang width along the length of the bridge. Curved steel plate girders are required to be erected in pairs in order to prevent the girders from rolling during erection. Based on site constraints and their impacts on the crane pick radius for girder erection, the girders for the Granite Falls Bridge No. 102 project are straight so that a single girder line could be picked. Picking a single girder line in-lieu of a pair of girders kept the pick weights down, which allowed the use of more conventional and readily available crane sizes.

![Figure 5. Alternative 3 – Two-Span Bridge; Steel Plate Girder](image_url)
The bridge span arrangement is not balanced and if unrestrained, the end of the girders at Pier 1 will want to lift off their bearings once the Span 2 girder segments are erected. This differs from the other concrete alternatives, whereby the girders has sufficient weight to prevent uplift from occurring at Pier 1. Therefore, a temporary hold-down assembly is required to transfer the uplift forces directly to the substructure elements of Pier 1. From a permanent perspective, to avoid the need of a mechanical hold-down assembly and a potential maintenance concern, the Pier 1 end diaphragm will be cast integrally around the steel girders and with the abutment stem wall and spread footing. Thermal expansion and contraction displacements in the longitudinal direction of the bridge will be designed to be accommodated at Pier 2 and Pier 3.

The steel plate girders were made to act composite with the cast-in-place concrete deck. Unlike the other alternatives, this option provides continuity along the full length of the bridge under dead loads, superimposed dead loads, and live loads.

7.2.4 Alternative 4: Two-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Steel Plate Girder Main Span Drop-In

Alternative 4 (see Figure 6) features a two-span, 351-foot-long bridge with span lengths of 88 feet and 263 feet, measured west to east. The superstructure consists of a cast-in-place post-tensioned box girder end span with steel plate girders for the drop-in span. The cast-in-place post-tensioned box girder end spans start at the abutments and cantilever out 35 feet past the intermediate pier into the main span. The main span (Span 2) drop-in girders consists of two segments that are at-grade field spliced on-site just prior to erection. The superstructure depth, measured from top of bridge deck to the top of bottom flange, is 10 feet 6 inches.

![Figure 6. Alternative 4 – Two-Span Bridge; Cast-in-Place Post-Tensioned Box Girder End Span with Steel Plate Girder Main Span Drop-In](image)
The cast-in-place box girder will have post-tensioning installed prior to removing falsework and installing the drop-in span. The steel plate girders will have a field splice in Span 2 with a closure pour at the cantilevered end of the cast-in-place box girder.

Similarly to Alternative 3, the steel plate girders will be straight with a variable overhang along the length of the bridge.

The steel plate girders will be made to act composite with a cast-in-place concrete bridge deck. The cast-in-place box girder will be continuous for dead loads, superimposed dead loads, and live loads over the pier. The steel plate girders will only be continuous for superimposed dead loads and live loads.

7.3 Substructure and Foundation
For alternative comparison purposes, the substructure and foundation types and locations were kept the same between each alternative. The substructure consisted of a cross beam supported by a rectangular concrete column at the intermediate piers, and L-shaped abutments. Spread footings were used at each pier location.

7.3.1 Cross Beams
The cross beams were assumed to be single staged dropped (i.e., located below the bearing elevation of the superstructure girders) variable depth (4 feet 6 inches minimum to 6 feet 6 inches maximum) hammerhead cross beams supported by a single column located at the pier centerline. The width of the cross beams was taken to be 8 feet.

Because the cross beams are single staged, there will not be a Stage 2 diaphragm pour, and the superstructure will remain continuous over the intermediate piers for all dead load, superimposed dead load, and live loads.

7.3.2 Columns
The intermediate piers consist of a single column located directly under the centerline of bridge. The column geometry (size and shape) was made equal and have been assumed to be 13 feet 9 inches wide by 8 feet thick. The longer dimension is oriented perpendicular to the longitudinal direction of the bridge. The column geometry was sized to meet minimum seismic strength requirements per Section 8.7 of the AASHTO Guide Specifications.

7.3.3 Foundations
Due to the site conditions, spread footing foundations that bear on bedrock will be used for the new bridge. The intermediate piers were located off of the steep rock slope and onto the flatter terrain just beyond the top of the slope. A minimum of 4 feet horizontal offset from the edge of the footing to the edge of the rock bluff was recommended by Shannon & Wilson (see Appendix A).

