Lowell Creek Flood Diversion
Seward, Alaska
Appendix C: Hydraulic & Structural Design

April 2021

US Army Corps
of Engineers®
Alaska District

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Integrated Feasibility Report and Environmental Assessment

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Seward, Alaska

Prepared By:

U.S. Army Corps of Engineers

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1. INTRODUCTION

1.1. Appendix Purpose

This Appendix describes the technical aspects of proposed modifications to the Lowell Creek Flood Diversion Project. It provides the engineering background information for determining the Federal interest in the major construction features, including tunnels, diversion dams, elevated outfalls, tunnel portal canopies, sediment retention basins, and support facilities. Existing data was gathered and analyzed to determine the site characteristics. Numerical modeling was performed to determine the physical impacts of the flood flows for the design of the proposed flood reduction measures.

2. PROJECT SUMMARY

2.1. Project Authorization

The existing Lowell Creek Flood Diversion project consists of a diversion dam and tunnel, with the diversion dam and tunnel entrance located approximately 0.1 mile west of the closest buildings of Seward, Alaska, near the mouth of Lowell Creek Canyon. The diversion dam and tunnel divert stream flow, from the natural stream channel, through Bear Mountain, and into Resurrection Bay at the south edge of downtown Seward. The project authority for the existing project is the Flood Control Act of 1936 (Public Law (PL) 74-738). The authorized project purpose is flood risk management.

The authority for this study is Section 5032 of the Water Resources Development Act of 2007 (PL 110-114), as amended. As of November 2007, in accordance with Section 5032, The Secretary of the Army has assumed responsibility for long-term maintenance and repair of the tunnel until an alternative method of flood diversion is constructed and operational, or until 20 years after the enactment of this Act (November 2027), whichever is earlier.

2.2. Project Description

The main components of the project are shown in Figure 1 below and include a diversion dam, emergency spillway, and tunnel. Drawings depicting the key features of the project are included in Attachment 12.1 of this Appendix.

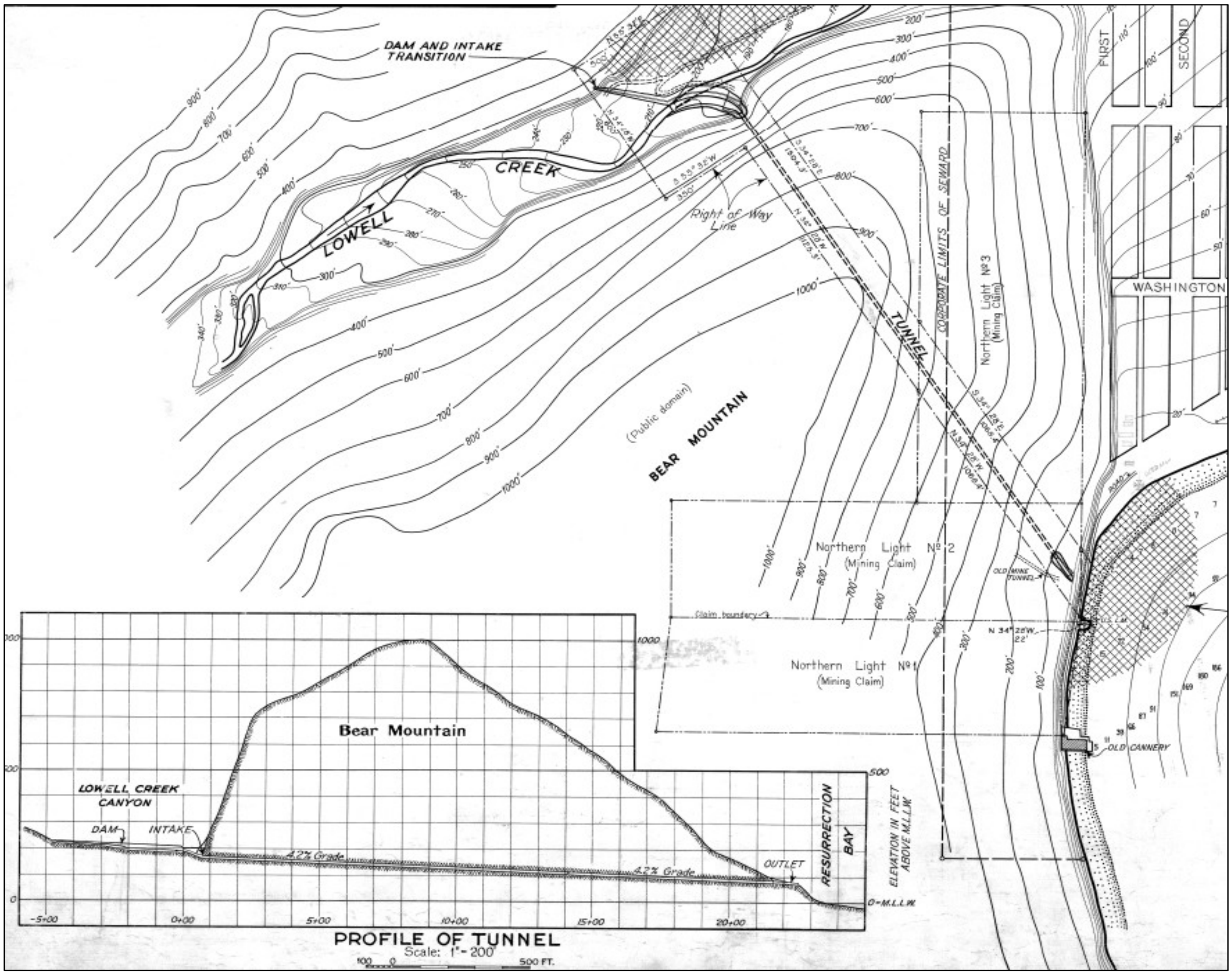

Figure 1. Project Overview.

The diversion dam and tunnel divert floodwater and debris away from the City of Seward. When constructed, the tunnel's outfall area was unused; the 1937 authorization document indicates that the area would be "obliterated,"; and the assumption was that the buildup of debris would spill into the deep water in Resurrection Bay. Subsequent use of this area, and adjacent areas, has required the City of Seward to use heavy equipment during flood events in an effort to protect adjacent infrastructure and the road and bridge serving the portion of the community that is south of the tunnel outfall. A summary of pertinent project data is found in Table 1. Note that no reservoir data is associated with this project. As a stream diversion on a steep gradient, no still-water pool is impounded behind the dam.

Table 1. Pertinent Data.

Lowell Creek Diversion Dam	
Type	Diversion Dam
Design crest elevation	Varies approx. 225.7 – 203.2 feet (ft) (NAVD88)
Crest width	5 ft
Length	450 ft
Structural height (maximum height above streambed)	25 ft
Type	Tunnel
Size	10-ft-diameter horseshoe
Length	2,089 ft
Average Grade	-4.2 %
Maximum discharge capacity	Approx. 2,800 cubic feet per second (cfs)
Lowell Creek Dam Spillway	
Type	Uncontrolled Weir
Crest Elevation	199.0 ft NAVD88
Width	~70 ft
Maximum discharge capacity	1,700 cfs

Notes:
1. All elevations given in Table 1 are based on the 1945 design drawing elevations rounded to the nearest 10th of a ft, comparing these values with the 2006 Light Detection and Ranging (LiDAR) topographic data, which is in NAVD88, and subtracting 3.5 ft to make the 1945 elevations roughly match the 2006 LiDAR elevations. Adjustments of this type are approximate.
2. The hydraulic height value given is based on the 2006 LiDAR data.
3. The source of data is the 2012 inundation report and original contract drawings.

2.2.1. Lowell Creek Diversion Dam

Lowell Creek Diversion Dam is located approximately 1,400 ft upstream from the mouth of Lowell Creek Canyon, immediately adjacent to the major population center, or downtown area, of the City of Seward, Alaska, which is built on the alluvial fan from the original stream course. The diversion dam consists of a 450-ft-long rock-filled embankment with a crest elevation that varies from 225.7 to 203.2 ft North American Vertical Datum of 1988 (NAVD88) and a maximum height of 25 ft above the adjacent streambed. The left abutment is the high end of the dam with the alignment crossing the canyon bottom and running downstream from left to right, with the crest falling on a 5% grade. The dam is designed to divert water into the tunnel and does not impound water. Figure 2 provides a typical cross section of the embankment dam.

The upstream face of the dam is a reinforced concrete slab sloped at one horizontal to one vertical (1H:1V). The downstream face of the dam is a grouted rock-fill sloped at two horizontal to one vertical (2H:1V). The rock-fill for the embankment was specified to range in size from 0.5 cubic ft to 1 cubic yard (cy), of which not less than 25% shall be in pieces of 5 cubic ft or more in volume. Rock chips and spalls were specified to be included only to the extent necessary to fill the voids between the larger stones. Rock slabs having an average thickness of less than 25% the average width were not allowed. The left abutment of the dam is constructed against the canyon wall, with the rock cut to a four horizontal to one vertical (4H:1V) slope and a concrete slab attached with dowels against the rock face. The right abutment of the dam, which doubles as the constant-elevation spillway is tied into the tunnel entrance and is cast against the rock of Bear Mountain. A 12-inch drain pipe was also installed for use during maintenance operations; however, debris has plugged this pipe, and it is not usable.

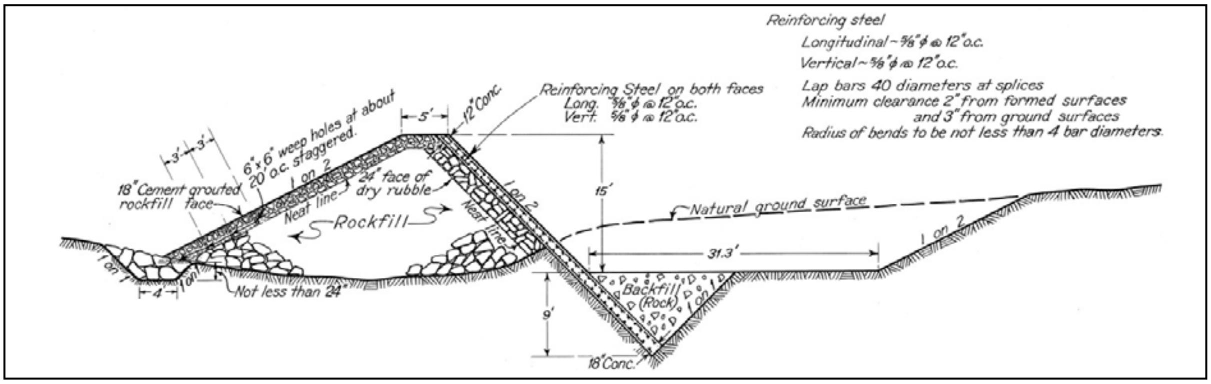

Figure 2. Typical Embankment Cross-Sections.

The City of Seward placed a 12-inch-diameter steel water line through the dam near the left abutment circa 1982. During the installation of this water line, a section of the dam was removed to facilitate construction. Third-hand information indicates that a concrete cap was placed in this area when the dam was rebuilt. However, no details regarding this penetration of the dam are available.

The emergency spillway is a 70-ft-wide, constant elevation, notch at the low-end of the dam. The crest elevation of the spillway is approximately 199 ft NAVD88. The spillway is constructed of rock-fill with a reinforced concrete upstream slope and a 5-ft-wide reinforced concrete crest. The discharge capacity is 1,700 cfs when flowing at elevation 203 ft NAVD88, the approximate elevation of the low end of the dam crest adjacent to the spillway. There currently is no channel below the emergency spillway, and no documentation was found to indicate that a channel was required to be maintained through Seward for spillway conveyance. The original channel across the alluvial fan was present when the project was constructed. The unregulated structures have no project staff.

2.2.2. Lowell Creek Tunnel

The dam functions to divert Lowell Creek into a 2,089-ft-long, 10-ft-diameter, concrete-lined, –4.2% slope, horseshoe tunnel (Figure 3) through Bear Mountain that exits into an approximately 100-ft-long, concrete trapezoidal channel. Construction began in 1939 and was substantially complete by the fall of 1940. The entrance to the tunnel (intake transition) has a large, ogee-like, drop, which accelerates the water to supercritical velocities, facilitates debris movement through the tunnel, and helps to prevent tunnel blockage. The tunnel was constructed with drill and blast techniques. The bedrock was supported with timbers and lagging until the placement of the tunnel liner. It is believed the timber supports were left in place during liner construction, and no contact grouting was performed after the liner was placed. The tunnel is lined with concrete throughout, and the inverts of both the tunnel and intake transition were originally armored with 40-pound/yard railroad rails bolted to channel crossties embedded in and bolted to the invert. The lower portion of the outside curve of the intake transition is rail lined. Both sidewalls of the tunnel near the entrance are also rail lined. Fully exposed rails and fasteners were damage-prone; thus, the rails were welded to the crossties, and the space between rails was filled with concrete before project turnover circa 1945.

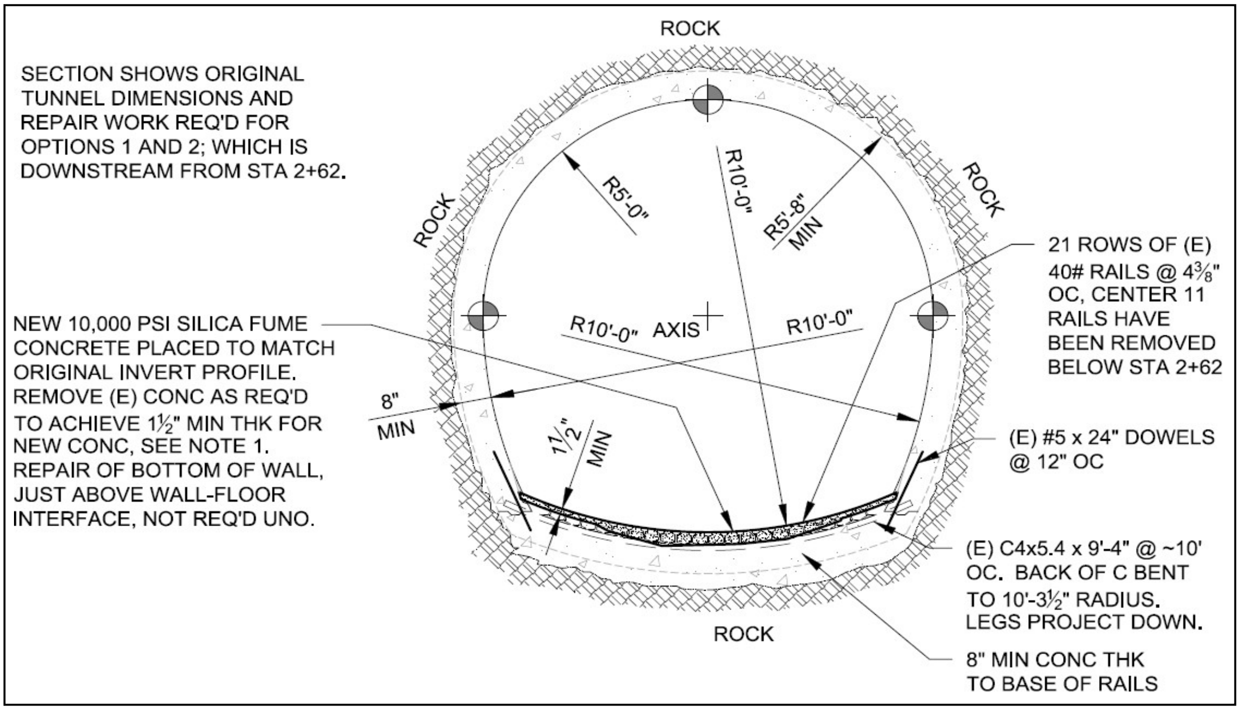

Figure 3. Typical Tunnel Section with Recent Repair Annotations.

