# Sterling Highway MP 45–60
# *Preliminary Bridge Structures Technical Report*

STERLING
HIGHWAY MILE POST 45 TO 60
ALASKA

***Prepared for:***

**State of Alaska**
**Department of Transportation and Public Facilities**

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# 1.0 INTRODUCTION

## 1.1 Background

The Sterling Highway Milepost 45 to 60 Supplemental Draft Environmental Impact Statement (SDEIS) process is being conducted by the Alaska Department of Transportation and Public Facilities (DOT&PF). The purpose of the SDEIS is to analyze and evaluate reasonable alternatives to address traffic flow, seasonal congestion, local access issues and roadway deficiencies on the section of the Sterling Highway that parallels Kenai Lake and the Kenai River through the community of Cooper Landing.

## 1.2 Scope

A number of alternatives have been evaluated for analysis in the SDEIS, and these have been narrowed down to four build alternatives as discussed in the Preliminary Engineering Report (HDR 2011). The four build alternatives are: Juneau Creek, Juneau Creek Variant, G South, and Cooper Creek alternatives. This memorandum discusses feasible bridge options and span configurations for each crossing, constructability issues, and the advantages and disadvantages of each option, followed by a brief discussion on the costs and recommendations for which options should be further studied.

## 1.3 Site Conditions

The Sterling Highway Milepost 45 to 60 project is in the vicinity of the community of Cooper Landing in the Kenai Peninsula Borough. The project is approximately 100 miles south of Anchorage on Highway 1 and is partially within the Kenai National Wildlife Refuge and the Chugach National Forest. The area includes several popular State and Federal campgrounds, recreation sites, and trails, including the Resurrection Pass National Recreation Trail (RPNRT), the Russian River Campground, and the Kenai River Special Management Area.

The highway corridor started as a mining trail in the early 1900s. The trail later evolved into a one-lane road to support homesteading activities, with the first bridge over the Kenai in the Cooper Landing area completed in 1921. Automobile travel west to Cooper Landing was possible by 1937. Rapid development and growth of the western Kenai Peninsula spurred road improvements in the 1960s and included construction of the Kenai River crossings at Schooner's Bend and Cooper Landing. Since then, upgrading has been limited in the project area.

## 1.4 Geography and Geology

According to the Affected Environment Technical Memorandum (HDR 2001), major mountain building and uplifting created the Kenai Mountains that rise above the Kenai lowlands. Much of the Kenai Peninsula still is predisposed to earthquakes in the range of 6.0 to 8.8 magnitude on the Richter scale, with a predicted 75-year recurrence interval for magnitude 7.3 earthquakes (pga ~ 0.46g). There are many small, although inactive, faults in the project area, including the Border Ranges Fault system located approximately 5 miles west of Cooper Landing.

The topography evident today was shaped within the last 100,000 years by periods of glaciation. Glaciers covered the entire project area as recently as 5,000 to 10,000 years ago. The project area is located in a deep glacial valley that trends east to west through the Kenai Mountains. Tributary valleys enter from the north (Juneau Creek) and from the south (Russian River and Cooper Creek). The terrain varies from steep and mountainous to level benches bordered by steep side slopes above the floodplain of the Kenai River to flat river bottom.

### 1.5 Meteorology

The average annual precipitation for the area is approximately 22 inches. Average annual temperature is 46 degrees Fahrenheit, with a mean January temperature of 26 degrees Fahrenheit and a mean July temperature of 68 degrees Fahrenheit. The highest temperature recorded in the past 30 years was 90 degrees Fahrenheit and the coldest was -40 degrees Fahrenheit (Western Regional Climate Center 2004).

## 2.0 Alignments

There are four build alternatives that resulted from the preliminary engineering and alternatives evaluation process: Juneau Creek, Juneau Creek Variant, G South, and Cooper Creek alternatives. Each alternative has at least one major crossing that requires a bridge. Table 1 presents the feasible bridge options at each major crossing location for the four build alternatives and a short discussion of each option follows. Appendix A contains conceptual drawings of all options. Figure 1 shows the build alternatives and crossing locations.

**Table 1: Preliminary Bridge Structure Options**

| Alternative | Crossing Location | Feasible Bridge Options |
|---|---|---|
| Juneau Creek / Juneau Creek Variant | Juneau Creek | • Steel Tied Arch<br>• Cable Stay (Asymmetric)<br>• Simple Span Steel Truss<br>• Continuous Cantilever Steel Truss<br>• Segmental Concrete Box Girder<br>• Segmental Concrete Box Girder (450 ft)<br>• Deck Arch w/ Prestressed Concrete Girders |
| G South | Juneau Creek | • Prestressed Concrete Girder w/ Cast in Place Pier Sections<br>• Deck Arch w/ Prestressed Concrete Girders<br>• Multiple Deck Arch |
| G South | Kenai River | • Prestressed Concrete Girder<br>• Steel Plate Girder |
| Cooper Creek | Cooper Creek | • Chorded Prestressed Concrete Girder<br>• Curved Steel Plate Girder |
| Cooper Creek | Kenai River at Cooper Landing | • Prestressed Concrete Girder<br>• Steel Plate Girder |
| Cooper Creek/ G South | Kenai River at Schooner's Bend | • Prestressed Concrete Girder<br>• Steel Plate Girder |