Pier 1 and Pier 4 consist of a rectangular footing measuring 20 feet wide by 46 feet 9 inches long. Pier 2 and Pier 3 consist of a hexagonal footing measuring 20 feet wide
between parallel exterior faces of the footing. The thickness of the spread footings were assumed to be 5 feet.

The footings were sized to remain in compression during a service and strength limit state. An additional hold-down system will be required to resist the overturning moment at the bottom of the spread footing during a seismic event.

8.0 ADDITIONAL STRUCTURAL STUDIES

8.1 Maintenance and Inspection
The proposed bridge will be maintained by the County. The County has expressed a preference for concrete bridges because of their long-term durability and because they do not require painting. Bridge Alternatives 1 and 2 are constructed almost entirely of concrete and, as a result, are very durable bridges requiring little maintenance. Bridge Alternatives 3 and 4 will be constructed from steel members, which will require maintenance in order to maintain durability. A method to minimize the maintenance with the steel bridge alternatives would be to metalize the steel prior to painting the steel members. The advantages of metalizing prior to painting are discussed in more detail in Section 10.2, Life-Cycle Cost Analysis.

The large difference in length of the main span relative to the end spans generates liftoff of the girder ends at the end piers, or uplift forces at the end piers if the girder is restrained from lifting off. In order to maintain drivability of the roadway, girder liftoff needs to be restrained. For the two-span bridge alternatives, girder liftoff is restrained by fully integrating the bridge girders with a Stage 2 diaphragm concrete pour that is made integral with the stem wall of the abutment. The intermediate pier allows for free longitudinal translation of the bridge superstructure, and all bridge movement is accommodated through one expansion joint at Pier 3 (the northernmost end pier). For the three-span alternatives, a similar concept would be applied; however, large counterweights or hold-down devices are required at each abutment, and one of the abutments needs to accommodate the longitudinal movements of the bridge, and thus a mechanical device would be required to prevent the girders from lifting off, but also to allow longitudinal movement of the bridge. Having such a mechanical device creates a maintenance concern, especially located adjacent to an expansion joint, which are notorious for causing maintenance issues.

8.2 Aesthetics
Bridge aesthetics were not specifically addressed as part of Phase 1B services. Based on discussions with the County, the bridge will incorporate a pedestrian overlook within the main span of the bridge. However, the overlook on the east side may need to be staged to ensure that it is not in conflict with southbound traffic on the existing bridge. Bridge barrier will be Texas Department of Transportation C412 bridge barrier, which is an aesthetically pleasing concrete parapet barrier system. Off the bridge, it has been
assumed that the walls will receive a concrete fascia; however, no discussions have occurred regarding finish treatments.

8.3 Geometric Constraints
During preparation of the 30 percent plans, we worked with Ott-Sakai & Associates to determine the minimum distance from back of rail of new bridge to back of rail of existing bridge. This was determined to be 5 feet based on past experience of constructing a new bridge directly adjacent an existing bridge. This will allow for shoring installation at Pier 1; Pier 1 footing construction, including sufficient workroom between the shoring wall and footing forms; and installation of overhang soffit forms and walkways. We also adjusted the alignment to reduce the amount of excavation and wall height on the north side of the bridge.

8.4 Approach Structures and Retaining Walls
There are a number of permanent and temporary walls needed on the project. A summary of the walls is provided in Table 2 below. Each of the walls are shown on the 30 percent plans included in Appendix B.
Walls 1 is a structural earth (SE) wall being used to retain the fill material required to reconfigure the parking area adjacent to the fish ladder access road based on realignment of the Mountain Loop Highway through the project extents. Wall 2 is an SE fill wall required to retain the fill material adjacent to Pier 1 such that the fish ladder access road can be constructed.

A temporary cut wall, Wall 3, is required to facilitate access to construct the new Pier 1 and Pier 2 bridge foundations while maintaining traffic on the Mountain Loop Highway. Wall 3 also allows room for construction of Wall 2 (i.e., provides space for the straps). Although temporary, the majority of Wall 3 will be buried in-place due to the sequencing of construction and phasing of traffic. Wall 3 will be a soldier pile cut wall that will need to be anchored with permanent ground anchors. The ground anchors will extend under the existing Mountain Loop Highway. In order to construct the final site grading, some of the permanent ground anchors will need to be detensioned while the backfill behind Pier 1 is being placed.