The tunnel exits to a trapezoidal concrete flume 10 ft wide at the bottom and 109 ft long. The outlet invert of the flume is 70.5 ft NAVD88, which allows for the accumulation of debris carried through the tunnel. The flume exits over a near-vertical rock cliff. At the toe of the cliff, the City of Seward works to maintain a creek channel, which currently continues about 500 ft to tidewater. A two-lane bridge crosses the channel about 100 ft from the toe of the cliff.

3. LOWELL CREEK HYDROLOGY

3.1. Location and Vicinity

Seward lies at the head of Resurrection Bay, a deep fiord about 25 miles long on the north shore of the Gulf of Alaska. Near Seward, the bay is 2 to 3 miles wide and about 500 ft deep. Water is deep immediately offshore with an exception for the head of the bay and at the toe of alluvial fan-deltas. The glaciated Kenai Mountains rise steeply above Resurrection Bay and the valley of the Resurrection River. The highest peaks on the west side of the bay and river reach altitudes of 4,000–5,000 ft.

3.2. Lowell Creek Canyon

Lowell Creek drains a 4.02 square mile basin between Mount Marathon and Bear Mountain to the west of Seward (Figure 4). The terrain in the basin is mountainous, consisting of steep slopes of loose rock. Due to the steep slopes of the basin and the rocky nature of the material, rain falling in Lowell Creek Canyon has a high runoff percentage and a low time of concentration.

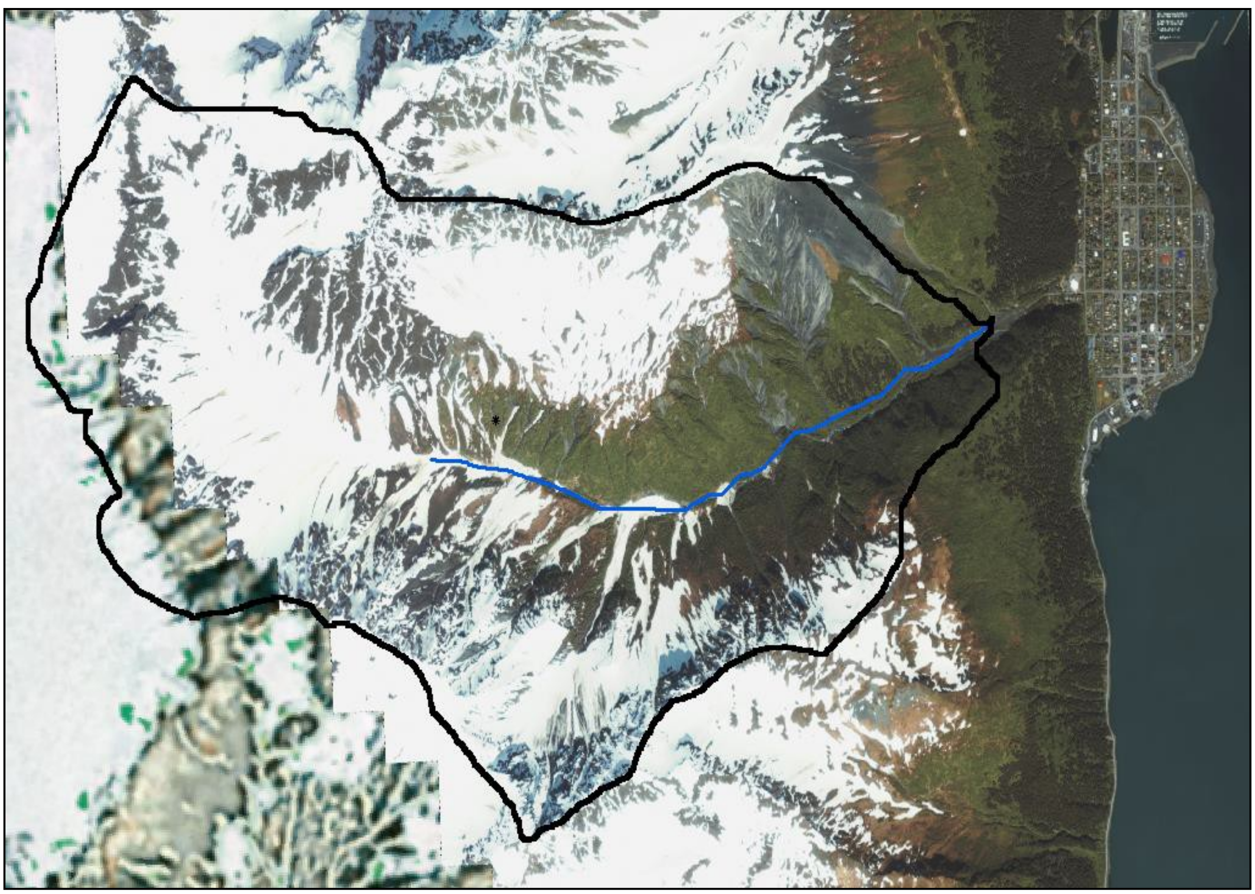

Figure 4. Lowell Creek Canyon.

3.3. Alluvial Fan

The downtown area of Seward is located on the alluvial fan of Lowell Creek (Figure 5). Alluvial fans are depositional landforms, located at the base of mountain ranges where a steep mountain stream emerges onto lesser valley slopes. They are usually conical or fan-shaped in plan-view. On topographic maps, they appear as contour lines that are concentric around the canyon mouth. Sediments deposited on alluvial fans are generally coarse-grained, composed of sand, gravel, cobbles, and boulders. The unbounded lateral dimensions and rapid depositional nature of alluvial fans support frequent avulsions (rapid change in channel direction) and flow spreading laterally on the fan surface.

Figure 5. Lowell Creek Alluvial Fan.

Flooding on alluvial fans is a type of flood-hazard that occurs only on alluvial fans with two areas generally defined. The upper area of the alluvial fan contains a section where the flow path can generally be determined with some degree of certainty. This area is subject to erosion and deposition, but a relatively stable flow path remains during floods. Downstream from this area, alluvial fan flooding is characterized by flow path uncertainty so great that this uncertainty cannot be set aside in a realistic assessment of flood risk or the reliable mitigation of the hazard. An idealized plan view of an alluvial fan is shown in Figure 6. The upper area of the alluvial fan is shown as the channelized zone with the lower braided and sheet flow zones consisting of the more active flooding areas. This active alluvial fan flooding area is indicated by three general conditions:

- Flow path uncertainty below the apex of the alluvial fan.
- Abrupt deposition and erosion of sediment as the stream loses its ability to transport material.
- A combination of sediment supply and steep slopes creates an extremely hazardous flood condition.

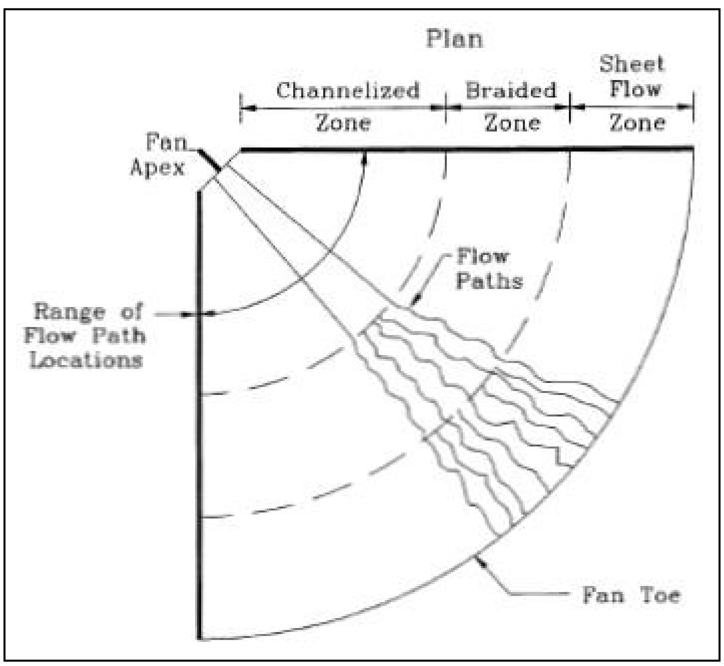

Figure 6. Plan View of Idealized Alluvial Fan.

The Federal Emergency Management Agency defines alluvial fan flooding in Section 59.1 of Chapter 44 of the Code of Federal Regulations as flooding occurring on the surface of an alluvial fan or similar landform which originates at the apex and is characterized by high-velocity flows, active processes of erosion, sediment/debris transport, deposition, and unpredictable flow paths.

Lowell Creek is a unique alluvial fan in that the river no longer actively flows past the apex of the fan but rather is diverted through Bear Mountain, and the entire alluvial fan is developed with the only available conveyance being overland flooding through the city. A profile for Lowell Creek is shown in Figure 7.

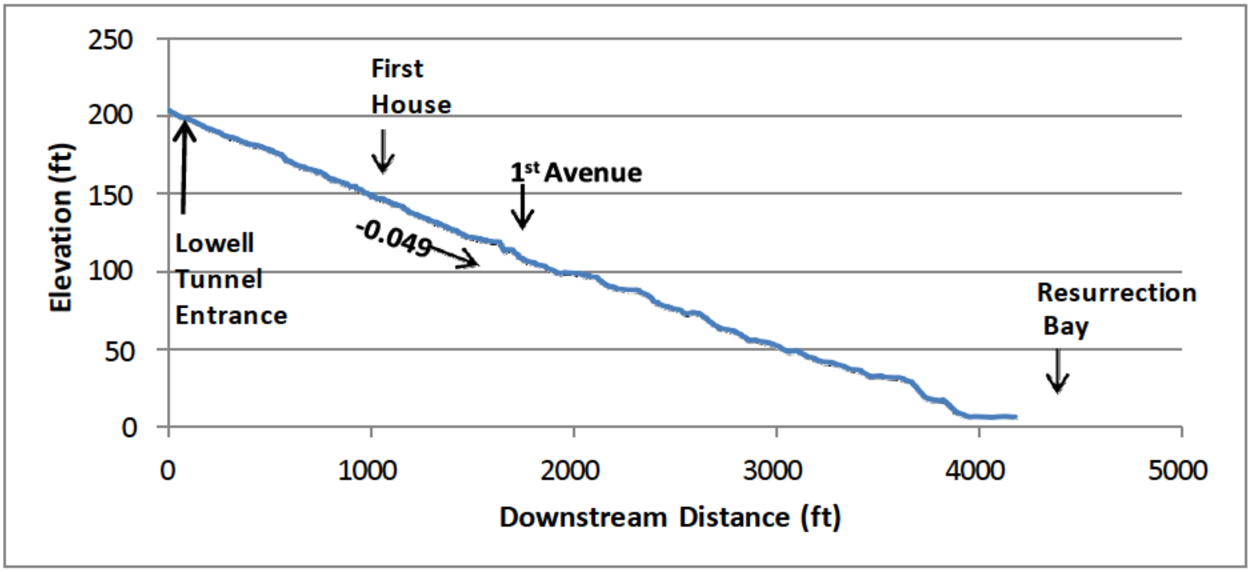

Figure 7. Lowell Creek Original Stream Path Profile Across Alluvial Fan.

3.4. Climatology

The extreme mountain relief and its effect on the coastal maritime climate cause great local variations in weather in the Resurrection Bay-Seward area. This general circulation of air masses is driven by migrating pressure centers in the Gulf of Alaska. The lifting and cooling of moist air masses at the mountain fronts cause a rapid increase in precipitation with increasing elevations along the windward side of the mountains. Mean annual precipitation ranges from 67 inches at Seward to more than 100 inches in the high-altitude glaciated areas. About 40% of the total annual precipitation falls as rain from September through November. Beginning in early October, the precipitation above an altitude of 2,100 ft is usually in the form of snow, most of which is stored in mountain and glacier snowpack. Severe flooding on Lowell Creek normally mirrors the October through November rainfall period, with one known major flood occurring as late as early December. Floods are normally of short duration, lasting only three or four days. Lowell Creek rises very rapidly, with flooding occurring soon after heavy rainfall begins. There are no flow measurement gages on Lowell Creek, except for the newly installed flow gage at the outlet, and no rainfall gages in the Lowell Creek basin.

3.5. Flow Frequency

No reliable peak flow data exists for Lowell Creek. The best available data are from the next basin to the south, Spruce Creek, which was gaged for discharge by the U.S.

Geological Survey (USGS) from 1966 to 2009. The annual peak flows for Spruce Creek were used in HEC-SSP 2.1.1.137 (January 5, 2017) model to perform a Bulletin 17C Expected Moments Algorithm flow frequency analysis. It is noted that the flood of record peak flow value was not taken directly from the USGS data set. Instead, an estimated value for the data of 11 October 1986 (Water Year 1987) has been included as a Systematic Event. The USGS data set lists this flow as 13,600 cfs. The abstract on page 1 of the 2016 USGS Scientific Investigations Report 2016-5024, titled "Estimated Flood Magnitude and Frequency at Gaged and Ungaged Sites on Streams in Alaska and Conterminous Basins in Canada, Based on Data through Water Year 2012." (page 9 of 51 of the document) states that the 13,600 cfs value was about 2.5 times as great as the runoff rate upstream from the debris dam. Additional discussion regarding these values is also found on pages 25 and 27-29 of this report. Thus, the estimated value for 11 October 1986 is 13,600 divided by 2.5 or about 5,420 cfs, as stated on page 29 of the USGS document. The use of this value is in line with the recommendation of Engineering Manual (EM) 1110-2-1415, paragraph 3.2, e. Incomplete Record, which states, "Missing high events may result from the gage being out of operation or the stage exceeding the rating table. In these cases, every effort should be made to obtain an estimate of the missing events."