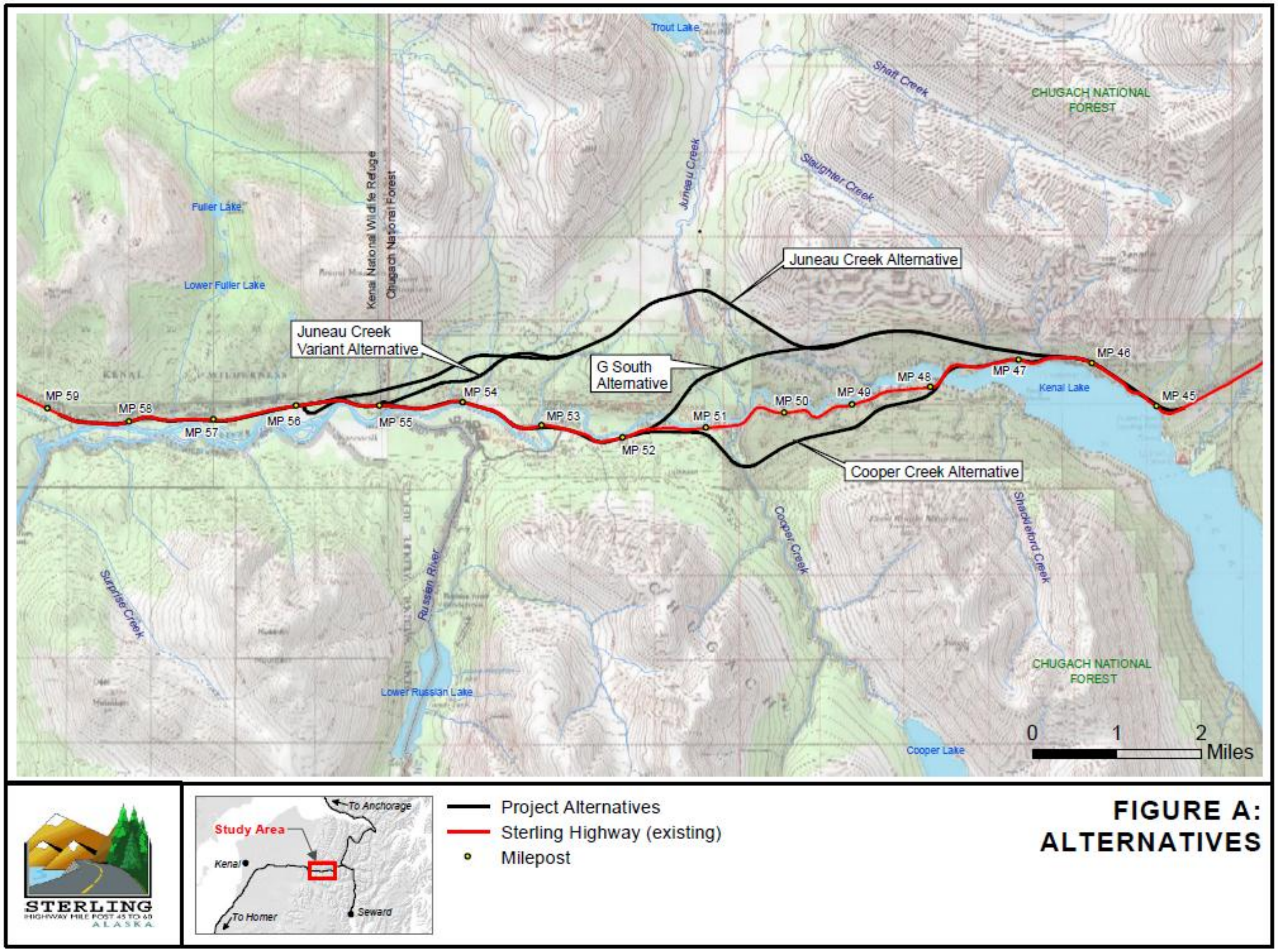


**Figure 1: Vicinity Map**

## 2.1 Juneau Creek Alternative / Juneau Creek Variant Alternative

Both the Juneau Creek and Juneau Creek Variant alternatives include a major crossing of Juneau Creek. This section of the report discusses structure options that would be equally applicable to either the Juneau Creek or Juneau Creek Variant alternatives.

Conceptual crossing options for the Juneau Creek crossing were completed in April 2004; however, a field reconnaissance trip in October 2004 revealed a very recent landslide in the Juneau Creek Canyon near the original bridge crossing location. A field investigation was conducted in the vicinity of this crossing site to investigate geotechnical concerns. The study involved visual inspection of the geology of the project area by engineering geologists working for R&M Consultants, who documented rock structure characteristics both near the Juneau Creek F alignment and along the canyon in search of more favorable alignment alternatives. The investigation was strictly surficial; no subsurface data were collected. The observations from the investigation were compiled into the Preliminary Rock Stability Investigation report (R&M 2005). The initial bridge crossing location was situated to avoid direct impacts to a Federal recreational withdrawal that encompasses the upper Juneau Creek Canyon and falls. Based on the results and recommendations from the Preliminary Rock Stability Investigation, the alignment was relocated to cross the canyon at a right angle, approximately 600 feet north of the original crossing location where more competent rock was observed.

### *2.1.1 Juneau Creek Crossing*

The proposed crossing at Juneau Creek is located between station 1627+00 and 1645+00 and consists of a cross section that will accommodate two 12-foot lanes, two 8-foot shoulders, one 12-foot east-bound climbing lane, and a 6-foot pathway. The overall width of the structure will be 62 feet, including bridge railings.

Based on the conceptual alignment and profile for this alternative, Juneau Creek is approximately 230 feet below the canyon rim and approximately 425 feet from rim to rim of the canyon at the crossing. The "rim," for the purposes of this report, is the point at which the existing topography takes a marked break in slope from approximately 10:1 (horizontal:vertical) to 1:1. The location of the west rim is near station 1633+85 and elevation 1,090 feet. The location of the east rim is near station 1638+15 and elevation 1,080 feet. In the interest of minimizing impacts within the canyon while still allowing room for excavation and foundation of substructure elements, elevation 1,060 feet has been established as an appropriate lower bound elevation for any permanent substructure within the canyon.

The preliminary geotechnical investigation discussed above revealed large cracks running parallel to the canyon up to 300 feet from the rim in some locations, but especially in the lower canyon. The instability caused by the cracks creates the potential for a large material slide from the crack to Juneau Creek, however the risk of rock instability decreases further up the canyon where the proposed Juneau Creek bridge will cross, therefore, it was recommended that the ends of the bridges be located no closer to the canyon rim than 200 feet, giving a bridge main span of 825 feet. This setback value of 200 feet from the canyon rim is preliminary, based on the information available and the level of detail of this study. This setback should be validated during subsequent design phases to ensure that bridge supports are located in competent founding materials. The setback will also allow for an aerial crossing of the RPNRT located on the west side of the Juneau Creek canyon. If the

Juneau Creek or Juneau Creek Variant alternative is selected for further study and design effort, additional rock stability and geotechnical studies should be performed to refine the required span length of the structure and foundation conditions.

Most of the superstructure options under study for the Juneau Creek crossing will bridge over the entire 825-foot span, between areas of anticipated competent material on each side. Two additional options were considered that would be designed for a shorter main span, founded near the rims of the canyon, resulting in a 450-foot main span with approach spans to tie into the proposed alignment. Because further geotechnical investigation will be required for developing the conceptual design through final design, the shorter span options will be studied in the event that the results of further geotechnical study show that founding closer to or on the canyon rims may be viable. Regardless, abutments and piers would not be located below the canyon rim at elevation 1,060 feet and no in-water structure or work is proposed.

While no bridge in Alaska has spans in the magnitude of 825 feet, traditional and proven techniques can be utilized to construct a bridge of this span length. Bridge types normally used for an 825-foot span length include steel tied arch, cable stayed, steel truss, or post-tensioned concrete box girders. Bridge options under consideration for the shorter span bridge (450-foot) are a concrete deck arch and a post-tensioned concrete box (referred also as segmental concrete box). Further discussion of these bridge options is provided below. The common element of all the bridge options listed for this crossing is the ability for construction to occur without impacting the walls or floor beneath the canyon rim. While the end results can appear quite different from one structure type to the next, there are generally two primary techniques that are used to erect the span without impacts to the site between abutments: balanced cantilever construction and support mast construction.

*Balanced Cantilever Construction (Figure 2):* In balanced cantilever construction, the main span is constructed outwardly from a primary pier in discrete segments, with a back span constructed simultaneously on the oppose side of the pier so that the forces counteract for minimal bending moment imposed upon the pier. In the case of concrete, typically a main span segment is cast through the use of formwork carriers, and while that segment is in the curing process a back span segment is cast to match. As can be seen in Figure 2, construction occurs from the in-place superstructure (consistent for both concrete and steel bridges), which allows the erection to progress without intermediate piers or falsework. This effort occurs on both sides of the feature to be bridged, and continues until the main span lengths meet in the middle of the span. The end result is a main span that can be bridged without impacts to the ground beneath, with additional back spans on both approaches that effectively double the total length of structure needed to cross the main span alone.

The Juneau Creek bridge options that use cantilever construction are the post-tensioned concrete box girder and the cantilever steel truss bridge.