An existing water line is located behind the proposed Wall 3. It is recommended that the existing water line be potholed to determine its location. Our current assumption is that it is in conflict with the shoring wall and approximately 120 feet of the water line will need to be temporarily relocated.

A cantilever soldier pile, Wall 4, will need to be constructed in-line with the Pier 1 abutment to facilitate construction of the south side bioswale allow for final grading adjacent to Span 1 of the new bridge, and removal of a portion of Wall 3 once the new bridge is in service. The northernmost pile of Wall 4 will be installed as part of the Wall 3 construction. This pile will be strategically placed in order to provide function and alignment for each wall, see 30 percent plans for details.

The north end of the new bridge requires cutting into the hillside to construct the bridge. Wall 5 is being provided to facilitate removal of the material required to construct the bridge. Wall 5 will be a soldier pile cut wall anchored to rock using rock anchors. The portion of Wall 5 south of Pier 3 is detailed as a permanent wall. The portion of Wall 5 north of Pier 3 is temporary and will be as required by the contractor to facilitate...
construction. Any portion of temporary wall constructed will most likely be buried and left in place.

Wall 6 is an SE wall required to retain the roadway approach fill on the quarry side of the bridge. The wall is L-shaped (in plan view) with the portion of the wall perpendicular to the alignment of Mountain Loop Highway being set back from Pier 3 to prevent fill from being placed in close proximity to the rock bluff, and also to allow for minimal rock excavation at Pier 3, and the portion of the wall parallel to the alignment of the Mountain Loop Highway creates space to construct the bioswale on the north side of the gorge.

Wall 7 is a soldier pile cut wall running approximately parallel to the roadway alignment on the north side of the bridge and on the west side of the road. Wall 7 allows the realigned approach roadway to be constructed. The wall retains a steep forested hillside. Due to the height of the wall and the large surcharge pressures associated with the steepness of the retained slope, the soldier pile wall will need to be anchored with permanent ground anchors.

Temporary geotextile walls, not identified in Table 2, will be needed to retain the fill from spilling onto the existing roadway during construction of the approach fills. These walls will be constructed as the fill proceeds. Towards the end of the project, we propose that traffic be split with the southbound traffic on the new bridge and roadway and the northbound traffic on the existing bridge. During this phase of the project, the remaining west roadway embankment and paving will be completed. The temporary walls will be left in place with the new fill placed against them.

9.0 CONSTRUCTABILITY ANALYSIS

9.1 Summary Description of Alternatives

- Alternative 1: Three-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Precast Concrete Girder Main Span Drop-In

- Alternative 2: Two-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Precast Concrete Girder Main Span Drop-In

- Alternative 3: Two-Span Bridge: Steel Plate Girder

- Alternative 4: Two-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Steel Plate Girder Main Span Drop-In

9.2 Construction Methods and Risks

9.2.1 Construction Access and Staging
Access involves constructing temporary ramps at both ends of the bridge. This will include on-site access and staging areas needed to facilitate the construction of the
project. All bridge alternatives with the exception of the preferred alternative, two-span steel plate girder, require an off-site staging area to splice the girders prior to final delivery to the site. This is needed because of haul restrictions for girders longer than 135 feet.

The on-site staging area is proposed to be on the west side of the road in the approach area adjacent to Pier 1 and will include the existing parking area. This area will be used to store bridge construction materials, connex boxes, and possible field office trailer or trailers. It is anticipated that the steel plate girders will be spliced in this location prior to erection. The area will also serve as an access pad for cranes used for girder erection, service cranes, concrete pump trucks, and concrete deliveries. Staging area on the east side of the river will be limited to a smaller area in the vicinity of Wall 6. This area will be used for limited storage of bridge construction materials.

During the review of bridge alternatives, off-site staging for girder splicing was evaluated by Charlie McCoy of Ott-Sakai for Alternatives 1, 2, and 4. Sites just east of Granite Falls were identified, which will allow for a minimum footprint large enough to accommodate two cranes and the splicing of all of the Span 2 girders. This would allow for a fairly short haul from the splicing yard to the project site. See Ott-Sakai constructability report for further detail on footprint and possible locations (Appendix F.)