The Regional Skew (0.420) and Mean Squared Error of the Regional Skew (0.1476) used for Spruce Creek was aided by the inclusion of Spruce Creek data in the development of USGS Scientific Investigation Report 2016-5024. Table 4 of the USGS report (Excel file, sir20165024_table04.xlsx) indicates that the USGS developed these values. It is believed that these values are an improvement of the more general values indicated by this USGS document's Table 6, where Spruce Creek is part of Regional Skew Area 2, the Regional Skew is indicated to be 0.18, and the Mean Squared Error of the Regional Skew is shown as 0.34. The Station skew was evaluated by HEC-SSP at -0.074. The weighted skew, based on the Station skew and the USGS site-specific skew, used for the best estimate of flows is 0.149.

The resulting Spruce Creek flow-frequency data has been scaled by the ratio of the flows for Lowell Creek and Spruce Creek as predicted by the 2016 USGS method presented in their Scientific Investigations Report 2016-5024, titled "Estimated Flood Magnitude and Frequency at Gaged and Ungaged Sites on Streams in Alaska and Conterminous Basins in Canada, Based on Data through Water Year 2012" (Curran et al. 2016). This scaling, which includes adjustments for differences in basin area and average annual precipitation, adjusted the Spruce Creek data to the Lowell Creek basin. Where our work required frequency information outside the range covered by the USGS methodology, the closest ratio from the USGS equations was used to make the adjustment from the Spruce Creek values to the values for the Lowell Creek basin.

It is noted that the Spruce Creek gage data inherently include some effects of bulking as the actual measurement is stage, which includes the sediment bulking present in Spruce Creek. The relative locations of the gage in Spruce Creek and the diversion dam in Lowell Creek supports the use of some additional sediment bulking for Lowell Creek. The gage in Spruce Creek was approximately 1.75 miles below the canyon portion of Spruce Creek in an area of the basin characterized by a meandering, alluvial channel. The diversion dam in Lowell Creek is within the Lowell Creek Canyon. Thus, additional bulking was assumed to be prudent for the Lowell Creek flows. The idea is that some of the sediment carried in Spruce Creek may have dropped out before reaching the gage location. The flows from HEC-SSP have been increased by a factor of 1.11 to address the uncertainty of the level of bulking in Lowell Creek. This bulking factor (BF) represents a volumetric concentration factor of 10%. It is noted that this sediment bulking is intended to address long-term sediment concentration issues and that short-term sediment concentrations likely vary considerably from this estimate.

The resulting flow frequency curves are our best estimate of the steady-state (non-surge) flow conditions for Lowell Creek and are shown in Figure 8. Discrete, numeric, flow values for various annual exceedance probabilities (AEP), and an approximate return period for the probable maximum flood (PMF), are provided in Table 2. The PMF flow estimate was also bulked by the same 1.11 multiplier taking it from 7,600 cfs to about 8,400 cfs.

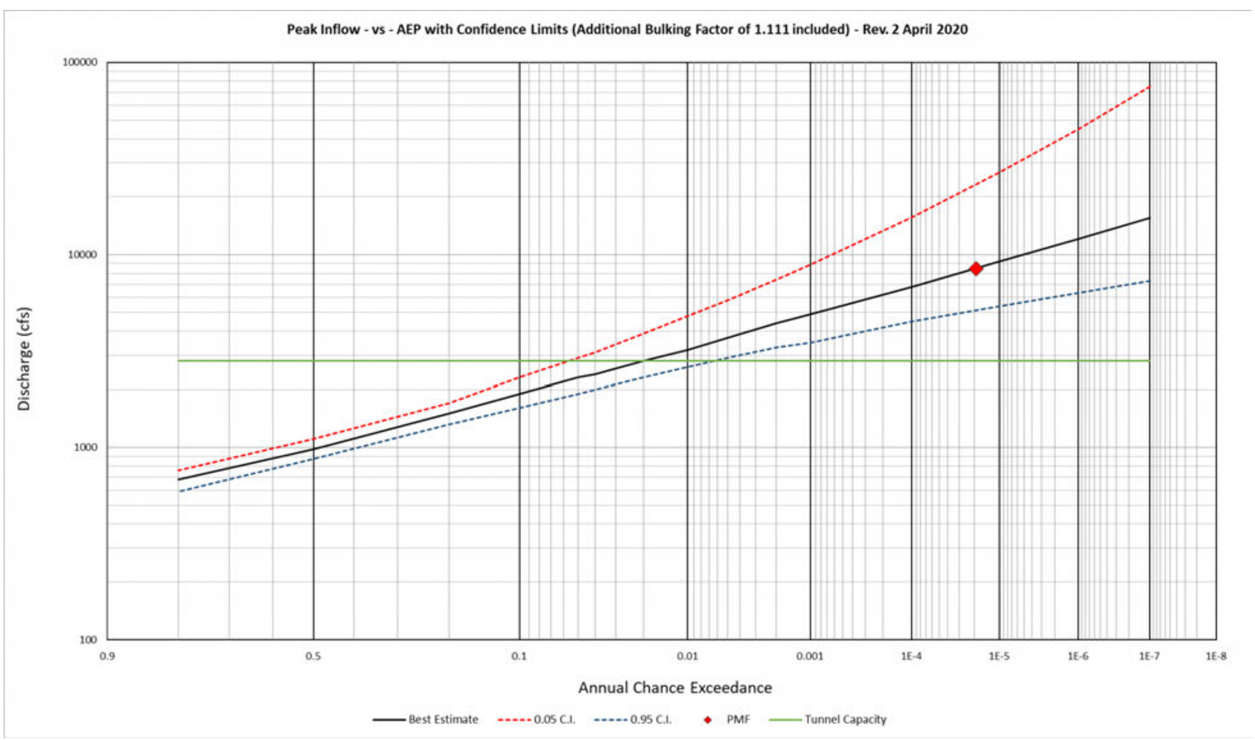

Figure 8. Annual Bulked (BF=1.11) Peak Flow Frequency.

Table 2. Numeric Annual Peak Flow-Frequency Data.

Lowell Creek Stream Flow Numeric Flow Frequency Data			
AEP	Best Estimate (cfs)	5% C.I. (cfs)	95% C.I. (cfs)
1E-07	15,500	75,000	7,300
1E-06	12,100	45,000	6,300
1E-05	9,200	26,800	5,400
1E-04	6,800	15,700	4,500
0.001	4,900	8,900	3,500
0.01	3,200	4,800	2,600
0.02	2,800	3,900	2,300
0.05	2,300	2,900	1,900
0.1	1,900	2,300	1,600
0.2	1,500	1,700	1,320
0.5	980	1,110	870
0.99	360	440	270

Notes:
- PMF plots at an AEP of about 1.89E-05
- Indicated approximate PMF return period is 53,000 years

3.6. Tunnel Capacity

The published capacity of the tunnel has varied throughout the historical documents, from a design flow of 1,935 cfs flowing at a depth of 6.64 ft to a high value of 3,173 cfs as reported in a 1949 document. More recent estimates in 1988 and 1992 were 2,600 and 2,200 cfs, respectively. The 2012 inundation study re-evaluated the hydraulics of the tunnel. Using the parameters listed in Table 3, the capacity of the tunnel was determined to be 2,800 cfs with the upstream water level 1 ft below the crest of the emergency spillway. Water velocity through the tunnel is approximately 35 ft per second. The tunnel continues to operate as an inlet-controlled conduit until the flow through the tunnel approaches 3,000 cfs, at which point flow through the tunnel would undergo a violent pulsating transition to outlet-controlled flow. It should be noted that both the capacity of the tunnel and the transition threshold to outlet control are significantly reduced as roughness within the tunnel increases above the values noted below in Table 3.

Table 3. Tunnel Parameters.

Parameter	Value
Entrance Loss Coefficient	0.3
Exit Loss Coefficient	0.1
Tunnel Bottom Roughness	*n = 0.02*
Tunnel Walls and Roof Roughness	*n = 0.014*

3.7. Design Flood Events

3.7.1. Original Design Flood (1937)

Based on the 1937 authorization request to the U.S. Congress, it appears that the original design flood was an estimate of the “largest flood known to have occurred on Lowell Creek” before that time. The 1937 letter from The Secretary of War states, “the maximum discharge is estimated at not more than 2,000 second-feet” (cfs). Thus 2,000 cfs is assumed to have been the design discharge for the Lowell Creek tunnel. The project was not designed to standards that would be required today.

3.7.2. Implementation of the National Dam Safety Program (1978)

In 1978, the Alaska District reviewed the hydraulic adequacy for the project as part of the implementation of the National Dam Safety program. The following description of the method used in the 1978 study is taken from Alaska District’s 1992 Revised Reconnaissance Report:

> “The PMF determination was made using the computer program HEC-1 and assuming Snyder’s unit hydrograph coefficients for C_p (peaking coefficient) and T_p (time to peak). The Probable Maximum Precipitation (PMP) used was a reduction of the PMP provided by the National Weather Service for the Swan Lake hydropower study by a ratio of the 100-year, 24-hour precipitation amounts at Seward and Ketchikan. Total 72-hour PMP was 27 inches, with a maximum 1-hour concentration of 3.38 inches. Assuming a loss rate of 0.1 inches per hour below elevation 1,500 feet and zero loss rate above elevation 1,500 feet, T_p = 2.00 hours, C_p = 0.63, and the peak flow of the PMF was computed to be 4,400 cubic feet per second (ft^3/s) for the 4.02-square-mile Lowell Creek drainage area. This PMF derivation did not consider the effect that some type of mass movement within the basin might have on the hydrograph” (USACE 1992).

During this review, the spillway design flood selected was ½ of the PMF or approximately 2,200 cfs.

3.7.3. Flood Damage Reduction Revised Reconnaissance Report (September 1992)

In 1992, the Alaska District reviewed the hydraulic adequacy of the project as part of a reconnaissance report investigation. The following paragraphs relating to the development of a reasonable PMF are taken from this 1992 report:

> “Final derivation of the PMF for Lowell Creek is planned during the feasibility phase of study, which would include a surge-release type of flooding

> mechanism in conjunction with the probable maximum precipitation. The National Weather Service Hydrometeorological Branch would be asked to review the existing PMP for the Seward area and the 1986 storm. For the current study, only an estimate of the PMF, with surge-release flooding, is used. Its derivation follows.
>
> A PMF of 4,400 ft^3/s seems low for Lowell Creek. The PMF unit runoff is 1,100 ft^3/s per square mile. A runoff of 1,020 ft^3/s per square mile was measured in the adjacent basin for the October 1986 flood, which was not affected by debris flow or surge- release flooding. A PMF of 4,400 ft^3/s relates to a 3,500-year return interval flow on a waterflood-based frequency curve for Lowell Creek. A 10,000-year return interval flow of 5,400 ft^3/s was therefore assumed to be more representative of a waterflood PMF than that derived previously.
>
> A surge-release type event was considered highly probable during the PMF. The 2.5 multiplier from the Spruce Creek surge-release event was applied to the 5,400-ft^3/s estimated rainfall PMF for a surge-release PMF of 13,600 ft^3/s, which was used for a design criterion in developing alternative solutions. The surge-release PMF hydrograph shown on Figure 4 [reference from 1992 report, see Figure 9 in this report] is a very crude approximation of what could happen during this type of event. The hydrograph shape and timing are based on HEC-1 output of the rainfall, ice, and snowmelt hydrograph and an estimate of the impacts of a landslide-created dam that fails” (USACE 1992).

The 1992 Alaska District report states the following concerning the Inflow Design Flood (IDF):

> “The IDF must be able to pass safely through the project without overtopping the structure. The capacity of the tunnel is approximately 2,350 ft^3/s at the spillway crest. The PMF is estimated to be in the range of 13,600 ft^3/s. The IDF would be the same” (USACE 1992).

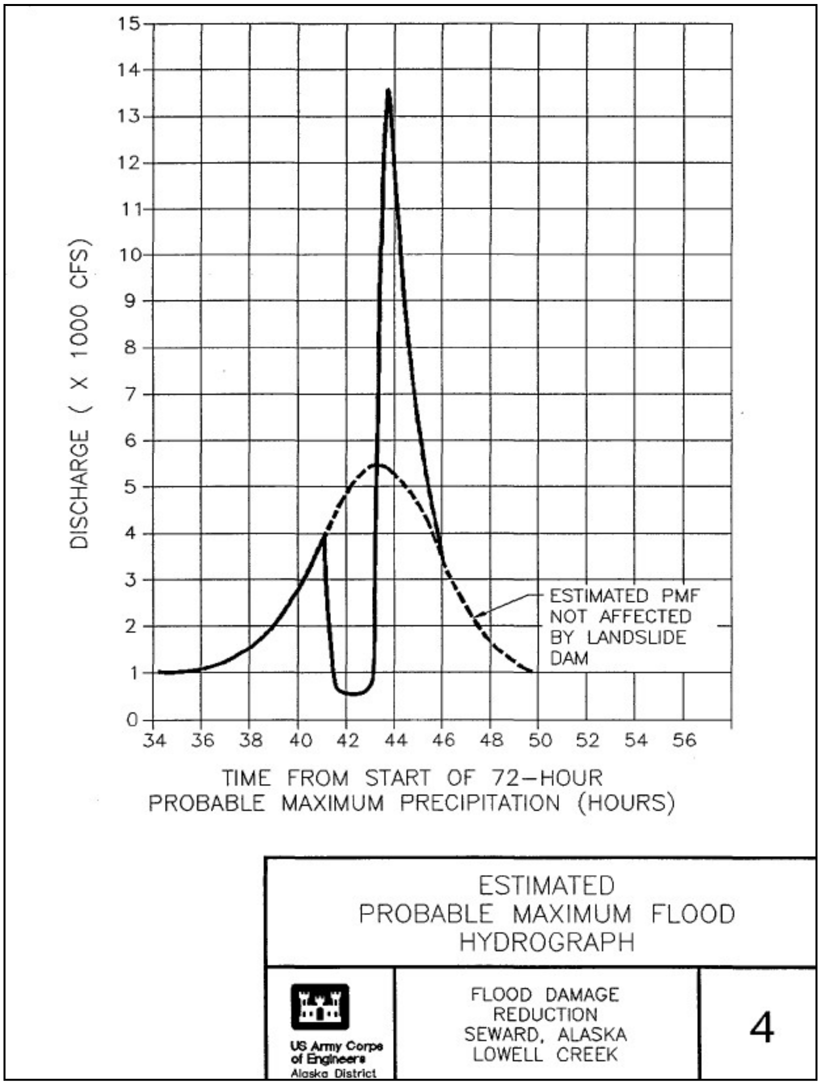

Figure 9. Graph from 1992 Reconnaissance Report (USACE 1992).