**Figure 2: Balanced Cantilever Construction**
**Metsovitikos Bridge, Greece**

Note: Although construction access roads are depicted in this photograph, such roads into the canyon would not be necessary for balanced cantilever bridge options discussed in this report.
(http://i208.photobucket.com/albums/bb294/corfu_album/E10-07155.jpg)

*Support Mast Construction (Figure 3):* Support mast construction utilizes tall towers located at or near the abutments for supporting the bridge superstructure from cables during construction, shown in Figure 3. Enough support is provided from the cables to allow construction to progress from the structure itself without the need for intermediate piers or falsework. Cables can be strung between towers for conveying overhead cranes, which are used to deliver materials, equipment, and personnel to the active spans. The bridge span is constructed from both ends until the superstructure is connected in the middle of the span. In some cases the masts are permanent features of the finished structure, as in cable stayed bridges. In other situations, such as deck arch, tied arch, or simple steel truss construction, the masts are in place only until completion of the main span, at which point the towers and cables are removed.

Whereas the balanced cantilever technique can result in additional construction cost with back spans that are not imperative for a given alignment, the premium cost for the support mast technique arises out of the erection and dismantling of temporary structural elements. However, even with the additional cost, this construction technique is often the most feasible method of construction for a long-span bridge, especially in situations with difficult access where additional piers and falsework become impractical.

The bridge options for crossing Juneau Creek that use support mast construction are the tied arch, cable stayed, steel truss, and deck arch bridge.

**Figure 3: Support Mast Construction**
**Mike O'Callaghan - Pat Tillman Memorial Bridge**

(Photo courtesy of HDR)

### *2.1.1.1 Option 1: Steel Tied Arch Option*

This structure option is comprised of twin steel arch ribs connected with tension tie girders between the ends of the arch span as shown on sheet A-1. The tie girders also function as edge beams. A floor beam system is connected to the edge beams on which either an orthotropic steel or composite reinforced concrete deck is placed. The arch has the capability of being constructed with minimal to no access below the canyon rim, through the support mast technique described above. While the structure in its completed condition would effectively span the RPNRT corridor on the west side of the canyon, activities above the rim would affect the trail and its use during construction. Activities that could affect trail usage include equipment movement and staging, clearing and grubbing, and temporary falsework. The impact area outside of the typical construction easement for the highway would not be extensive, as the foundations are situated on relatively level ground and would not require substantial excavations and resulting access roads to be built. Access by recreationalists around or through the project area could likely be maintained throughout construction, though a trail detour may be required during construction.

Due to the lack of redundancy in the primary steel tension members, the Federal Highway Administration (FHWA) classifies steel tied arch bridges as 'fracture critical' structures. The implication of such a classification is a greater degree of inspection scrutiny throughout the life of the structure to identify any structural distress prior to what could be catastrophic failure. Fracture critical bridges require a 'hands on' inspection on a biennial frequency in which the fracture critical members, in their entirety, are viewed from within an arm's length. Additionally, Non-Destructive Testing (NDT) is often required as a second level screening when cracks or other problems are detected in critical bridge members. This can represent a significant expense, especially in situations such as the Juneau Creek crossing, where the primary members cannot be reached from the ground or from typical bridge inspection equipment (i.e., "snoopers"). A common scenario is for a team of trained rope access technicians to accompany the team of bridge inspectors. From mobilization of such a crew to the field inspection and subsequent inspection documentation, the costs associated with one inspection could be in the range of $30,000 to $40,000. Additional costs could be expected for properly maintaining the structure based upon the observations from the routine inspections. The classification as a fracture critical structure (and the associated costs) should be considered when assessing the steel tied arch bridge as a feasible superstructure option.

This bridge option is extremely stable once constructed but remains vulnerable during construction. Construction could last up to five years, during which the arch would need to be laterally braced, but once complete, the arch configuration would smoothly integrate into the surrounding environment, creating an aesthetically pleasing design. In addition, the tied arch option has the minimum structure length (825 feet for spanning the canyon plus setbacks) required for this particular highway alignment.

#### *2.1.1.2 Option 2: Asymmetric Cable Stay Option*

For the Juneau Creek crossing, the cable stay option has a span configuration consisting of two 68-foot anchor blocks, two 125-foot back spans, and an 825-foot main span as shown on sheet A-2. As the name indicates, cable stays attached to pylons at the abutments support longitudinal steel or concrete edge girders which in turn support a transverse floor beam system with an orthotropic steel or composite reinforced concrete deck. The cable stays balance the gravity loads from the deck around the pylons so that the stays on one side of the pylon are supporting the same gravity loads as the stays on the other side of the pylon. This is traditionally accomplished by making the deck symmetric around the pylon; however, the back spans must then be equal to half the main span, increasing the overall length of the bridge. Although a symmetric cable stay would be a viable and more conventional option for the Juneau Creek crossing, the steep canyon walls, rock stability issues, and below-canyon access restrictions dictate the pylon locations and require a main span length of

825 feet. For a symmetric cable stay the overall bridge length would need to be 1,650 feet. To save approximately 450 feet of excess deck length and to avoid substantial issues with the vertical and horizontal alignment, an asymmetric cable stay arrangement was considered in which the main span gravity loads are counterbalanced using large concrete blocks at the ends of the bridge, producing the span arrangement discussed previously. Although the cable stay bridge would not be classified as a fracture critical structure according to FHWA's criteria, the inspection effort and maintenance concerns associated with cable stay bridges would still represent significant costs.

The impacts to the RPNRT are comparable to those caused by the steel tied arch. Specifically, equipment movement and staging, clearing operations, and other activities during the construction effort would affect a footprint immediately around the highway construction easement. As with the steel tied arch, recreational access around or through the project area could likely be maintained using a trail detour during construction. The completed structure would span the trail corridor and not impede trail usage.

The long period and relatively light superstructure of the cable stay is beneficial when considering seismic design loads but makes it vulnerable to high winds; therefore, on top of the normal seismic studies, wind studies are recommended for this structure type beyond what is commonly done for wind loading on the other bridge options considered.

One of the main advantages to a cable stay bridge is that it would be constructed using a cantilever method and thus would not require access to below the canyon rim or temporary shoring during construction. The disadvantages would be the typically increased operation and maintenance costs associated with maintaining the stays and the slightly longer span length than the tied arch and simple span truss options that would encroach into a horizontal curve in the alignment. Although the bridge could accommodate the horizontal curve by widening the deck, it would probably be more economical to realign the highway near the bridge. The realignment that the cable stay bridge option requires is minor compared to the continuous span steel truss or box girder options.

#### *2.1.1.3 Option 3: Simple Steel Truss Option*

Steel trusses can be classified as deck trusses or through trusses, depending on whether the majority of the structure is below or above the roadway. This particular alignment is most conducive to using a through truss because of the limited space below the bridge deck near the abutments. Theoretically a simple span truss with a span of 825 feet (as shown on sheet A-3) is possible, though it would be the longest of its kind in the world (current longest span is 745 feet in Chester, West Virginia) and would be difficult to fabricate and erect due to the size and number of strut members needed. The simple steel truss is considered impractical for this crossing.