The proposed access plan to construct the footings is included in the 30 percent plans located in Appendix B. Access to construct Piers 1 and 2 will be accomplished by cutting in a temporary ramp at an approximate 15 percent grade. To accommodate ramp construction and construction of the bridge piers, the Mountain Loop Highway will be shifted to the east. It is proposed to reduce the speed limit to 25 mph to accommodate the curve radius and length of the temporary shift alignment. To construct the ramp and piers, a temporary shoring wall will be required adjacent to the westbound lane of the Mountain Loop Highway. It is anticipated this will be a drilled soldier pile wall and possibly include tieback anchors, because the cut next to the road will be approximately 20 feet. It is also anticipated that the ramp construction will require both soil excavation and rock excavation. The temporary relocation of the existing water line in the vicinity of this wall will be required because it appears to be in conflict. The proposed relocation is also shown on the 30 percent plans. Access to the fish ladder access road will be adjacent (west) of the temporary ramp. The existing bridge guardrail to the approach guardrail will be further evaluated for modification to a temporary configuration. It is anticipated that the existing guardrail will be attached to a Type 2 concrete barrier at the south approach. The Type 2 barrier will run south for a short distance from the bridge end and taper away from the roadway.

The proposed access plan to Pier 3 will not require a wall next to the existing roadway. A temporary ramp will be constructed to access Pier 3 rock bolting in the vicinity of
Pier 3 and Wall 5 construction. This road will be at an approximate grade of 15 percent and involve both soil and rock excavation. The access ramp and Wall 5 will be constructed concurrent to allow excavation to proceed, because Wall 5 is needed to hold back the hillside during excavation and also for the permanent configuration. Access to the temporary ramp will be approximately 200 feet up station from the end of the bridge. The existing water line may need to be relocated in this vicinity, but we are evaluating if the relocation can be into its final location. The existing bridge guardrail to approach guardrail will be further evaluated for modification to a temporary condition for Phase 1.

9.2.2 Rock Bolt Installation
The 30 percent plans include rock bolting plans for the north and south slopes as discussed in Section 5.6 of this report. Installation will present numerous challenges to the contractor during installation. It is anticipated that a service crane will be needed at both Piers 2 and 3 to raise and lower equipment and materials to the rock face. The drill equipment will need to be fairly compact because it will probably need to be lowered down the face of the bank and tied off while drilling the rock anchor hole. The production rate will be slow based on the logistical challenges of getting labor, equipment, and materials to the drilling location. More research will be conducted regarding equipment and production rates as the design progresses.

9.2.3 Foundation Construction
After the access roads are constructed, the bridge footings will be constructed. However, vertical rock bolts will be required at Piers 2 and 3, which will need to be installed prior to footing construction. The pier footings will be stepped due to the steep slope and Piers 2 and 3 will be in an octagon shape to help reduce the steps and eliminate skew. It is anticipated that rock excavation will be required to ensure the footing will be embedded a minimum of 2 feet into the rock.

9.2.4 Girder Erection
The team has evaluated girder erection for all four alternatives. The preferred alternative, two-span steel plate girder is the only alternative that does not require an off-site splicing yard. Splicing of Span 2 steel girders will be done on the south side of the bridge in the widened parking area. The steel alternative allows for smaller, more manageable sections delivered to the site, which is the lowest risk option for construction. It is anticipated that the Span 2 girder erection will require nightly road closures for one week to facilitate girder erection. It is assumed this operation will occur between the hours of 8 p.m. and 5 a.m. For girder erection concepts, see Ott-Sakai constructability report located in Appendix F.

9.2.5 Existing Bridge Demolition
Bridge demolition will occur late in the project after all traffic is routed onto the new bridge. The conceptual demolition plan is reverse construction from how the bridge was constructed. Access to the existing bridge will be down the existing roadway from both
sides. One constructability and water quality concern is that the final stormwater
treatment facilities cannot be constructed until after the bridge has been demolished.
The bioswales at the north and south bridge approaches lie directly in the access roads
that the bridge demolition equipment will need. This will need to be addressed in the
TESC plan and project provisions.