3.7.4. Risk Assessment for FRM PMF (2018) from Lowell Creek Inundation Study (January 2012)

In the 1978 report, a 72-hour PMP storm of 27 inches was utilized to develop the PMF for Lowell Creek. This value adjusted to a 24-hour PMP storm using Figure 30 from Hydro- Meteorological Report 54 is approximately 16 inches. The maximum 1-day observed rainfall for Seward is 15.06 inches on October 10, 1986. USGS analysis in 1988 on precipitation in Seward indicates that this October 1986 event was on the order of a 200 to a 500-year precipitation event. Based on this observed precipitation and a comparison to the 24-hour PMP listed for Seward in the National Weather Service Technical Paper No. 47 (TP47; NWS 1963), the 1978 PMP estimate appears to be low. The 24-hour PMP for Seward shown within TP47 is 27 inches before any adjustment for basin elevation and area.

The PMP can also be approximated based on the relationship between the mean annual precipitation (MAP) and the PMP and based on the relationship between the MAP and 100-year, 24-hour rainfall. These relationships are described in Hydrometeorological Report No. 54, “Probable Maximum Precipitation and Snowmelt Criteria for Southeast Alaska” (Schwartz & Miller 1983). These two methods yield a PMP estimate of between 18.9 and 30.6 inches, respectively. For the Alaska District’s 2012 inundation study, a new 24-hour PMP hyetograph was developed based on the methods defined in TP47. Though dated, TP47 provides the only generalized method for developing a PMP estimate for this drainage basin. The resulting 24-hour PMP storm is 27 inches. This hyetograph is shown in Figure 10.

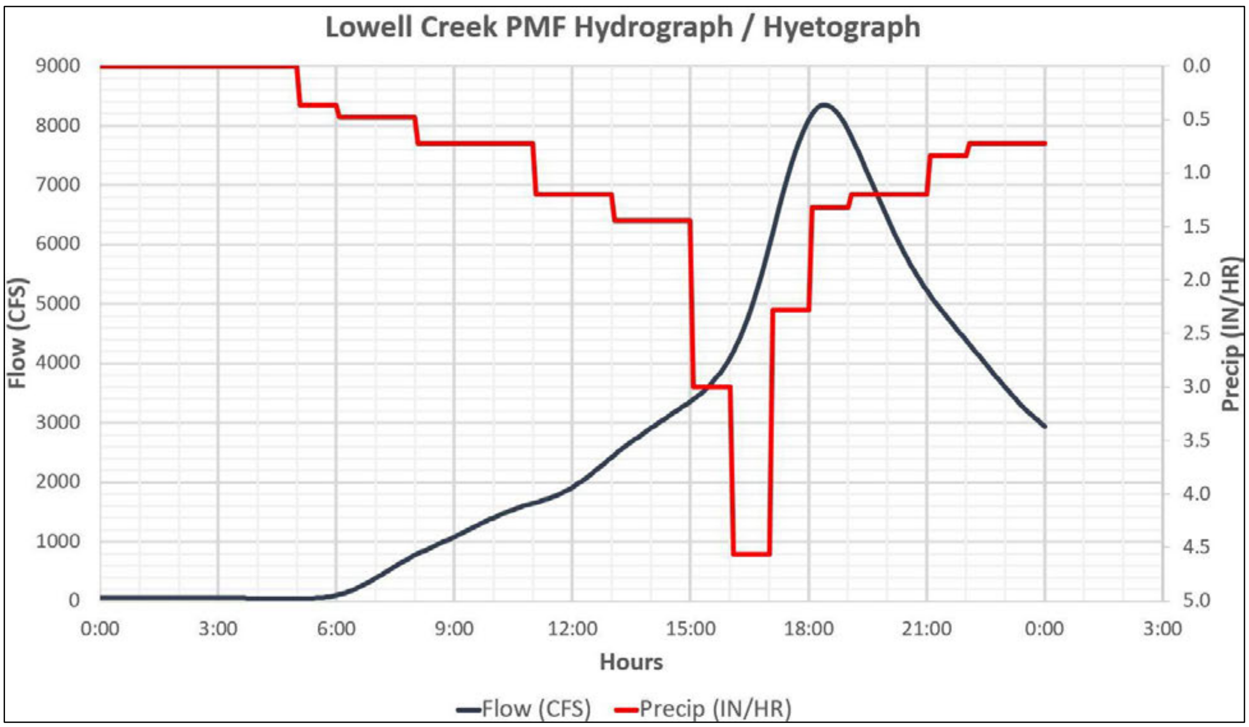

Figure 10. PMP Hyetograph (27 Inches in 24 Hours) and Resulting PMF Hydrograph at the Lowell Creek Tunnel Entrance.

A hydrologic model was used to estimate the PMF for Lowell Creek based on this PMP. The calculated PMF discharge for Lowell Creek upstream from the diversion dam was 7,600 cfs. The PMF was developed using an HEC-HMS (Version 3.5, 2010) model with values for the “Synder" unit hydrograph from the 1978 report (T_p = 2.00 hours and C_p = 0.63) and an initial/constant loss rates of 0.1 and 0.05-inches per-hour, respectively. The 1978 report does not describe how these unit hydrograph parameters were estimated, nor if they were peaked appropriately for PMF analysis. Typically, calibrated unit hydrographs are peaked between 25 and 50% when used for PMF analysis. 10% of the watershed was set as impervious based on the area of glaciers shown on USGS topographic maps. The model routed the event through the Lowell Creek flood control project. The diversion dam was modeled using a series of weirs with each crest set 1 ft higher every 20 ft to account for the 5% grade on the diversion dam. Figure 10 shows the PMP hyetograph and the resulting probable maximum flow hydrograph. Storage volume upstream from the diversion dam was calculated using 2006 LiDAR data for this area. The volume of water impounded when the flow reaches the emergency spillway crest is 3.7 acre-feet. Note that "impounded" is used loosely here due to the nature of the dam as a diversion structure. The 7,600 cfs flow was bulked using a BF of 1.11, yielding a bulked PMF flow of 8,400 cfs. Using the surge release multiplier estimated by the USGS at 2.5 for the 1986 flood on Spruce Creek, the Surge Release PMF maximum flow is estimated to be 19,000 cfs.

3.7.5. Antecedent Conditions

According to Section 8.f of Engineer Regulation (ER) 1110-8-2, an antecedent pool should be assumed to occur before the IDF event. Experience has demonstrated that an unusual sequence of floods can result in filling all or a major portion of the flood control storage in a reservoir immediately before the beginning of the IDF. ER 1110-8-2 states two scenarios to establish the minimum starting pool elevation before the IDF routing:

1. The full flood control pool level.
2. The elevation prevailing five days after the last significant rainfall of a storm that produces half of the IDF hydrograph.

The "more appropriate" of the two starting elevations should be used for the best estimate of adequacy, using engineering judgment.

The lack of a reservoir at the Lowell Creek Diversion Dam makes the antecedent condition of pool elevation immaterial. The antecedent conditions that need to be considered for various flow scenarios are:

- A partially or fully blocked tunnel.
- Tunnel damage leading to reduced flow through increased roughness (note that this condition can be included under the description of a "partially blocked tunnel").

3.7.6. Surge Release Events

The Lowell Creek watershed has been rated by the USGS as having a high potential for landslide induced surge release flooding. A BF may be used to address this issue with the controlling scenario being that of a landslide induced surge release, which would also include sediment/debris-laden flood flows. See the Design Floods section of this chapter for additional discussion of upstream landslide dam, breach induced, surge flow impact on the estimate of the PMF's maximum discharge.

The USGS in 1988 published a comprehensive summary of the Seward area flooding that occurred in 1986 (USGS WRI 87-4278). The following five area streams all had debris blockages upstream that resulted in surge releases during the flood (Figure 11):

- Godwin Creek
- Lost Creek
- Box Canyon Creek
- Japanese Creek
- Spruce Creek

Indirect discharge measurements were performed at Godwin, Lost, and Spruce creeks. Results from these three surge release events were plotted against maximum known flood peaks for other maritime streams in South-central Alaska. The surge release floods are an order-of-magnitude above the envelope curve developed from peak events that do not include surge release flows. Indirect discharge measurements upstream and downstream of the debris blockage on Spruce Creek showed a peak flow 2.5 times greater than would have otherwise occurred, as a result of the debris dam failure and surge release.

The USGS report concluded, based on the geomorphology of Lowell Creek, that there was a high potential for landslide induced surge release flooding on Lowell Creek. Work related to updating the Flood Insurance Rate Maps for the Seward area, completed in 2010, also included adjustments to the 1% chance flood flows to account for surge-release floods as a result of debris dam failures (Northwest Hydraulic Consultants 2007). It was estimated in this 2007 report that these extreme floods increased the 1% chance peak discharge by between 30 to 300% for the various streams analyzed in the Seward area. Lowell Creek was not included in this analysis. The report concluded that for streams where debris dam formation is likely, but no extreme flood observations have been quantified, an increase of 75% is reasonable for the 0.01 AEP flood.

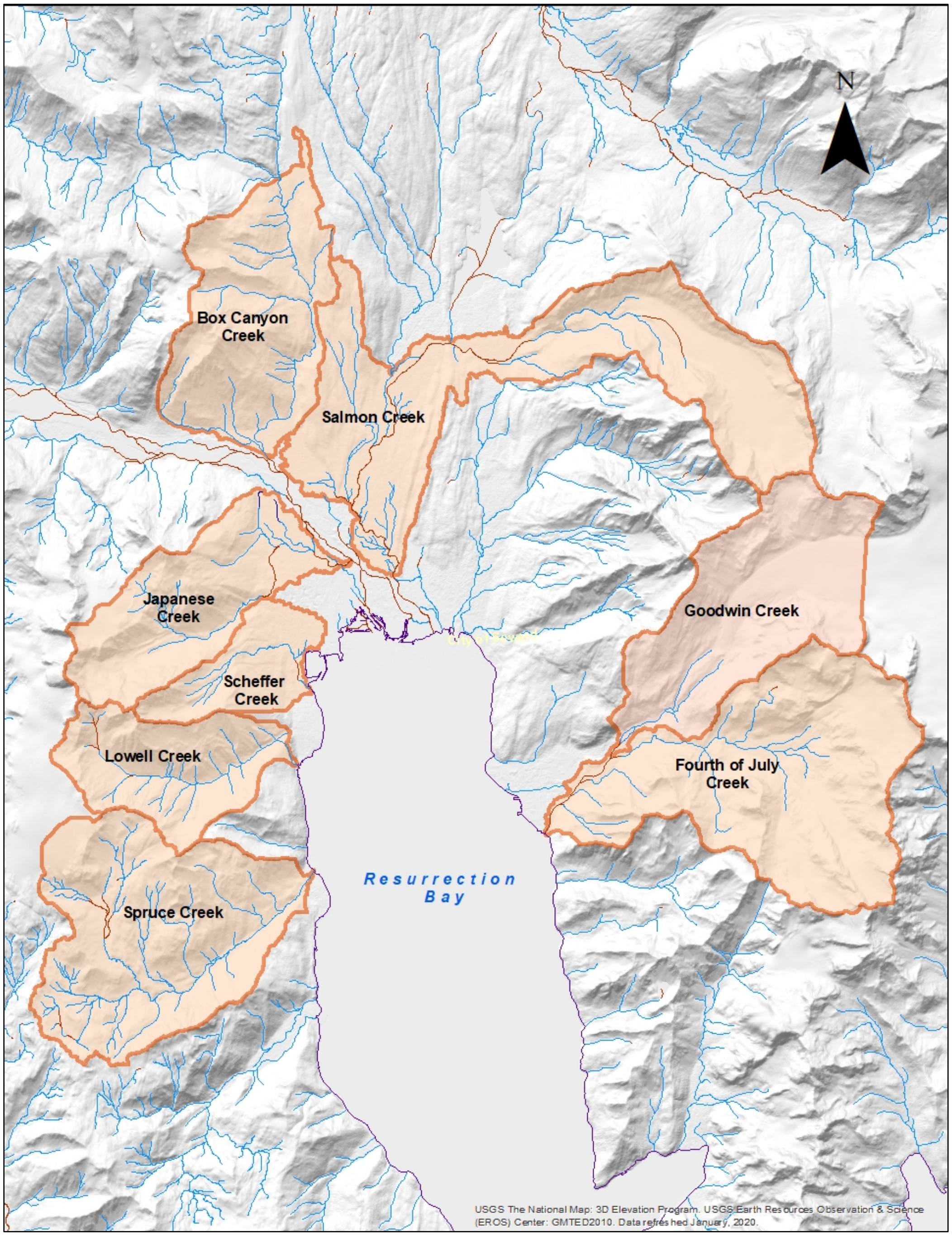

Figure 11. Location of Watersheds in the Seward Area. Godwin Creek is a Tributary of Fourth of July Creek fed by Goodwin Glacier. Lost Creek is a Tributary of Salmon Creek on the West Side of the Basin.

The PMF definition states, “A flood that can be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in a region.” Based on this definition, consideration of debris dam surge release events on Lowell Creek should be included in any PMF determination. The original multiplier of 2.5 used in the USACE 1992 study appears to be a reasonable initial estimate of the surge release adjustment for the PMF discharge based solely on rainfall-runoff. This increase to the estimated flow in alluvial fan systems is a BF to account for uncertainties in the hydrologic data, entrained sediment, and potential surge release floods. BFs are typically used when dealing with alluvial fan flooding problems. These factors are based on watershed characteristics and are developed for a specific region.