#### *2.1.1.4 Option 4: Continuous Cantilever Steel Truss Option*

The other steel truss option for this 825-foot crossing would be a continuous cantilevered truss which would allow the truss to be constructed from the piers and limit the construction access needed below the canyon rim. The span configuration for a continuous truss would consist of two 410-foot back spans and an 825-foot main span as shown on sheet A-4. Although the span length is well within the limits of this type of bridge, it requires a balancing of the spans and thus effectively doubles the total length of the structure needed. To maintain the existing preliminary alignment the end of the bridge would project into the side of the mountain and would require some excavation to produce the required length. Furthermore, additional excavation would be required to allow passage of the RPNRT beneath the structure. The profile could also be raised but trying to accommodate the bridge by changing the highway geometry is contrary to normal convention and counterproductive in most cases. Also, the continuous cantilever steel truss would be classified as a fracture critical structure, and would thus require the aggressive inspection treatment discussed under the description of the steel tied arch option. As with the simple span truss option, although the continuous cantilever option would be viable, it would not present the most practical solution for this project due to the reasons discussed above and is considered impractical for this crossing.

#### *2.1.1.5 Option 5: Post-tensioned Concrete Box Girder Option*

Like the continuous span truss option, the span configuration for the post-tensioned concrete box girder consists of two 410-foot back spans and an 825-foot main span, shown on sheet A-5. As the name indicates, this type of bridge consists of a precast or cast-in-place concrete box that is longitudinally post-tensioned with high-strength cables to derive its spanning capabilities. Cantilever arms extend from the top slab of the box to provide the required roadway width. The span length is within the limits of this type of structure but may be toward the upper limits. In fact, the longest post-tensioned concrete box girder in the United States is the Kanawha River Bridge located in Kanawha County, West Virginia, with a length of 760 feet. The length and location of this proposed option would likely come at a premium cost. A discussion of the associated costs is found in Section 3.0.

Common construction techniques for post-tensioned concrete box girder bridges allow for overhead construction (both the balanced cantilever and support mast method apply) which makes them well suited for crossings with limited access or crossings over environmentally sensitive areas such as that at Juneau Creek. Because of the construction method, this option requires a bridge that is effectively twice as long as the minimum span length that could be constructed. Similar to the continuous cantilever steel truss, the increased structure length would require extensive earth removal or raising the highway profile. Neither of these changes is desirable, since the highway and/or terrain geometry would have to be changed to accommodate the bridge rather than vice versa, which is the normal convention. This option is considered impractical for this crossing.

#### *2.1.1.6 Option 6: Post-tensioned Concrete Box Girder Option (450-Foot Main Span)*

Although the post-tensioned concrete box girder solution was deemed impractical for the 825-foot span configuration (see Option 5), it may provide a more reasonable solution if the primary towers were to be moved nearer the canyon rim for the shorter clear span of 450 feet. It is important to note that this solution does not provide for a 200-foot setback from the canyon rim, and is therefore potentially more exposed to significant geotechnical failure than the longer-span options. Because of the serious concern for stability of the soil/rock comprising the canyon walls, a comprehensive geotechnical investigation would be necessary before viability of this option can be confirmed. Piers would be built at or above elevation 1,060 feet on either rim of the canyon. The concrete box girder segments could be constructed from the piers and limit the construction access needed below the canyon rim. The span configuration would consist of two 225-foot back spans and a main span of 450 feet, for an overall structure length of 900 feet. Although the depth of the structure may necessitate some additional excavation behind the piers to accommodate construction efforts, the length would not require realigning the road geometry to match the bridge. Also, the back span on the west side of the crossing would provide a route for the RPNRT to pass beneath the finished highway.

Construction roads would be necessary for conducting the excavation and erection effort for the piers. Because additional excavation may be necessary behind the piers to accommodate the back spans, the roads would likely pose little or no permanent impact on the landscape, though these activities would temporarily impact the RPNRT on the west side of the canyon. Trail users may have to use a trail detour during construction. The completed structure would accommodate the trail and allow unrestricted passage underneath. And as discussed, the location and elevation of the permanent substructure elements were selected so there would be no impacts to the walls or floor of the canyon.

A similar configuration could be achieved through the use of a continuous cantilever steel truss bridge, which would also be built out from the primary support towers. However, the steel trusses would represent fracture critical elements, and the fabrication and erection of the structure would be less efficient and less economical than the concrete box girder at such a configuration. Likewise, a simple steel truss option would also be feasible for the 450-foot clear span (eliminating the need for the back spans), though it would be less efficient than the cantilever option. The extensive temporary construction associated with erecting such a structure without impacting the canyon floor would much more extensive than other options, and the finished structure would require either additional approach spans or extensive fill behind the abutments for tying into the proposed road profile. The post-tensioned concrete box girder method represents a more effective solution for crossing Juneau Creek on this alignment. See sheet A-6 for a depiction of the crossing option.

#### *2.1.1.7 Option 7: Concrete Deck Arch with Prestressed Girder Approaches Option*

The deck arch option utilizes a 450-foot main span with two 95-foot approach spans on either end for an overall structure length of 830 feet, as shown on sheet A-7. It is important to note that this solution does not provide for a 200-foot setback from the canyon rim, and is therefore potentially more exposed to significant geotechnical failure than the longer-span options. Because of the serious concern for stability of the soil/rock comprising the canyon walls, a comprehensive geotechnical investigation would be necessary before viability of this option can be confirmed. The arch solution transfers gravity loads into lateral forces acting on the walls of the canyon through large thrust blocks. The 450-foot main span was selected to achieve a structurally efficient balance between span length and depth of the arch, while also locating the towers just within the upper limits of the canyon walls to take advantage of the lateral resistance of the soil. As such, construction activities would need to occur approximately 30 feet below the canyon rim (to elevation 1,060 feet) to found the primary towers and thrust blocks into competent bearing material. The impact to the canyon floor would be minimal, as the superstructure can be constructed from either end to be joined in the middle, but unlike other longer span options which have the intended effect of spanning beyond potentially unstable soils near the canyon rim, significant foundation work would occur within the top 30 feet of the canyon.

The proximity of the towers to the canyon rims and the depth of the substructures would require more extensive excavation than the longer-span options described above. As mentioned, the foundations for the deck arch would be located at or above elevation 1,060 feet. Because the respective elevations of the east and west canyon rims are approximately 1,080 feet and 1,090 feet, the cuts would exceed 20 to 30 feet to reach elevation 1,060 feet. Assuming 10 percent (maximum) grades for the construction access roads, the length of access roads required to reach the bottom of the excavations would be 200 to 300 feet at a minimum (ultimately longer in order to accommodate the sloping topography above the canyon rim). Because additional excavation may already occur behind the foundations there may be room for the access roads to descend along the highway alignment toward the deck arch structure, but it is likely the roads would require additional space north or south of the alignment to achieve the necessary length for the grade. Also, depending upon the exact rise of the arch at the crown, the road profile may need to be raised to accommodate the structure height.

Precast, prestressed concrete girders would be used for the approach spans. The precast girders could be fabricated while the substructure is being built to accelerate the construction effort, with further economy being realized in maintaining consistent span lengths. The approach span piers would be constructed of concrete, and would all be located above the canyon rim. The spans could be erected from the ground within the highway alignment. Similarly to the longer span options previously discussed, the elevated approach spans on the west side of the canyon would allow for aerial crossing of the RPNRT when finished. The temporary impacts to the RPNRT would be comparable to those discussed under Option 6. Although steep side slopes would not be a factor in the intermediate pier construction, the potential for instability in the soil near the canyon rims would warrant further geotechnical investigation prior to selection of the deck arch as a preferred alternative.