After access is constructed, the first item of work will be to install containment tarps
and/or platforms under the bridge to ensure that no debris falls into the river. After this
is complete, the contractor will remove the existing utilities from the bridge along with
the guardrail and bridge rail. It is anticipated the utilities can be removed using a lift
truck for personnel access to the hangers and a track hoe or other equipment to lift the
pipe segments onto the existing bridge for transport off the bridge. The bridge rail could
be removed using a track-hoe-mounted muncher with a catch skiff underneath to
contain any concrete debris. The concrete deck could be sawed or broken into smaller
sections and removed from both sides, working back from the middle. Stringers could
be removed as the deck is removed.

After the deck and stringers are removed, the existing concrete approaches can be
demolished using track-hoe-mounted impact breakers. The debris will be loaded into
trucks and hauled off site. It is proposed that the existing concrete be removed to a
depth of 2 feet below grade. The exception to this would be at the main abutments. At
those two locations, it is recommended to break off the pier at the top of footing because
both are located on very steep banks.

After the approach segments are removed, the demolition of the existing steel span can
commence. The proposed concept involves installing anchors at the south and north
ends of the bridge and tying off the arch at U0 and U8. Cranes will be positioned at the
north and south approaches with sufficient boom to reach mid span. The segments are
fairly light, so pick radius will not be an issue. Segments of the truss arch will be
removed from both sides. The segments can be loaded onto trucks on the new bridge or
swung back around and loaded onto trucks near the cranes. The demolition sequence is
included in the 30 percent plans in Appendix B.

9.3 **Approach Roadway Construction**

It is anticipated that the project will involve three phases to construct. Phase 1 includes
initial relocation of utilities, bridge construction, partial wall construction, fish ladder
access road construction, and partial approach construction. Phase 2 involves
constructing the tie-in work from the new alignment to the old alignment. It is
anticipated this will involve a few days of one-lane closures to accommodate grading,
surfacing, and paving at the tie-ins. After that, southbound traffic is switched onto the
new bridge to allow for completion of the approaches and hot-mix asphalt paving. It is
anticipated that this stage will be of short duration. Phase 3 switches all traffic onto the
new bridge, completion of bridge demolition, final construction of drainage treatment
facilities, construction of Wall 4, and project landscaping. See Section 5.1 for details and
recommendations for transitioning the new roadway into the existing roadway and possible issues at the tie in locations. (See the 30 percent plans in Appendix B for conceptual staging plan.)

10.0 ALTERNATIVE COST COMPARISON

10.1 Bridge Segment
The bridge alternatives were compared for relative cost differences. Absolute project construction costs were not investigated for alternative comparison purposes. The relative cost comparison included bridge substructure and superstructure costs including costs for excavation and site access, the cost of the walls bridge demolition, and mobilization. Costs were developed by WSP lumping material and labor costs into unit prices gathered from the WSDOT Bridge Design Manual and recent WSDOT and local agency bridge project bid tabulations, and then also independently by Ott-Sakai following a “contractor style” estimate using crew-based costing, local wage rates, and budget pricing provided by suppliers for concrete supply, concrete girders, girder transportation, erection cranes, and bridge demolition. Any differences in costs were discussed and reconciled amongst WSP and Ott-Sakai. A summary of the relative cost comparison is shown in Table 3.

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<th>Alternative 1</th>
<th>Alternative 2</th>
<th>Alternative 3</th>
<th>Alternative 4</th>
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(1) Includes cost to metallize steel members

10.2 Life-Cycle Cost Analysis
Bridges designed per the AASHTO LRFD Bridge Design Specifications have a design life of 75 years. Concrete bridges are long-term durable structures that do not require painting over their design life. Steel bridges require a protection system. Protection systems typically last on the range of 25 to 30 years for paint systems and 50 years for metalizing systems. Therefore, steel bridges require repainting a couple times throughout the structure’s design life. The cost of repainting, especially at a remote site such as this, can be expensive, and should be considered in evaluating the cost between bridge alternatives. Bridge Alternatives 1 and 2 are concrete structures, and bridge Alternatives 3 and 4 have steel structural components, and thus, bridge Alternatives 1 and 2 would have lower life-cycle costs than bridge Alternatives 3 and 4. Alternative 3 has more steel components than Alterative 4, and thus, Alternative 3 could be considered as having the largest life-cycle costs.