Further refinement of the PMF during future work could include the following items below, in order of significance, to reduce uncertainty:

1. Calibration of the rainfall-runoff model and unit hydrograph parameters to confirm the 1978 Snyder unit hydrograph parameters.
2. The geometry of Lowell Creek upstream from the tunnel entrance, combined with debris blockage size estimates and failure characteristics, could be utilized to define further the increase in PMF flows due to a debris dam release.
3. Site specific estimate of the PMP.
4. Refine the surge release multiplier.

Regardless of the level of analysis performed, there will always be more uncertainty in the Lowell Creek PMF estimate than in other watersheds due to both the rainfall/runoff relationship and surge release contributions. The current PMF peak flow estimate with a surge release event is 19,000 cfs.

Based on 43 years of record on the adjacent Spruce Creek watershed and evaluating Lowell Creek based on comparative hydrology, this screening-level hydrologic and hydrology analysis estimates the AEP of the IDF (PMF without surge) ranges from 1/820 to beyond 1/10,000,000 with the best estimate of 1/53,000.

3.8. Climate Change

3.8.1. Climate Change Impacts to Lowell Creek

The analysis of climate change was conducted in accordance with Engineering and Construction Bulletin (ECB) 2018-14, Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects. The publication “Recent US Climate Change and Hydrology Literature Applicable to US Army Corps of Engineers Missions – Water Resources Region 19, Alaska, 2015”, was used in this analysis. The Climate Hydrology Assessment Tool and the Vulnerability Assessment tool described in the ECB was not used because the HUC-4 units that cover Alaska are not included in the tool’s database.

Climate in the project area is projected to change over this century. Temperatures are expected to increase for the Alaska Region, with winters becoming milder, and summer becoming hotter. These effects are projected to be more prevalent in the latter part of the century as opposed to the early part.

According to the Fourth National Climate Assessment (2017, Vol. 1), a warming trend relative to average air temperatures was recorded from 1925 through 1960. A trend of increasing temperatures starting in the 1970s has been identified and is projected to continue throughout the state of Alaska. The largest temperature increases have been found in winter months with average minimum temperature increases of around 2 degrees Fahrenheit statewide.

In the Region 19 Report, a consensus among the peer-reviewed literature emerged that indicates a warming trend for the Alaska Region, especially in the winter and spring seasons. The Region is experiencing warmer average winter temperatures, warmer average annual temperatures and earlier spring onset/longer growing seasons. Extreme cold temperatures have become less frequent while extreme warm temperatures have become more frequent.

The primary potential climate change impacts to the hydrology of Lowell Creek would be changes to precipitation volumes. An increase in 24-hour precipitation would generally increase the frequency flow values for the basin. For most values, the system selected would have capacity to pass the increased inflow. For low frequency (infrequent) events, there would be greater overtopping flow routed through Seward.

Precipitation is expected to increase over the remainder of this century. In the Region 19 Report, there is general agreement of increases in projected annual precipitation, increased occurrence of large rain events, and a corresponding increase of dry days in the Alaska Region. This will result in a projected increase of runoff.

Annual maximum 1-day precipitation is projected to increase by 5%–10% in southeastern Alaska and by more than 15% in the rest of the state, although the longest dry and wet spells are not expected to change over most of the state.

While temperature increases have been observed throughout the state and are projected to continue into the future, snowmelt hydrology does not produce peak stream flow in Lowell Creek and changes to snowmelt should have minimal impact on the effectiveness of the project.

3.8.2. Nonstationary Analysis

According to the Fourth National Climate Assessment (Wuebbles et al. 2017), evidence for changes in maximum gauged streamflows is mixed, with a majority of locations having no significant trend. There is the significance for seasonal changes in the timing of peak flows in interior Alaska, though increases in the absolute magnitude are not well evident in existing data.

To investigate whether a trend of changing peak annual flow is occurring in the Lowell Creek Watershed, the Spruce Creek gage record was tested using the Nonstationary Detection Tool in accordance with Engineering Technical Letter 1110-2-3 (Figure 12). The gage record includes peak annual stream flow from 1966 to 2009, excluding 1986, which is a 43-year period of record. The gage captures a drainage area of 9.3 square miles and is located 0.7 miles upstream from the mouth of Spruce Creek at Resurrection Bay and 2.4 miles south of Seward.

The tool notes a discontinuity in the data set, which corresponds with the absence of data for 1986. The 1986 rainfall event produced a high flow record in the data set of 13,600 cfs for water year 1987 while the average peak stream flow observed over the period of record is 2,035 cfs with a standard deviation of 1,945 cfs and a variance of 3,782,208 cfs^2. Monotonic trend analysis of this period did not detect a statistically significant trend using the Mann- Kendall Test at a 0.5 level of significance (exact p-value of 0.721) or using the Spearman Rank Order Test at the 0.5 level of significance (p-value of 0.754). No trends were detected using parametric statistical methods or Sen's Slope method. No nonstationarities or monotonic trends are detected within the streamflow record recorded on Spruce Creek.

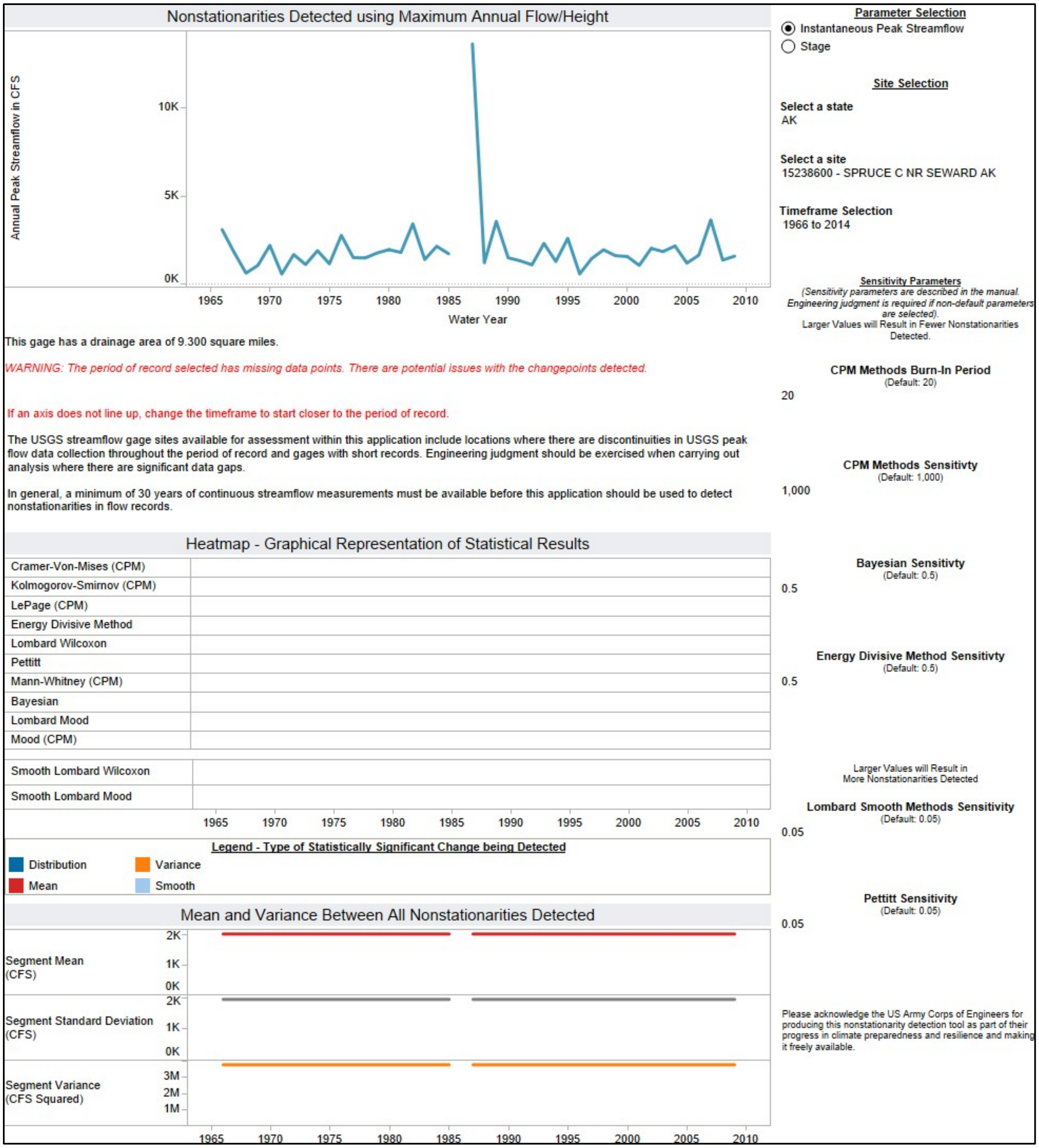

Figure 12. Nonstationary Detection Tool Results.

3.8.3. Climate Risks

ECB 2018-14 requires the evaluation of the risk climate change poses to the project features. Table 4 illustrates the features under consideration in this project and how they may be affected by climate change.

Table 4. Climate Change Risk.

Feature	Trigger	Hazard	Harm	Qualitative Likelihood
New Diversion Dam	Increases in the frequency and magnitude of precipitation (storms larger and more intense)	Increases in flood discharge and frequency	Increased possibility of overtopping	Low
New Tunnel			Increased possibility of exceeding tunnel capacity	
Tunnel Inlet Portal			Increased possibility of Structural failure	
Extended Outfall			Increased possibility of exceeding outfall capacity	
Refurbished Tunnel			Increased possibility of exceeding tunnel capacity	

Project features are designed to accommodate the Probable Maximum Flood (PMF) event. The annual exceedence probability (AEP) associated with the PMF is 0.002% (1/50,000-yr event). Thus, the project is designed to withstand extreme conditions. Despite there being evidence that the region is warming, and extreme precipitation events are becoming more intense and frequent, there is no evidence in the literature reviewed or the observed record indicating that streamflow magnitudes will change as a result of changes to meteorologic response in the project's life cycle. Thus, the likelihood of climate change impacting project performance is low.

3.8.4. Sea Level Change (SLC)

USACE requires that planning studies and engineering designs consider alternatives that are formulated and evaluated for the entire range of possible future rates of sea level change (SLC). Designs must be evaluated over the project life cycle and include evaluations for the three scenarios of “low,” “intermediate,” and “high” SLC. According to ER 1100-2-8162 (USACE 2019a) and Engineer Technical Letter 1100-2-1(USACE 2019b), the SLC “low” rate is the historic SLC. The “intermediate” and “high” rates are computed by:

- Estimating the “intermediate” rate of local mean SLC using the modified National Research Council (NRC) Curve I, the NRC equations, and correcting for the local rate of vertical land movement (VLM).

- Estimating the "high" rate of local mean SLC using the modified NRC Curve III, NRC equations, and correcting for the local rate of VLM. This "high" rate exceeds the upper bounds of the Intergovernmental Panel on Climate Change (IPCC) estimates from both 2001 (IPCC 2001) and 2007 (IPCC 2007) to accommodate the potential rapid loss of ice from Antarctica and Greenland.

The 1987 NRC described these three scenarios using the following equation:

$$\boldsymbol{E(t) = 0.0012t + bt^2} \qquad \text{Equation 1}$$

in which *t* represents years, starting in 1986, *b* is a constant, and *E(t)* is the eustatic SLC, in meters, as a function of *t*. The NRC committee recommended, "projections be updated approximately every decade to incorporate additional data." At the time the NRC report was prepared, the estimate of global mean sea-level (GMSL) change was approximately 1.2 mm/year. Using the current estimate of 1.7 mm/year for GMSL change, as presented by the IPCC (IPCC 2007), results in this equation being modified to be:

$$\boldsymbol{E(t) = 0.017t + bt^2} \qquad \text{Equation 2}$$

The three scenarios proposed by the NRC result in global eustatic SLR values (by the year 2100) of 0.5 meters, 1.0 meters, and 1.5 meters. Adjusting the equation to include the historic GMSL change rate of 1.7 mm/year and the start date of 1992 (which corresponds to the midpoint of the current National Tidal Datum Epoch of 1983-2001), results in updated values for the variable b being equal to 2.71E-5 for modified NRC Curve I, 7.00E-5 for modified NRC Curve II, and 1.13E-4 for modified NRC Curve III.

Manipulating the equation to account for it being developed for eustatic SLR starting in 1992, while projects will be constructed at some date after 1992, results in the following equation:

$$\boldsymbol{E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2)} \qquad \text{Equation 3}$$

where t_1 is the time between the project's construction date and 1992 and t_2 is the time between a future date at which one wants an estimate for SLC and 1992 (or $t_2 = t_1$ + the number of years after construction). Using the three *b* scenarios required by ER 1100-2-8162 (USACE 2019) results in the three GMSL rise scenarios.

An analysis of the potential SLR was performed in the project area. The gage at Seward, Alaska (National Oceanic and Atmospheric Administration (NOAA) ID:9455090) was used for the analysis. This gage was established in 1925 and is in its present location since 1989. It is located on the Alaska Railroad Pier, inside the Cruise Ship Terminal building. This location was input into the USACE SLC Calculator (Version

2019.21). The result of the calculation indicates a relative SLC of 3.71 ft was determined in the year 2100 at the high condition. For the intermediate condition, the change is 0.42 ft, and the low condition shows a decrease in sea level of 0.62 ft. These values are relative to Local Mean Sea Level (LMSL) as the calculator states NAVD88 datum is not available for this station. The resulting SLC curve is shown in Figure 13. The calculator also outputs a table showing the progression of SLC. This table was derived in 10-year increments and is shown in Table 5. The calculator also provides the expected SLR across several datums. These datums and their respective values are shown in Table 6 and Figure 14.

Figure 13. Relative Sea Level Change in feet.

Table 5. Sea Level Change by Year.

YEAR	USACE		
	Low	Int	High
1992	0.00	0.00	0.00
2002	-0.06	-0.05	-0.02
2012	-0.11	-0.08	0.03
2022	-0.17	-0.09	0.16
2032	-0.23	-0.09	0.37
2042	-0.28	-0.06	0.64
2052	-0.34	-0.02	0.99
2062	-0.40	0.04	1.42
2072	-0.46	0.11	1.92
2082	-0.51	0.21	2.49
2092	-0.57	0.32	3.14
2100	-0.62	0.42	3.71

Note: Lowell Creek
9455090 - Seward, AK
NOAA 2006 Published Rate: -0.00571 ft/yr.
All Values are expressed in ft relative to LMSL

Table 6. Sea Level Change and Extreme Water Level by Datum.