One advantage of this option is that the reduced main span length (450 feet versus 825 feet for other structures) could be built with a cross section that is materially more efficient. The arch may offer greater visual aesthetic appeal. Consistent approach span design could also result in project cost savings. In addition, this structure would not be considered fracture critical. Disadvantages include a complicated structure design, a long construction timeframe, construction impacts within the rim of the canyon, and potentially unsuitable soil conditions. As discussed, the critical caveat for this option to be viable for further consideration, as with the post-tensioned concrete box girder option, is that a geotechnical investigation reveals acceptable conditions for placement of substructure adjacent to the canyon rim.

### *2.1.2 Sportsman's Landing Grade Separation*

The Juneau Creek Variant Alternative includes a loop ramp for connecting the existing Sterling Highway to the new alignment. A bridge structure with a span of approximately 140 feet would be necessary for conveying the new alignment over the access ramp. Because such loop ramp structures are relatively common on the Alaska Highway system, specific drawing sheets have not been prepared for this report.

#### *2.1.2.1 Option 1: Prestressed Concrete I-Girders*

For the Sportsman's Landing crossing, the concrete bridge option would consist of prestressed concrete decked bulb-tee girders in a single span arrangement. Such girders are commonly used for spans less than 150 feet, and offer the advantages of low maintenance costs and local materials and labor.

#### *2.1.2.2 Option 2: Steel Plate Girders*

Although certainly feasible for a crossing of this nature, steel girders typically have higher design, fabrication, and material costs than precast concrete girders in short-span situations (up to approximately 150 feet), and require more maintenance after installation. The benefits of steel plate girders are often realized in longer span situations that are no longer practical for concrete construction. The relative simplicity and economy of concrete girders makes the use of steel plate girders a less practical choice for the Sportsman's Landing grade separation.

## 2.2 G South Alternative

The G South Alternative has two new major river crossings and one replacement crossing along the alignment. They include new crossings over the Kenai River between stations 1616+40 and 1621+26 and at Juneau Creek between stations 1656+77 and 1670+03. One bridge will be replaced over the Kenai River at Schooner's Bend. The alignment for Cooper Creek is coincident with the G South Alternative at this location; the Schooner's Bend crossing is discussed in Section 2.3.3.

### *2.2.1 Kenai River Crossing*

The proposed structure at this location consists of a cross section that will accommodate two 12-foot lanes, one 12-foot climbing lane, one 16-foot center turn lane, two 8-foot shoulders, and one 6-foot walkway on the upstream side. The overall width of the structure will be 78 feet, including bridge railings.

The relatively short spans for this crossing are conducive to using either prestressed concrete girders or steel plate girders. The bridge will be constructed on new alignment; therefore there are no traffic maintenance issues to contend with during construction; however, a work bridge will be necessary.

#### *2.2.1.1 Option 1: Prestressed Concrete I-Girder and Deck Bulb-Tee Girder*

For the Kenai River crossing, the concrete bridge option would consist of four 134-foot spans using seven prestressed concrete bulb-tee girders with a cast-in place concrete deck, 16 deck bulb-tee girders, or a similar girder arrangement, as shown on sheet A-8. The final girder configuration for the bridge would be determined through a comprehensive structural design based on bridge geometry, live loading and transportation constraints (the short span concrete options discussed in the Cooper Creek Alternative may also be subject to variability in the girder arrangement upon further structural design). The anticipated unit costs of alternative concrete girder arrangements would remain similar to the unit costs presented in Section 3.1.1. The advantages of this arrangement over steel would be lower maintenance costs and the ability to use more of the local materials and labor.

#### *2.2.1.2 Option 2: Steel Plate Girder*

The steel bridge option for this crossing would consist of seven steel plate girders supporting a cast-in-place concrete deck with 152-foot approach spans and a 182-foot main span as shown on sheet A-9. The advantage to this arrangement over concrete is the reduction of the number of intermediate piers needed from four to three.

### *2.2.2 Juneau Creek Crossing*

The proposed structure at this location consists of a cross section that will accommodate two 12-foot lanes, two 8-foot shoulders, one 12-foot climbing lane, and one 6-foot pathway. The overall width of the structure will be 62 feet, including bridge railings.

This crossing is necessary where the proposed alignment crosses Juneau Creek between stations 1656+77 and 1670+03. The proposed alignment runs nearly perpendicular to the east canyon wall and intersects the west canyon wall where the canyon begins to flare out and open up to the Kenai River Valley. At its highest point, the alignment is approximately 200 feet above the creek. With structure heights of this magnitude, it is economical to create longer spans in order to reduce the number of piers which can become quite massive due to their height. Three span arrangements were considered at this crossing including using prestressed girders reinforced near the piers, a deck arch with prestressed concrete girder approach spans, and a series of deck arches to span the canyon below.

There are consistent constructability issues between this crossing of Juneau Creek and the crossing proposed under the Juneau Creek / Juneau Creek Variant alternatives. There are no existing access roads to the project site, and the crossing occurs through environmentally sensitive terrain, both factors contributing to the need for efficient and unobtrusive construction techniques. The G South Alternative avoids impacts to the RPNRT, so trail detours, delays, or closures would not be necessary for any of the proposed crossings.

#### *2.2.2.1 Option 1: Cast in Place Piers with Prestressed Girders*

One option considered for spanning Juneau Creek involves a cast-in-place concrete deck supported by precast prestressed concrete girders with a span configuration of 120’-120’-120’-160’-240’-240’-160’-80’-80’ as shown on sheet A-10. However, the precast girders are not capable of spanning more than 120 feet in their current design. To accommodate the longer spans of 240 feet and 160 feet, the bottom flange of the girders would need to be thickened to create a haunch at the piers, effectively increasing the spanning capabilities of the precast girders. The advantage of this option is that precast members can be fabricated while the substructure is being built to accelerate construction while still reducing the number of intermediate piers needed. The disadvantage is that access roads would be needed in the canyon wall to get the precast members to the base of the foundations for placement. The roads would be banked into the walls and would require either long approaches from the north or south, switchbacks into the slopes, or both. If a 10 percent grade is assumed for construction vehicle access, their length would be approximately 2,000 feet to descend from the rim to the canyon floor.

A steel girder with a cast-in place concrete deck or segmental concrete box girder superstructures was also considered for this span arrangement. The steel girder option has similar speed of construction advantages as the prestressed girders but with higher initial and maintenance costs. The segmental concrete box girder arrangement, which is similar to that shown on sheet A-5, would be as durable as the prestressed girders but with a longer expected construction time.