For the bridge alternatives with primary structural steel elements, a method to minimize the life-cycle costs would be to metalize the steel prior to painting the steel members. Metallizing is a common term used to describe thermal sprayed metal coatings. In this
case, it would be for corrosion protection and refer to the thermal spraying of zinc or aluminum alloys as a coating directly onto steel surfaces. The coatings are created by using a heat source to melt the metal, which is then applied via an airstream spray onto the steel surface in a thin film. Once the metal strikes the steel, it resolidifies quickly to become a solid coating. The coating itself provides a barrier between the environment and the steel surface, especially when applied in combination with conventional sealer coatings as topcoats. Due to the electrochemical reaction between the steel and zinc/aluminum, the coatings tend to "sacrifice" themselves to protect the steel at the site of any damage in the coating. This sacrificial protection is similar to the protection provided by zinc-rich primers or galvanizing.

The 50-year service life of metalizing systems, versus 25 to 30 years for painting, is closer to the design life of the structure, thus making the steel structure closer in terms of maintenance to the concrete structure. That said, there is an upfront cost associated with metalizing the steel components. It may be worthwhile comparing the upfront construction costs associated with metalizing the steel components to repainting the steel components a couple of times throughout the life of the structure.

### 11.0 CONSTRUCTION DURATION

A conceptual construction schedule has been prepared for the preferred alternative. The conceptual schedule assumes advertisement in mid-April. Under this assumption, the contractor would mobilize in August. After installation of TESC measures and clearing and grubbing, the existing power will be relocated. Also a portion of the existing water line will be relocated and possibly the Frontier Communications fiber-optic line. Concurrent with this work, it is assumed that steel girder shop drawings will be developed, and fabrication of steel girders will commence immediately after shop drawing approval. A total of 210 working days are allotted for girder shop drawings and girder fabrication. However this item is not on the critical path. Site work will continue concurrent with girder fabrication. The main activity that is critical to the project schedule is excavating rock and installing rock anchors on the north and south slopes at and below Piers 2 and 3. At the 30 percent design level, we have assumed 60 working days for both Piers 2 and 3 to excavate and install rock anchors. The schedule assumes that rock anchor work on the north and south slopes will not be concurrent. Dependent upon site conditions, this duration could increase. After installation of all rock bolts, substructure construction can begin. It is anticipated that girder erection will occur in August. There is a potential risk that the steel delivery date could slide if steel demand ticks upward. However, the schedule for that activity currently contains 50 days of float.

It is anticipated that southbound traffic will be switched to the new bridge (Phase 2) in December and both directions of traffic switched to the new bridge (Phase 3) by January. Demolition access and bridge demolition could commence immediately after the traffic switch with demolition work anticipated to be complete in March 2020. After that, final drainage treatment facilities will be completed along Wall 3, final grading, landscape,
and punch-list work. Physical completion is anticipated in early May 2020. Total project duration from notice to proceed to physical completion is approximately 450 working days. A conceptual construction schedule is included in Appendix E.

12.0 ALTERNATIVE EVALUATION AND PREFERRED ALTERNATIVE

As part of the scope of work on the Granite Falls Bridge No. 102 replacement project, our team started by performing a review of the original TS&L report from November 2012 prepared by AECOM (see Appendix K). The report was reviewed for constructability, cost-effectiveness, and conformance to the latest AASHTO, WSDOT, and Snohomish County EDDS design standards. A preliminary design review memorandum was prepared and submitted to Snohomish County on 26 May (see Appendix H).

A kickoff meeting was held in Snohomish County on 4 March 2016. WSP presented a conceptual three-span precast, prestressed concrete girder option that moved Piers 2 and 3 closer to the top of bank (away from the rock bluffs of the river gorge) from what was shown in the November 2012 TS&L report. The total bridge length presented was 408 feet with Spans 1, 2, and 3 lengths of 83 feet, 260 feet, and 65 feet, respectively. Comparing to the original TS&L report, the design presented use of spread footings and lightweight concrete precast girders in-lieu of drilled shafts and normal weight concrete precast girders. The 260-foot main span included a 195-foot-long standard WSDOT WF83PTG precast concrete drop-in girder segment using sand lightweight aggregate concrete in order to minimize the pick weight of the girder. The end segments consisted of standard WSDOT WF100PTG precast girders with large cast-in-place end diaphragms to avoid needing permanent mechanical hold-down devices at the abutment locations.