Datum/EWL	Reference Datum		
	LMSL	MLLW	NAVD88
Highest Astronomical Tide:	8.23	13.79	13.56
Mean Higher High Water:	5.07	10.63	10.4
Mean High Water:	4.15	9.71	9.48
Mean Sea Level:	0	5.56	5.33
Mean Low Water:	-4.18	1.38	1.15
MLLW:	-5.56	0	-0.23
NAVD88:	-	0.23	0
EWL Type:	NOAA Generalized Extreme Value		
*100 Yr.:	10.02	15.58	15.35
*50 Yr.:	9.84	15.4	15.17
20 Yr.:	9.58	15.14	14.91
10 Yr.:	9.36	14.92	14.69
5 Yr.:	9.1	14.66	14.43
2 Yr.:	8.67	14.23	14
Yearly:	7.8	13.36	13.13
Monthly:	-	-	-
From:	1964		
To:	2007		
Years of Record	43		

Note: According to the benchmark data sheets, 0 ft MLLW = -0.23 ft NAVD88, so in the table above, NAVD88 is equivalent to -5.33 ft Mean Sea Level.
EWL = Extreme Water Level; LMSL = Local Mean Sea Level; MLLW = Mean Lower Low Water

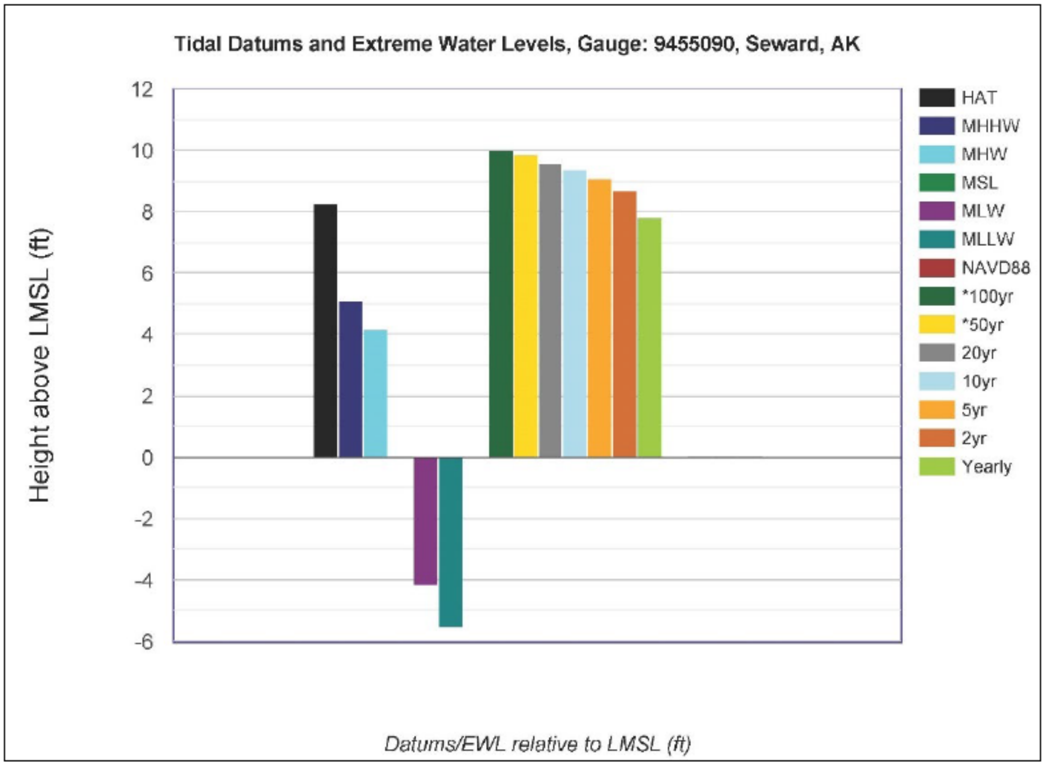

Figure 14. Tidal Datums and Extreme Water Levels relative to LMSL.

In 2011, NOAA published a datasheet for the gage at Seward (Station ID 9455090; NOAA 2011). A reference to the datasheet is included at the end of this Appendix. This datasheet established a relationship between NAVD88 and Mean Lower Low Water (MLLW). This relationship was NAVD88 = MLLW – 0.23 ft. The benchmark is documented with NOAA's Online Positioning User Service, PID BBFH75, and Designation 5090 B 1978 (NOAA 2020). A link to the site for this benchmark is included at the end of this Appendix. Table 6 shows the relationship between MLLW, LMSL, and NAVD88. The highest tide level occurred in January 1987 and was 15.70 MLLW (15.41 ft NAVD88). With the potential SLR of 3.71 ft, this max tide level would be 19.41 MLLW (19.18 NAVD88).

An analysis of SLC was performed for the project area. The terrain model for the project was used to determine if SLC had any detrimental effect on the project. The terrain model is based on the NAVD88 datum. The USACE SLC Calculator was used to determine the estimated SLC in the year 2100.

To determine the potential SLC effect on the project, a comparison of the sea level elevation and specific project features was made. The outfall elevation of Alternative 4A is 59.7 ft NAVD88. This elevation is similar for all alternatives analyzed. Ground elevation below the outfalls is at about +20 feet MLLW. Operations to clear this area continue to be practical under the high SLC scenario in year 2100 indicating that the functionality of the recommended plan is not sensitive to sea level change.

Another comparison was made with the structures and infrastructure in the project area. Ballaine Blvd. and Railway Ave. run parallel to the shoreline. Elevations along Ballaine Blvd. range from 24–30 ft NAVD88, and Railway Ave. range from 20–24 ft NAVD88. The majority of structures are located above the roads and are therefore higher in elevation.

Located between the above-mentioned roads and the ocean are a few properties. The Recreation South Campground is located along Ballaine Blvd. This campground contains mostly transient campers with some seasonal sites. The lowest ground elevation in the campground is about 16 ft NAVD88. Along Railway Ave., several commercial structures are in place, notably the Iditarod Campground and the Alaska Sea Life Center. Low ground elevations in the campground area are about 17.5 ft NAVD88, and at the Sea Life Center are about 21 ft NAVD88.

An additional examination was made using the Coastal Assessment Regional Scenario Working Group (CARSWG) site. The task of the working group was to develop localized adjustments leading to different future SLC and Extreme Water Level scenarios to support vulnerability and impact assessment for Department of Defense (DoD) coastal and tidally influenced sites. One such site is located near the project area.

The Seward Recreation Annex is located on Hwy 9, about 2 miles north of Lowell Canyon Rd. While the facility is not within the project limits, it is included in the CARSWG site listing and was examined in this analysis. The result of the CARSWG listing is shown in Figure 15.

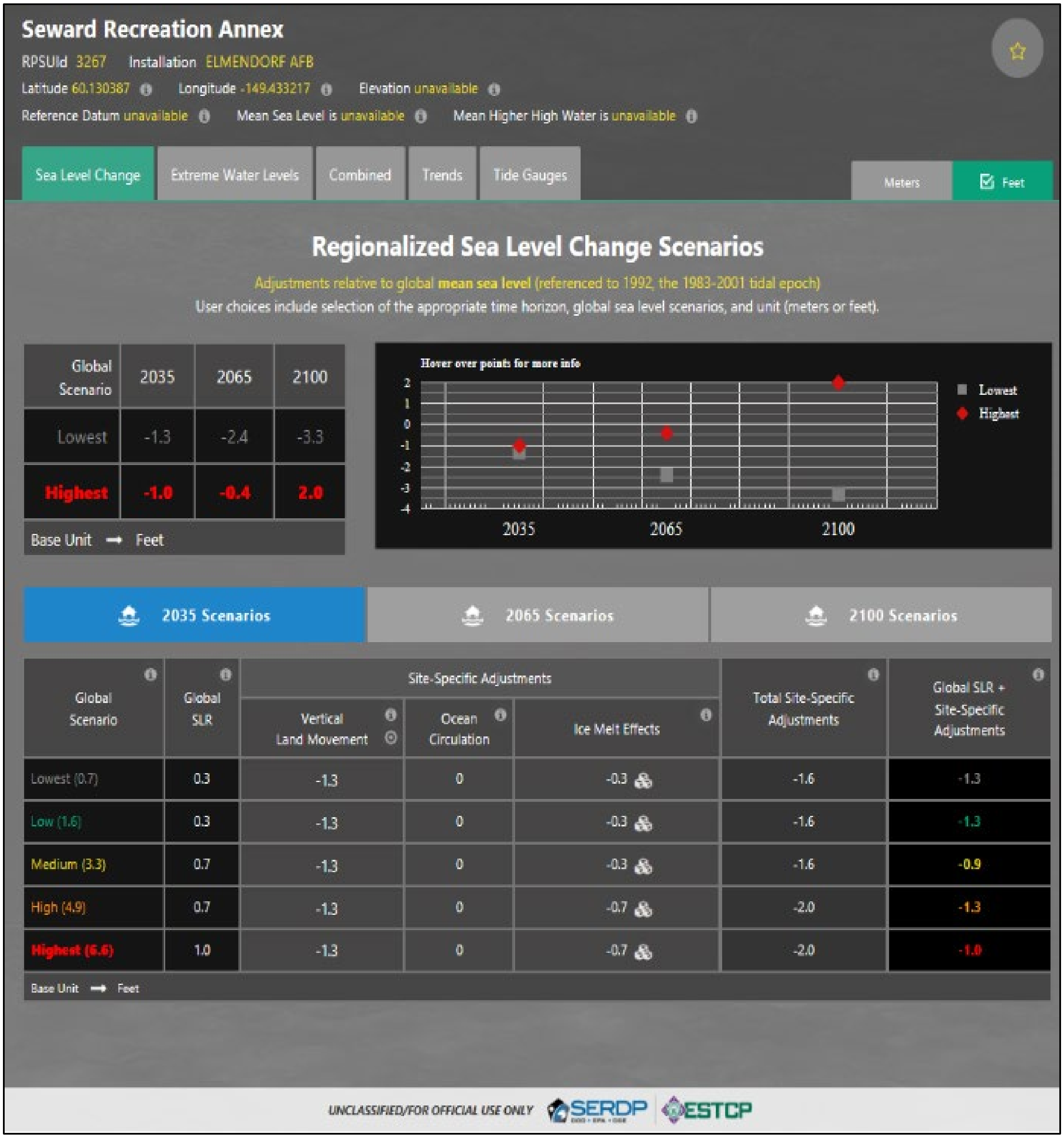

Figure 15. Seward Recreation Annex – (CARSWG).

The CARSWG analysis shows a net SLC of 2.0 ft in the year 2100 as compared to the 1992 baseline year for the highest global scenario. The global SLC at the year 2100 is estimated to be 6.6 ft,, which is less than the SLC estimated in accordance with ER 1110-2-8162. This SLC is tempered by site-specific adjustments based on several variables. Vertical land movement is the change in elevation in the earth's crust caused by subsidence, tectonic movement, isostatic rebound, etc. Isostatic rebound is the gradual, upward movement of land mass resulting from the retreat of large ice mass.

The VLM is estimated to be an upward trend to 3.0 ft in 2100. The melting of glaciers and large ice sheets result in an additional upward trend in land movement as the loss of ice mass reduces the pressure on the land mass. This effect is estimated to cause an upward movement in land elevation of 1.6 ft in the year 2100. The total site-specific adjustments for the Recreation Annex are estimated to be 4.6 ft. When compared to the global SLR of 6.6 ft, the net adjusted SLR is 2.0 ft. The terrain model for the project does not extend to the Recreation Annex. Elevation values for the ground at the Annex were extracted from Google Earth for this comparison. Ground elevations are in excess of 30 ft within the Annex property. They are above any expected SLC.

Based upon the SLC calculator and existing ground elevations, the City of Seward may be at risk of coastal floods in its lowest areas, such as the campgrounds. This increasing risk will reduce, in those areas, the marginal benefit of reducing river flood risk via the tunnel project.

4. ALTERNATIVE DESIGN CRITERIA

4.1. Hydraulic Event Scenarios

Alternative development for the study considered several cases to assess the performance of the existing project and proposed alternatives. These scenarios are divided into two categories, rainfall-runoff events with and without a landslide and subsequent surge release.

4.2. Tunnel Capacity Calculations

The discharge capacity of the tunnels investigated was calculated using an open flow equation, assuming that a free water surface would exist through the majority of the tunnel. While higher pressure flows may be theoretically possible through the tunnels, it was assumed that, under these scenarios, the tunnel lining would be damaged to the point where the integrity of the tunnel would be compromised. Diversion dams were designed for open channel flow tunnel conditions so that water at the tunnel inlet could reach this point of full capacity, then spill over the dam to minimize the potential for tunnel damage to occur under high flow. Under these conditions, the existing tunnel is considered to have a capacity of 2,800 cfs.

The new 18-ft-diameter tunnel being considered in alternative plans is considered to have a capacity of 8,500 cfs, which is based on the existing tunnel entrance invert to spillway crest height. This combination has been chosen based on it passing the non-surge, PMF peak discharge, and minimizing the size of the intake transition and diversion dam. Cost analysis indicates that larger intake transitions/dams are relatively more expensive than larger tunnels. Similarly, the new 20-foot diameter tunnel under consideration is considered to have a capacity of 19,000 cfs, which is based on raising the spillway crest an additional 19.5 ft from the tunnel entrance invert to spillway crest height. This combination was chosen based on it passing the surge-based PMF and minimizing the size of the intake transition and diversion dam.

The various combinations of tunnel diameter and spillway height configurations that have been evaluated are shown in Table 7.

Table 7. Tunnel Flow Capacities.