#### *2.2.2.2 Option 2: Concrete Deck Arch with Prestressed Girder Approaches*

Instead of strengthening precast girders, another option considered would be to use a concrete deck arch to span 480 feet across the canyon floor as shown on sheet A-11. Prestressed concrete girders would still be used for the approach spans, but they would be limited to no more than 110 feet. The locations of the substructure elements would require access roads for transporting the girders to the job site. Although the number of intermediate piers would be the same as the previous option, the size of the approach span towers would be reduced because of the shorter span lengths. A deck arch gains its structural efficiency by being located where the lateral forces created from the gravity loads applied to the arch are easily transferred to the canyon walls through large thrust blocks. Since this configuration places the thrust blocks near the center of the canyon floor, the lateral forces would need to be resisted by using multiple large piles. The increase in foundation costs needed for the thrust blocks in addition to the complicated nature of arch construction would most likely offset the cost savings from the approach span towers. A detailed analysis of both options would be needed if the G South Alternative was chosen.

#### *2.2.2.3 Option 3: Three Consecutive Deck Arches*

A final option considered was to use three consecutive deck arches with span lengths of 410 feet, 480 feet, and 320 feet to span the Juneau Creek Canyon as shown on sheet A-12. With only three large thrust blocks and two abutments to support the deck arches, this option would have the least permanent impact to the canyon floor and be the most aesthetically pleasing design out of the three options considered. The construction effort for the thrust blocks within the canyon would require the mobilization of piles, concrete, steel, heavy equipment, personnel, etc. One possible method of staging the components would be through the use of overhead cranes strung between temporary towers on opposite walls of the canyon, which could lower the components to the bottom of the canyon. The towers could be used also for supporting the erection of the bridge superstructure. However, the sheer volume of materials and labor involved in building the piers and arches, including pile driving equipment, may necessitate the installment of temporary access roads to the bottom of the canyon. The more gradual topography of the west side of the canyon would be best suited for an access road to the bottom, though a road on the east side or a stream crossing would be required for construction of the east thrust block.

An additional advantage of this configuration would be that a portion of the lateral forces being transferred to the thrust blocks from the arch are offset by the adjacent arch, reducing the number of piles needed to resist lateral forces. The overall cost of the structure would most likely still be the highest of the three options because of the complicated nature of construction of the arches and the three large thrust blocks needed, but a complete analysis of all options would need to be completed in order to arrive at the optimum design, should the G South Alternative be selected for further study.

## 2.3 Cooper Creek Alternative

The Cooper Creek Alternative has three major water crossings within its alignment, including replacement of two existing bridges over the Kenai River and a new crossing over Cooper Creek. Sections 2.3.1 through 2.3.3 discuss each crossing and the structure options considered.

### *2.3.1 Cooper Creek Crossing*

The proposed structure at Cooper Creek is between station 1666+97 and 1675+43 and consists of a cross section that will accommodate two 12-foot lanes, two 8-foot shoulders, one 12-foot climbing lane for the 6 percent grade, and a 6-foot sidewalk on the downstream side. The overall width of the structure will be 62 feet, including bridge railings.

The bridge is conceptually sited approximately one-half mile upstream of the existing bridge and crosses over Cooper Creek in a 1,340-foot radius horizontal curve and 6 percent grade. During preliminary engineering, it was determined that the height of the canyon, distance rim to rim, and accessibility of the canyon bottom was such that shorter spans would be more economical than longer spans. In addition, the profile was lowered to also facilitate a shorter bridge structure and to minimize depth of fill under the west abutment. However, this change resulted in an increased cut on the east side of Cooper Creek. To minimize impacts to the electric transmission line towers in that area, the roadway alignment was shifted north. Only two span arrangements were considered for this crossing which includes three bridge options: chorded prestressed concrete I-girders, deck bulb-tee girders, or curved steel plate girders. A post-tensioned concrete box girder bridge option is also feasible at this location; however, this type of bridge is usually used for longer span lengths. The number of relatively tall piers could be reduced by increasing the span length, but it would probably prove to be difficult to construct because of the horizontal curvature in the road. These complications would cause the box girder bridge to be significantly more expensive than the other two options, and is considered impractical for this crossing.

#### *2.3.1.1 Option 1: Prestressed Concrete I-Girders and Concrete Deck Bulb-Tee Girders*

The total structure length for the Cooper Creek crossing is 846 feet, but the bridge is located where the alignment is in a 1,340-foot radius horizontal curve. By using seven prestressed concrete I-girders or thirteen deck bulb-tee girders as shown on sheet A-13, Cooper Creek can be spanned with seven 120-foot spans that are chorded to provide the necessary horizontal curve. The advantage of using concrete girders over steel girders includes close proximity to some of the raw materials needed, such as aggregate, and lower maintenance costs compared with steel. The disadvantage is that because of the curvature in the road, the girders need to be chorded, which limits the span length of the concrete girders; therefore, the concrete girder option requires two more piers than the steel plate girder option.

#### *2.3.1.2 Option 2: Steel Plate Girders*

The advantage of steel plate girders is that they can be curved to match the horizontal curvature of the road. This curvature is usually more aesthetically pleasing than a chorded arrangement and it increases the maximum span length beyond what the chorded concrete girder arrangement is capable of. Using six curved steel plate girders with a cast in place concrete deck, the span configuration would consist of 142.5-foot approach spans and three 185-foot spans as shown on sheet A-14. The disadvantage with steel is a higher initial and maintenance cost associated with protecting the girders from corrosion.

### *2.3.2 Kenai River Crossing at Cooper Landing*

The existing structure at this location was constructed in 1965 and is approximately 400 feet long and 30 feet wide between curb lines. The superstructure is comprised of wide flange steel beams with a reinforced concrete deck. The substructure for the bridge is comprised of steel H-piles with pier nose protection on the upstream side. Reinforced concrete web walls were constructed between the H-piles to stiffen the substructure and prevent drift from accumulating between the H-piles.

The north end of the structure appears to have been constructed on a large fill section which extends into the Kenai River.

In 1973, pedestrian facilities were added to the structure which provided a 4-foot-wide walkway on the downstream side of the bridge. The pedestrian walkway is comprised of steel tubing cantilevered from the bridge deck and features a timber planked walking surface. Chain link fencing was installed on the outbound edge of the walkway as a safety measure.

The proposed structure at the outlet of Kenai Lake is between station 1816+35 and 1823+05 and consists of a cross section that will accommodate two 12-foot lanes, one 12-foot right turn lane, one 16-foot center turn lane, two 8-foot shoulders, and one 6-foot walkway on the downstream side. The overall width of the structure will be 78 feet, including bridge railings.

The proposed replacement bridge is approximately 670 feet long. The reason for the increase in bridge length from the existing structure is primarily due to the abutment location on the north end of the bridge. When the new alignment is offset from the existing, the abutment can no longer be located on the existing fill placed out in the Kenai River, therefore the new abutment location must be setback approximately 220 feet from the existing abutment. In order to minimize the setback amount from the river bank and eliminate the need to construct costly bridge structure over the end slope, a retaining wall could be utilized to minimize the proposed bridge length. Bridge lengths studied assume the use of a retaining wall at the north end of the structure.

Regardless of the superstructure type, substructures for this bridge would most likely be comprised of reinforced concrete abutments founded on steel pipe piles. Interior bents would likewise be pile supported with multiple piles per bent. It is expected that the standard DOT&PF pile extension bents would be an economical choice for this structure. Pile extension bents consist of a steel casing advanced down to competent bearing soil. Bents are aligned (skewed) as necessary to provide minimal impact on river flows. The steel casing is then cut off slightly below the bottom of the proposed cap elevation. The steel casings are then partially or fully filled with reinforced concrete depending on the loading which must be resisted. A reinforced cap beam is constructed to connect the piles and provide a bearing area for the bridge girders.