Alternative bridge concepts discussed at the 4 March 2016 kickoff meeting included a two-span variant of the presented lightweight precast concrete girder bridge option, and also cast-in-place box girder end spans in-lieu of the normal weight precast concrete girders. Steel plate girders were not desired by the County due to long-term maintenance concerns. Thus, the bridge alternatives carried forward for further evaluation were the following.

- Three-span bridge: normal weight precast, prestressed concrete girder end spans with lightweight precast prestressed concrete girder main span drop-in
- Three-span bridge: cast-in-place box girder end spans with a lightweight precast, prestressed concrete girder main span drop-in
- Two-span bridge: normal weight precast prestressed concrete girder end span with lightweight precast, prestressed concrete girder main span drop-in
- Two-span bridge: cast-in-place box girder end span with a lightweight precast, prestressed concrete girder main span drop-in
WSP met with Snohomish County on 8 November 2016 to discuss the concrete bridge options developed after the kickoff meeting. Discussions focused on the findings from the bridge study. Notable findings included

- Large cranes were required on the completed end spans of the bridge to erect the main span drop-in girders. This required a portion of the bridge deck to be constructed ahead of placement of the main span girders, which raised concern with rideability of completed bridge structure. In addition, the bridge end spans had to be overdesigned to accommodate a temporary construction condition.

- The required length of the precast, prestressed concrete girders required an off-site splicing yard due to the remote location of the project site. The off-site splicing yard required finding a site within close proximity of the project site to splice girders, having cranes to off-load and load girder segments at the splicing yard, and a splicing yard long enough to post-tension girder segments together.

- In-span splicing of precast girders required experienced general contractor. Not necessarily conducive in a low bid environment.

- Elaborate camber control and sequence of construction required with lightweight post-tensioned precast, prestressed concrete girders.

- All options required falsework or shore towers duration construction.

- Alternatives require transport of long precast elements from the splicing yard to the project site; especially the two-span alternatives.

For more detailed information, referenced should be made to Appendix F.

After the 8 November 2016 meeting, Snohomish County requested a screening of steel plate girder options to determine the feasibility of steel plate girder bridge alternatives relative to the concrete bridge alternatives originally investigated (initiated in Supplement No. 5). The following steel plate girder options were screened based on the relative superstructure constructability, schedule impacts, construction costs, and inspection/maintenance.

- Two-span steel plate girder
- Three-span steel plate girder
- Single-span steel plate girder
- Two-span bridge: cast-in-place box girder end spans with a steel plate girder main span drop-in

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The screening process included like assumptions for the bridge span layout and the substructure/foundation elements. Based on the screening analysis, the three-span steel plate girder option and single-span steel plate girder options were dropped because they either required the girders to be curved (three-span alternative), thus erected in pairs, which required large cranes to erect, or required deeper heavier girders (single-span alternative), and an elaborate bridge deck system with transverse post-tensioning.

A similar assessment was conducted on the four concrete bridge alternatives considered. All the precast, prestressed concrete girder options were eliminated based on the relative construction risks/specialization compared to the precast, prestressed concrete girder options with cast-in-place box girder end spans. Thus, the resulting four final alternatives for evaluation were as follows.

- Alternative 1: Three-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Precast Concrete Girder Main Span Drop-In
- Alternative 2: Two-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Precast Concrete Girder Main Span Drop-In
- Alternative 3: Two-Span Bridge: Steel Plate Girder
- Alternative 4: Two-Span Bridge: Cast-in-Place Post-Tensioned Box Girder End Spans with Steel Plate Girder Main Span Drop-In

These are the four alternatives presented in Section 7.0 of this report and are the four alternatives presented to the County on 10 February 2017. Following the presentation to the County, the two-span steel plate girder alternative (Alternative 3) was selected as the preferred alternative.

13.0 RECOMMENDATIONS

The engineer’s recommendation is Alternative 3, the two-span steel plate girder alternative. Alternative 3 was chosen based on minimizing construction risk and simplifying construction. Alternative 3 uses only one structure type and follows conventional construction practices.