(A) Tunnel Diameter (ft)	(B) Capacity based on matching existing tunnel-inlet invert to spillway crest height (cfs)	(C) Approx. depth of flow and percent of diameter away from the tunnel entrance (ft / %)	(D) Capacity based on raising spillway crest higher than existing condition (cfs)	(E) Approx. depth of flow and percent of diameter away from the tunnel entrance (ft / %)	(F) Amount spillway crest must be raised to achieve column (D) stated capacity (ft)
10	2,800	7.8 / 78	2,800	7.8 / 78	0.0
12	4,100	8.6 / 72	4,500	9.3 / 78	6.0
14	5,500	9.2 / 66	5,800	9.6 / 68	3.5
16	7,000	9.7 / 61	7,600	10.3 / 64	5.5
18	8,500	10.1 / 56	14,000	14.7 / 82	42.5
20	10,000	10.4 / 52	19,000	16.8 / 84	58.0
22	11,500	10.7 / 49	19,000	14.8 / 67	34.0
24	14,000	11.4 / 48	19,000	13.8 / 58	19.5

5. ALTERNATIVE PLANS

A final array of alternative plans was formulated and evaluated for effectiveness. This array of plans consists of:

1. No-Action
2. Improve Existing Flood Diversion System
3. Enlarge Existing Flood Diversion System (3A 18 ft and 3B 24 ft)
4. Construct New Flood Diversion System (4A 18 ft and 4B 24 ft)
5. Construct Debris Retention Basin
6. Floodplain Relocation

5.1. Alternative 1: No-Action

The no-action alternative maintains the existing project in its current state and has no change to downstream risk or consequences.

5.2. Alternative 2: Improve Existing Flood Diversion System

This alternative includes the following structural measures: refurbish existing tunnel; extend tunnel outlet 150 ft to shelter existing road; protect tunnel inlet from landslide blockage, and to improve low flow diversion system. Nonstructural measures include the implementation of an early warning system and evacuation plan; and remove trees. Hydraulically, Alternative 1 and Alternative 2 are identical. Numerical analysis of overflow for Alternative 2 also represents the effects and consequences of the no-action alternative. A site plan for Alternative 2 is displayed in Figure 16.

5.3. Alternative 3A: Enlarge Existing Flood Diversion System – 18 ft

This alternative consists of the following structural measures: enlarge existing tunnel to an 18-ft-diameter horseshoe; replace the existing intake transition and diversion dam; extend tunnel outlet 150 ft to shelter the existing road; protect tunnel inlet from landslide blockage, and improve low flow diversion system. Nonstructural measures include implementing an early warning system and evacuation plan and removing trees. Increasing the tunnel diameter to 18 ft produces a tunnel capacity of approximately 8,500 cfs. With greater tunnel capacity, greater flow can be diverted from Seward, and as a result, the frequency and magnitude of overtopping flows are reduced.

The frequency and magnitude of overtopping consequences are also reduced. This alternative is negatively impacted by the necessity to perform the majority of the work during the short winter construction season as the tunnel must remain operational during the summer and fall flood seasons. A site plan for Alternative 3A is displayed in Figure 17.

5.4. Alternative 3B: Enlarge Existing Flood Diversion System – 24 ft

This alternative consists of the following structural measures: enlarge existing tunnel to a 24-ft-diameter horseshoe; replace the existing intake transition and diversion dam; extend tunnel outlet 150 ft to shelter the existing road; protect tunnel inlet from landslide blockage, and improve low flow diversion system. Nonstructural measures include implement an early warning system and evacuation plan; and remove trees. Increasing the tunnel diameter to 24 ft produces a tunnel capacity of approximately 19,000 cfs with the new diversion dam and associated spillway sized to provide this tunnel flow capacity. With greater tunnel capacity, all anticipated flows can be diverted from Seward, and as a result, the frequency and magnitude of overtopping flows are eliminated. The frequency and magnitude of overtopping consequences are also eliminated. This alternative is negatively impacted by the necessity to perform the majority of the work during the short winter construction season as the tunnel must remain operational during the summer and fall flood seasons. A site plan for Alternative 3B is displayed in Figure 17.

5.5. Alternative 4A: Construct New Flood Diversion System – 18 ft

This alternative consists of constructing a new 18-ft tunnel and diversion dam upstream of the existing diversion dam and tunnel. The new tunnel would have a capacity of 8,500 cfs, and the existing tunnel would have a capacity of 2,800 cfs.

Surface flow from Lowell Creek would be diverted through the new tunnel. Should an event occur that exceeds the new tunnel capacity, flow overtopping the new diversion dam would be intercepted by the existing dam and routed through the existing tunnel. The combined tunnel capacity of this alternative is 11,300 cfs. The provision of having two operating tunnels improves the efficiency of maintenance operations. When the upstream tunnel needs to be repaired, flow can be diverted through the upstream diversion dam to the downstream tunnel. Since the capacity of the tunnel is far greater than the existing diversion drain under Jefferson Street, maintenance operations can be conducted during the summer months. Being able to divert typical summer flows greatly increases the amount of time available to perform tunnel maintenance and greatly improves working conditions. A site plan for Alternative 4A is displayed in Figure 18.

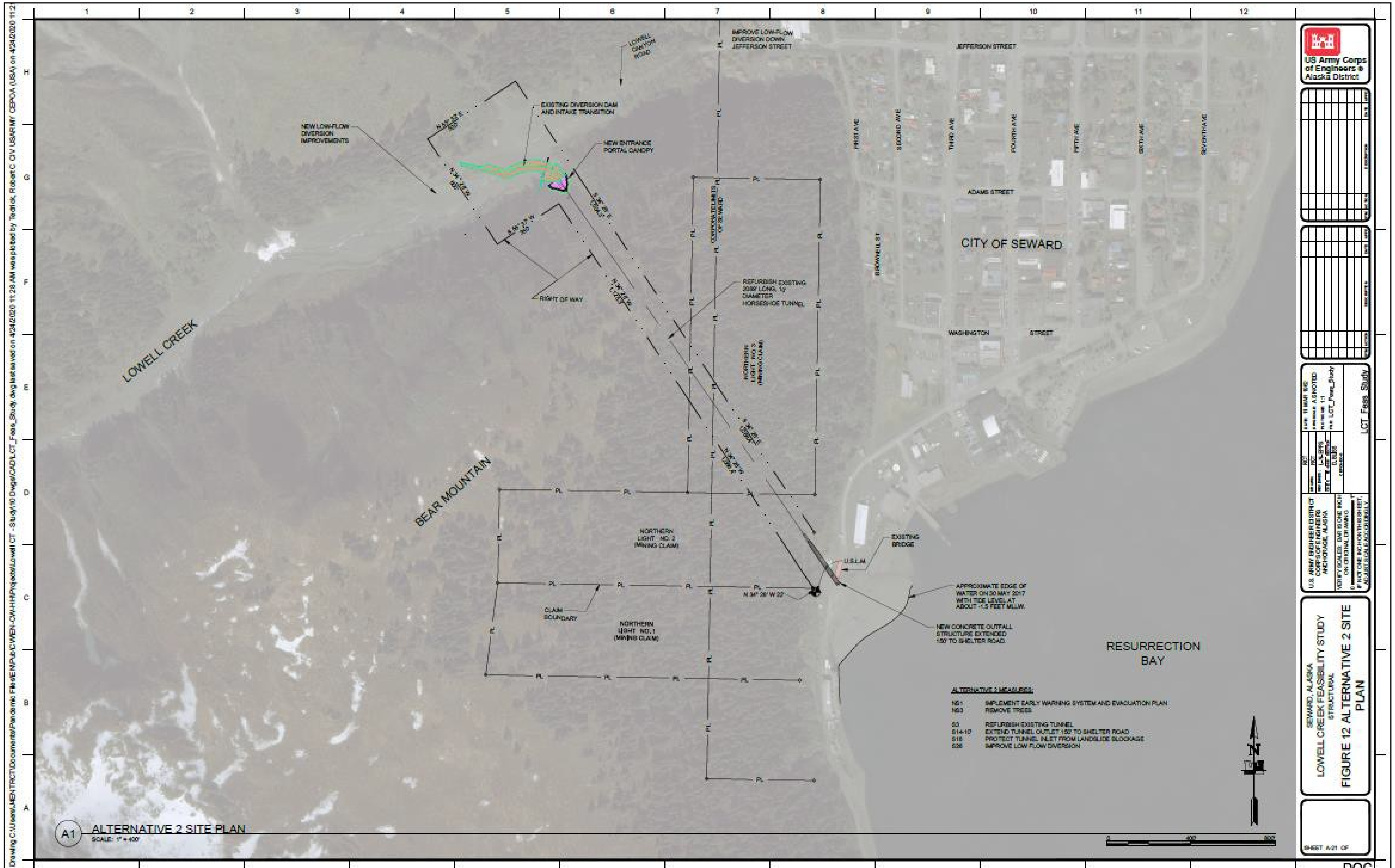

Figure 16. Alternative 2 Site Plan.

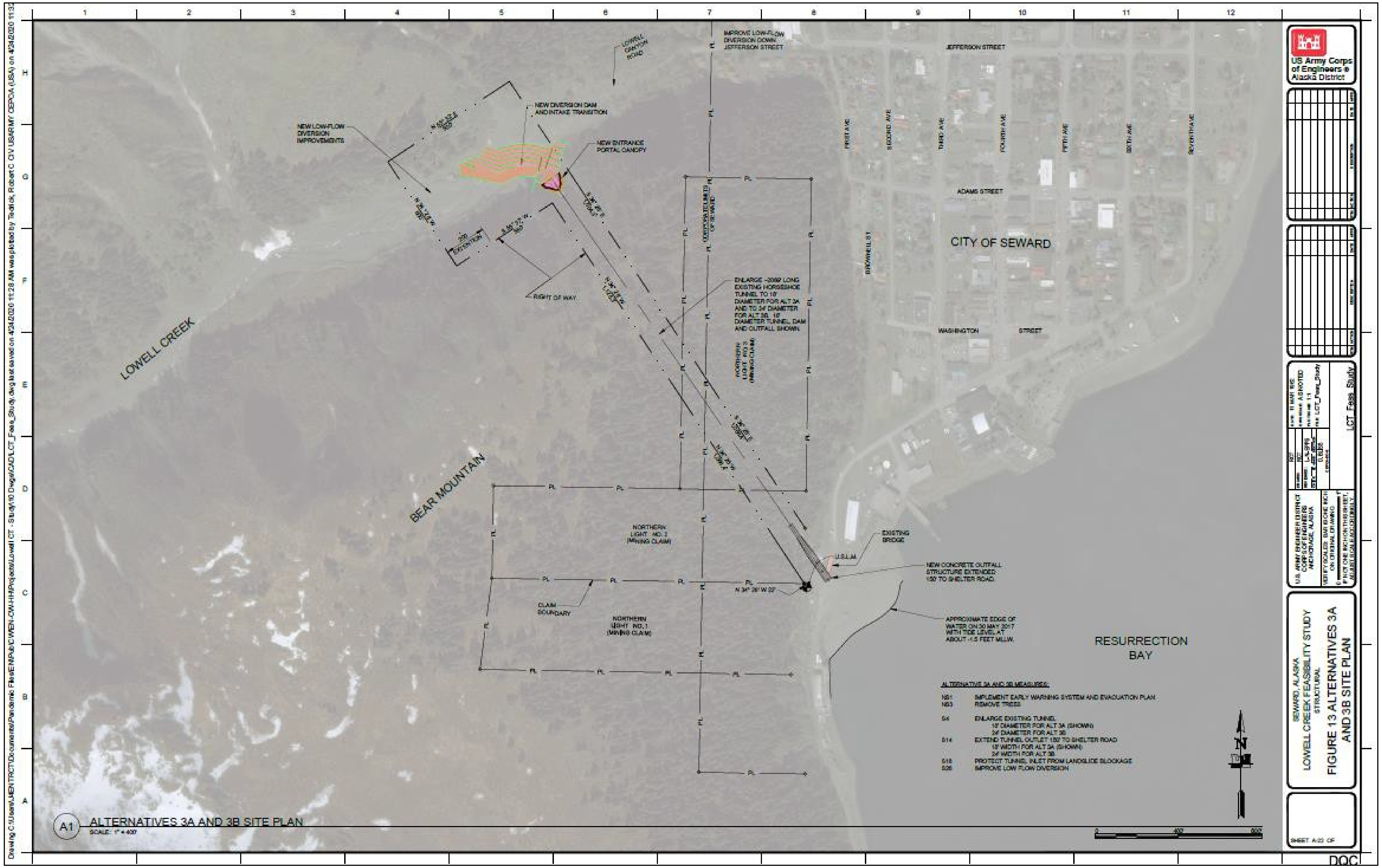

Figure 17. Alternatives 3A and 3B Site Plan.

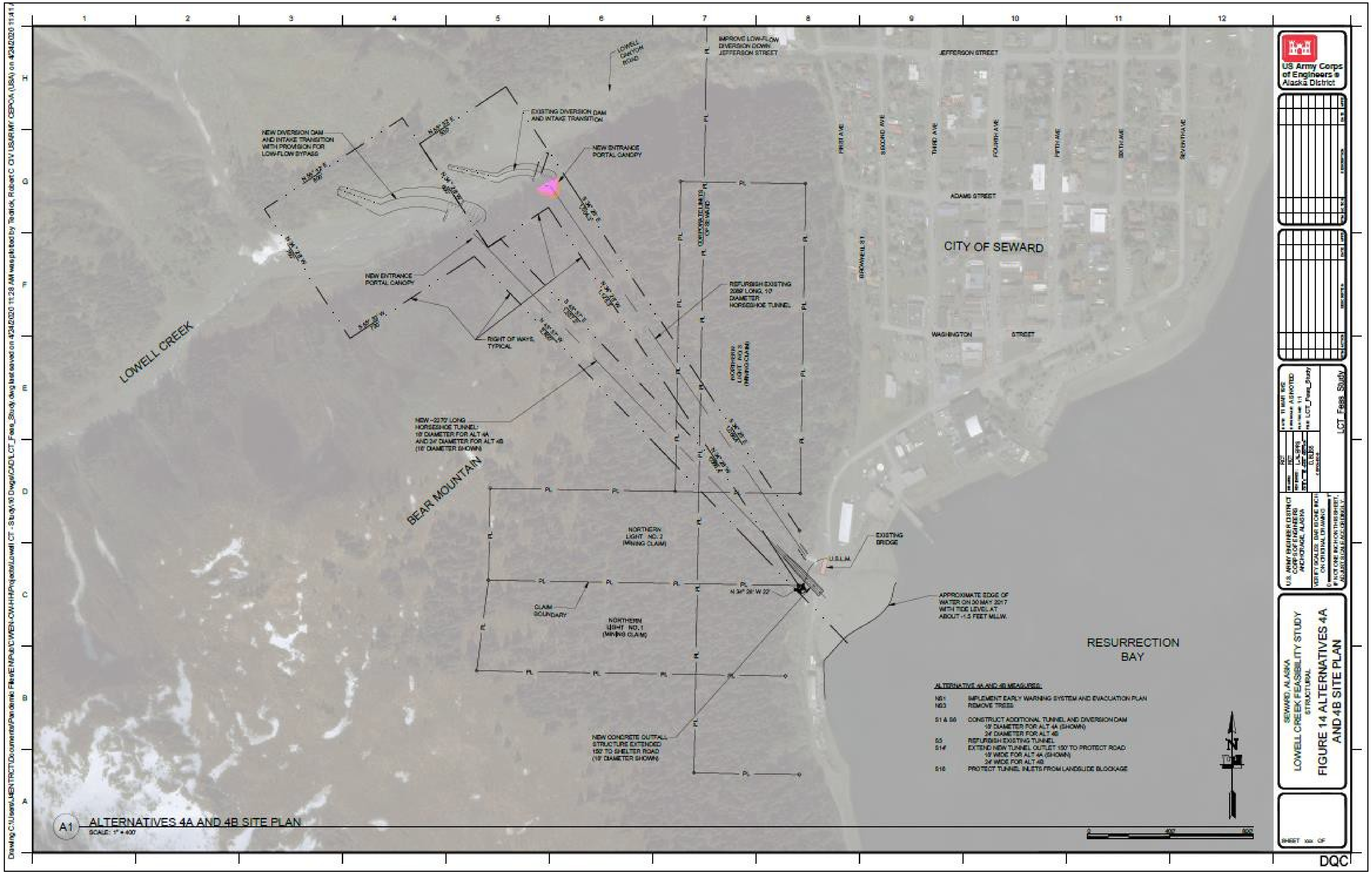

Figure 18. Alternatives 4A and 4B Site Plan.