#### *2.3.2.1 Option 1: Prestressed Concrete I-Girders or Concrete Deck Bulb-Tee Girders*

For the Cooper Landing crossing, the concrete bridge option would consist of seven prestressed concrete bulb-tee girders with five continuous span lengths of 134 feet and a cast-in-place concrete deck or sixteen deck bulb-tee girders with the same span arrangement as shown on sheet A-15. The advantages of this arrangement over steel would be lower maintenance costs and the ability to use more of the local materials and labor.

#### *2.3.2.2 Option 2: Steel Plate Girder*

The steel bridge option for this crossing would consist of seven steel plate girders with 150-foot approach spans, two 185-foot intermediate spans, and a cast-in-place concrete deck as shown on sheet A-16. The advantage to this arrangement over concrete is the reduction of the number of intermediate piers needed from four to three.

### *2.3.3 Kenai River Crossing at Schooner's Bend*

The existing bridge at Schooner's Bend is located approximately 80 feet upstream of the proposed bridge site and consists of a cast-in-place concrete deck on four steel plate girders with four 70-foot spans. The new bridge relocation reduces right-of-way impacts, improves the highway geometrics, accommodates traffic control during construction, and avoids an eroding bend in the river.

The proposed structure at Schooner's Bend starts at station 1531+50 and consists of a cross section that will accommodate two 12-foot lanes, two 8-foot shoulders and one 6-foot walkway on the downstream side. The overall width of the structure will be 50 feet, including bridge railings.

The proposed replacement bridge will be approximately 325 feet long and has a 25-degree skew relative to the alignment centerline. The relatively short spans are conducive to using prestressed concrete girders and steel plate girders.

#### *2.3.3.1 Option 1: Prestressed Concrete I-Girders or Concrete Deck Bulb-Tee Girders*

For the Schooner's Bend crossing, the concrete bridge option would consist of five prestressed concrete bulb-tee girders with three continuous span lengths of 108.33 feet and a cast-in-place concrete deck or nine concrete deck bulb-tee girders with the same span arrangement, as shown on sheet A-17. The advantages of this arrangement over steel would be lower maintenance costs and the ability to use more of the local materials and labor.

#### *2.3.3.2 Options 2 and 3: Steel Plate Girders*

The steel bridge option for this crossing would consist of five steel plate girders with two 162.5-foot spans and a cast in place concrete deck as shown on sheet A-18. The advantage to this arrangement over concrete is the reduction of the number of intermediate piers needed from two to one.

Another span configuration which would work for both the steel and concrete girder options, as shown in Option 3 on sheet A-19, would be a cast-in-place concrete deck on top of five girders with 100-foot approach spans and one 125-foot intermediate span. This configuration provides a larger navigational clearance for boat traffic trying to navigate the swift waters of the Kenai.

### 2.4 Design Criteria (All Alternatives)

***Permanent Loads (AASHTO LRFD 3.5)***

| | |
|---|---|
| **Density** | |
| Prestressed (P/S) and Reinforced Concrete | $\Upsilon_c$ = (160 pcf) (including reinforcement) |
| Steel | Per AASHTO *LRFD Bridge Design Specifications, Fifth Edition* |
| **Wear Course** | 4 inches of asphalt total |
| **Barrier** | Alaska Multistate Bridge Rail (TL-4) |

***Live Load (AASHTO LRFD 3.6)***

| | |
|---|---|
| **Vertical Moving Loads – Truck Traffic and Lane** | Per AASHTO *LRFD Bridge Design Specifications, Fifth Edition:* (HL-93 plus Lane Surcharge) |
| **Longitudinal Forces** | Per AASHTO *LRFD Bridge Design Specifications, Fifth Edition* |

***Wind Load (AASHTO LRFD 3.8)***

Design for wind and thermal loads will be according to AASHTO specifications. Most bridges along the three alignments will be designed using conventional code techniques; however the Juneau Creek crossing on the Juneau Creek and Juneau Creek Variant alternative alignments may need additional wind studies conducted, especially for wind sensitive structures such as the cable stay option.

***Earthquake Load (AASHTO LRFD 3.10)***

Seismic design of the bridges will be according to AASHTO *Guide Specifications for LRFD Seismic Bridge Design, 2010.*

A literature search of geotechnical data resulted in values of peak ground acceleration around 0.46g for the project area. Peak ground acceleration and other seismic criteria will be determined when a geotechnical investigation is completed.

| | |
|---|---|
| **Acceleration coefficient** | A= to follow with geotechnical investigation |
| **Importance classification** | IC = to follow |
| **Seismic performance category** | Per AASHTO *LRFD Bridge Design Specifications, Fifth Edition* or DOT&PF |

***Thermal Forces (AASHTO LRFD 3.12.2)***

**Temperature range**

| | |
|---|---|
| Concrete structures | -25˚ to 80˚ F |
| Steel structures | -30˚ to 90˚ F |

**Coefficients of thermal expansion (α)**

| | |
|---|---|
| Concrete | 0.0000108 m/m/˚ C (0.000006 ft/ft/˚ F) |
| Steel | 0.0000117 m/m/˚ C (0.0000065 ft/ft/˚ F) |

***Uplift (AASHTO LRFD 3.7.2)***

# 3.0 STRUCTURE COSTS

## 3.1 Methodology

Typically at this stage in project development, costs for bridges are calculated on a square foot basis. This method is appropriate for this situation, where limited engineering information is available upon which to draw firm engineering decisions. The costs presented are for comparison of the various structure types only and are not intended to represent the actual construction costs. Once an alignment has been chosen, a more refined cost analysis will be performed to better determine expected construction costs and the optimal structure option for each crossing. This section discusses values (costs per square foot) generally used to provide bridge cost estimates.

### *3.1.1 Short Span Concrete or Steel Girder Options*

Only crossings over the Kenai River and the Sportsman's Landing Grade Separation fall into this category because they consist of either a prestressed concrete girder or steel plate girder option on a simple tangent alignment. Most of the crossings used either a cost of \$450 per square foot ($ft^2$) for concrete superstructures or \$625 per $ft^2$ for steel superstructures, the basis of which is discussed later in this section, with the exception of Kenai River crossing at Cooper Landing. At Cooper Landing, the new alignment is fairly close to the old alignment requiring the use of a detour bridge to construct the new bridge. To estimate the extra cost of the detour bridge, the square footage cost of the new bridge was increased by about \$50 per $ft^2$, bringing the total square footage cost used for the Cooper Landing crossing estimate to \$500 per $ft^2$ for concrete and \$675 per $ft^2$ for steel.

Overall costs for projects are often well and above the cost that are generally associated with "furnishing and installing" materials. The DOT&PF informally provided these costs for another project that HDR is working on in cooperation with the DOT&PF. These average costs were assembled to provide comparison numbers for the costs HDR developed on that project. The costs presented and discussed below were based on Alaska Bid Data from 2000 to 2003. The study identified that additional costs not directly tied to "furnishing and installing materials" include: approximately 15 percent cost increase to account for detours, bridge removal, riprap, etc.; 10 percent for mobilization/demobilization; 15 percent for construction engineering and 4.3 percent for Indirect Cost Allocation Plan (ICAP). These result in an approximate increase of 55 percent over the basic cost of furnishing and installing materials directly associated with bridge construction.

Based on DOT&PF bid history, the base cost for a "deep pile" decked bulb-tee girder bridge with a pile extension substructure such as that used for the Kenai River crossings is on the order of \$250 to \$350 per $ft^2$ with an adjusted price of \$450 per $ft^2$. For bridges with steel superstructures, assuming the substructure stays the same, it is estimated that the cost of the bridge increases to an adjusted price of approximately \$625 per $ft^2$.

### *3.1.2 Long Span or Complicated Alignment Options*

A square footage cost of \$800 per $ft^2$ was used for all long span bridges or bridges on curved alignments. This results in all long span bridges with the same deck area to be estimated at similar overall costs. In reality the various options for each crossing could be considerably different depending on total materials used and the difficulty of construction; however, the simplistic approach to cost estimation used in this report is useful in comparing the cost of bridge structures for different alignments. Once an alignment is chosen, a more refined cost analysis can be performed to determine the preferred option for each crossing. This will most likely involve a more in depth structural analysis of each structure to determine foundation and structural member sizes.

The \$800 per $ft^2$ cost estimate used for estimates of structures in this category was based on other long span structures either recently completed or currently in the project development stage. The Mike O'Callaghan - Pat Tillman Memorial Bridge (shown in Figure 3) was constructed at a cost of approximately \$680 per $ft^2$. Some of the cost estimates used in the development of the Gravina Access Project were based on bridge costs of approximately \$750 to \$1,200 per $ft^2$, with the bridges at the upper cost range containing curvature in the superstructure or other costly characteristics. Based on the level of detail currently known and the complexity of these bridges, using an estimated cost of \$800 per $ft^2$ seems reasonable at this stage of project development. As another means of approximating the cost of the long span option at this level of detail would be to assume costs that are nearly double what would be expected for the short span concrete superstructure.

The costs presented in this document are intended to be used for cost comparisons between the alternatives and are not intended to reflect actual construction costs. The complicated nature of each of the long span bridges would likely come at a premium cost, whether due to uncommon design methodology (i.e., the asymmetrical cable stay tying into concrete blocks) or due to structure length exceeding common bounds (i.e., the 825-foot post-tensioned concrete box girder). Rather than attempt to identify and quantify the premium cost that may accompany each respective structure type, the high unit cost was applied equally to all with the understanding that the unit cost would approximate the various premiums for those options. At this stage of project development, the information contained herein is useful to compare the alternatives at a planning level. Given the limited amount of geotechnical information and the number of feasible bridge options per alternative, a more detailed bridge option selection report should be prepared for the selected alternative to select a bridge option and refine the structure costs in order to arrive at a more accurate construction estimate.

# 4.0 RECOMMENDED STRUCTURE OPTIONS

At this stage in project development it is premature to recommend the optimum structure for each crossing; however, there are several options discussed in this report that can be eliminated due to impracticalities and inefficiencies in their design, particularly some of the crossing options for the Juneau Creek crossing on the Juneau Creek or Juneau Creek Variant alternatives. During the preliminary design of the Juneau Creek crossing, all of the technically feasible structure options were considered; however, there currently has never been a simple span steel truss over 745 feet long. Since a span length of 825 feet is required for the long span Juneau Creek crossing, this option should not be considered further due to the unconventional span length, which would require designs that have not been tested in real world applications. In addition, the continuous span steel truss and post-tensioned segmental box girder for the Juneau Creek crossing would need substantial earthen cuts and roadway realignments, so they too should not be considered further.

Of the remaining bridge options, the cost per square foot values were used to determine relative bridge cost estimates, which are summarized in Appendix B. Based on these estimates, the least expensive alignment in terms of bridge costs would be the Juneau Creek Alternative, which has an estimated total major crossing cost of \$41.1 to \$81.8 million. The Juneau Creek Alternative is followed by the Juneau Creek Variant Alternative (\$45.0 to \$87.2 million), Cooper Creek

Alternative ($75.3 to $87.2 million), and G South Alternative ($90.0 to $99.4 million). These numbers are for comparison purposes only, and additional considerations such as environmental impact and roadway cost require consideration before an optimum alignment can be selected.

Table 2 provides a summary of the bridge options for the various alignment alternatives, including recommendations regarding which options should be studied for further consideration.

The Juneau Creek / Juneau Creek Variant bridge options that contain the shorter span length of 450 feet are kept as recommendations for further consideration, though their implementation is predicated upon geotechnical investigation revealing competent founding material. Surficial geotechnical reconnaissance documented in R&M's Preliminary Rock Stability Investigation cautions the founding of bridge components near the canyon walls. Such options may ultimately represent successful structures for this crossing, but stability in the foundations must be established.

**Table 2: Bridge Recommendations**

| Alternative | Possible Bridge Option | Consider Further? | Advantages |
|---|---|---|---|
| Juneau Creek / Juneau Creek Variant at Juneau Creek | • Steel Tied Arch | YES | • Shortest structure length |
| | • Cable Stay (Asymmetric) | YES | • Cantilever construction |
| | • Simple Span Steel Truss | NO | • NA |
| | • Continuous Steel Truss | NO | • NA |
| | • Segmental Concrete Box Girder | NO | • NA |
| | • Segmental Concrete Box Girder (450-ft) | YES | • Shortened main span<br>• Cantilever construction |
| | • Deck Arch w/ Prestressed Concrete Girder Approaches | YES | • Shortened main span |
| Juneau Creek Variant at Sportsman's Landing | • Prestressed Concrete Girder | YES | • Precast members for construction ease and speed |
| | • Steel Plate Girder | NO | • NA |
| G South at Kenai River | • Prestressed Concrete Girder | YES | • Lower maintenance cost<br>• Uses local materials/labor |
| | • Steel Plate Girder | YES | • Reduced needed piers |
| G South at Juneau Creek | • Prestressed Concrete Girder w/ Cast in Place Pier Sections | YES | • Precast members for construction ease and speed |
| | • Deck Arch w/ Prestressed Concrete Girder Approaches | YES | • Reduced towers and canyon floor impact |
| | • Multiple Deck Arch | YES | • Aesthetically pleasing<br>• Least environmental impact |
| G South at Schooner's Bend | • Prestressed Concrete Girder | YES | • Lower maintenance cost<br>• Uses local materials/labor |
| | • Steel Plate Girder | YES | • Reduced needed piers |
| Cooper Creek at Cooper Creek | • Chorded Prestressed Concrete Girder | YES | • Lower maintenance cost<br>• Uses local materials/labor |
| | • Curved Steel Plate Girder | YES | • Reduced needed piers |
| Cooper Creek at Cooper Landing | • Prestressed Concrete Girder | YES | • Lower maintenance cost<br>• Uses local materials/labor |
| | • Steel Plate Girder | YES | • Reduced needed piers |
| Cooper Creek at Schooner's Bend | • Prestressed Concrete Girder | YES | • Lower maintenance cost<br>• Uses local materials/labor |
| | • Steel Plate Girder | YES | • Reduced needed piers |

# 5.0 REFERENCES

HDR Alaska Inc. *Affected Environment Technical Memorandum*. Sterling Highway Milepost 45 to 60. September 2001.

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