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5.6. Alternative 4B: Construct New Flood Diversion System – 24 ft

This alternative consists of constructing a new 24-ft tunnel and diversion dam upstream of the existing diversion dam and tunnel. The new tunnel would have a capacity of 19,000 cfs, and the existing tunnel would have a capacity of 2,800 cfs. Surface flow from Lowell Creek would be diverted through the new tunnel. Should an event occur that exceeds the new tunnel capacity, flow overtopping the new diversion dam would be intercepted by the existing dam and routed through the existing tunnel. The flow frequency analysis indicates that the new tunnel would be able to pass all anticipated flood flows. The combined tunnel capacity of this alternative is 21,800 cfs. The provision of having two operating tunnels improves the efficiency of maintenance operations. When the upstream tunnel needs to be repaired, flow can be diverted through the upstream diversion dam to the downstream tunnel. Since the capacity of the tunnel is far greater than the existing diversion drain under Jefferson Street, maintenance operations can be conducted during the summer months. Being able to divert typical summer flows greatly increases the amount of time available to perform tunnel maintenance and greatly improves working conditions. A site plan for Alternative 4B is displayed in Figure 18 above.

5.7. Alternative 5: Construct Debris Retention Basin

This alternative consists of constructing a detention basin upstream of the Lowell Creek Tunnel inlet to retain debris before it enters the tunnel and accumulates at the outlet requiring flood fighting activities (e.g., Figure 19). The concept of this alternative is to maintain a volume capacity upstream of the tunnel capable of containing the volume of debris anticipated for frequent flood-fighting events. The purpose of this alternative description is to prevent or reduce sediment transport during an event so that downstream debris management requirements and associated debris induced damages can be reduced or eliminated.

Figure 19. Debris Accumulation at the Tunnel Outlet in 2006 and Flood Fighting Activities.

Based on discussions with the Seward Department of Public Works, debris buildup at the tunnel outlet occurs when rainfall exceeds 3 inches in a 24-hour period measured at the airport. This was estimated to occur 4 to 6 times per year. No numerical analysis of the frequency of debris volumes or the relationship between rainfall intensity and duration and debris volumes has been performed. A cursory evaluation of the volumetric growth of the alluvial fan at the tunnel outfalls shows a rough average volumetric change of 25,000 cy per year. Several large debris accumulations have occurred, notably, the large debris buildup that occurred in 1986 that buried the bridge at the outfall in an estimated 10-ft depth of material.

5.7.1. Basin

A structure would be required upstream of the tunnel to intercept debris, and this debris would then have to be hauled out of the basin. Debris movement upstream of the existing diversion dam has not been studied due to the lack of available information and instrumentation for measurement. No effective plan could be justified to manage debris by removal of material without a structure to intercept it and contain it before removal. This alternative calls for a roller-compacted concrete structure to be constructed approximately 700 ft upstream of the existing tunnel entrance to intercept debris before it passes through the tunnel (Figure 20). The structure is designed to create a 25,000 cy detention volume where debris, mostly sand and gravel with cobbles and some boulders, can accumulate and be hauled out after rain events. The structure is approximately 200 ft in length, with a crest approximately 15 ft above the canyon floor. The upstream embankment face would be constructed at a 1H:1V slope, and the downstream face would be constructed at a 2H:1V slope, similar to the existing diversion dam (Figure 21). The entire embankment would be constructed of roller-compacted concrete.

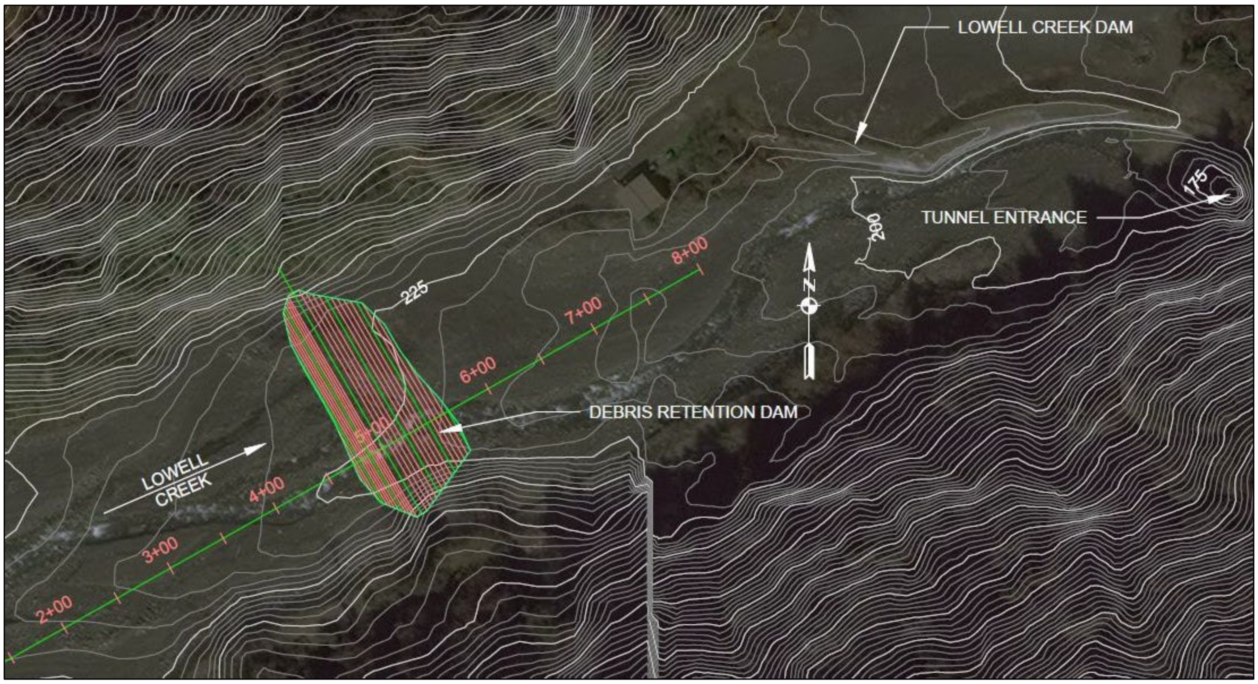

Figure 20. Plan View of the Debris Retention Structure.

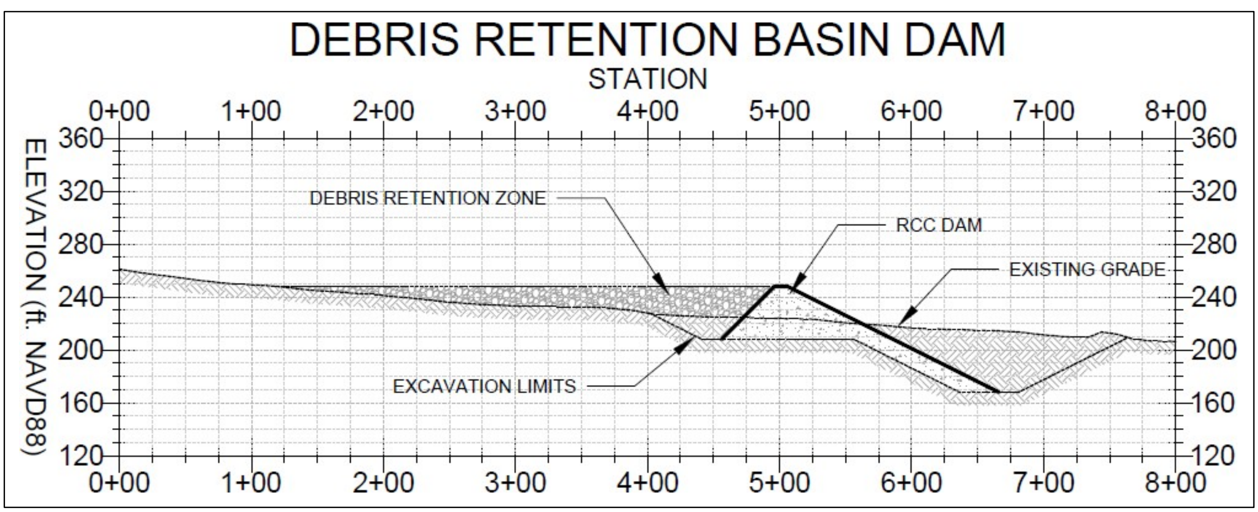

Figure 21. Cross-Section of the Debris Retention Structure.

While the plan would only be effective at intercepting debris from small events, the structure would need to be designed to survive larger, less frequent events. It was assumed that the upstream toe would need to be constructed to 20 ft below the existing canyon floor and the downstream toe constructed to 40 ft below the canyon floor to prevent scour and head-cutting under the embankment foundations. The downstream embankment face extending below the base elevation would be constructed in 30-ft-wide lifts, creating approximately a 13-ft overall thickness.

5.7.2. Operations

This alternative requires considerable maintenance to operate. The debris retention basin must be cleared to design capacity after all rainfall events that move material upstream of this basin to maintain effectiveness. It is not known how fast a debris basin would fill under weather conditions that produce less than 3 inches of rain in 24 hours. There may be an effective base flow of debris down the canyon that is not accounted for between flood fight events. All of this material would also be intercepted and require removal for this plan to remain effective. Removal costs for this process have not been accounted for.

5.7.3. Prescriptive Costs

A formal concept design and cost estimate of this alternative was not performed. Cursory investigation shows that the cost to construct and maintain this alternative exceeds likely benefits to be attained. Construction of the debris detention structure would require excavation of approximately 90,000 cy of material from the canyon floor and placement of approximately 35,000 cy of roller-compacted concrete to construct the embankment. Prescriptive unit costs for excavation and removal of material from the site are $10 per cy. A preliminary estimate for roller compacted concrete construction is $450 per cy, leading to a rough construction cost estimate of about $16.7 million for this alternative. Details such as preparing canyon wall surfaces and providing access for equipment to move debris from the detention basin downstream have not been considered. They would add to the cost of construction. This cost also does not include typical costs associated with construction, such as mobilization, establishing a field office, and supervisory labor to direct the work.

Operational costs are based on the need to remove material for the basin to maintain the capacity to intercept debris before it reaches the tunnel. A cursory evaluation of the volumetric growth of the alluvial fan at the tunnel outfall shows a rough average volumetric change of 25,000 cy per year. The debris basin has been assumed to operate with 50% efficiency, leading to an accumulation of 12,500 cy per year. At $10 per cy to remove this material, it would cost $125,000 per year to maintain the functionality of this feature of the project. The additional 12,500 cy per year would be handled at the tunnel outfall, roughly cutting the current operational costs at the outfall in half.

The frequency of debris loads has not been evaluated; however, it is a fair assumption that some years could see little debris movement in the basin, and some years could see more than has been assumed. No attempt has been made to quantify the frequency which this basin would be overfilled, and debris would flow past the basin and route through the tunnel to the outfall. This cost does not include provisions to maintain equipment access to the basin after rainfall events.

5.7.4. Residual Risk

While this alternative provides an alternate method for handling debris volumes associated with Lowell Creek, functionally, the project has no significant impact on downstream risk. Since the basin is sized for smaller, high-frequency events, the capacity to intercept debris is likely to become quickly overwhelmed during larger infrequent events. Overtopping hydrographs investigated for Alternative 2 would also represent the risks associated with this alternative.

5.8. Alternative 6: Seward Floodway

Several plans for evacuating a floodway through Seward were studied; a partial evacuation with the construction of a contained floodway to prevent overflow and debris from damaging remaining structures on the Lowell Creek alluvial fan, and several variants of relocating structures within the confines of Lowell Creek Canyon to a location outside of the alluvial fan. Relocation alternatives to remove structures from the canyon are discussed in the Integrated Feasibility Report and Environmental Assessment main report.

5.8.1. Alternative 6A: Floodway Through Town

This alternative designates a floodway across the Lowell Creek alluvial fan to be contained by dikes to prevent damage and life safety risk to the remainder of the developed area. In the event of significant overtopping flow discharging to the Lowell Creek alluvial fan (Seward), concentrated flow would place structures in the canyon west of First Street at significant risk. Beyond this point, mobilization of debris will cause randomly shifting flow paths and accumulation of debris which also poses a risk to the remainder of the floodplain. To mitigate life loss risk and potential structural collapse during significant overtopping events, relocation of infrastructure within a defined floodway through the developed area of Seward was considered.

The plan includes relocating all structures south of Madison Street and north of Adams Street. The area to be relocated is approximately 82 acres. The area is composed of a mix of residential, commercial, and public structures including the hospital, City Hall, the public library and Resurrection Bay Historic Society, the Public Works Department, and the Kenai Fjords National Park visitors center.

The floodway was designed to be 750 ft wide which is estimated to flow 2–3 ft deep during a 19,000 cfs event. Containment of the floodway will require the construction of 4,200 ft of new dikes armored on the floodway side to protect the remaining developed areas. A highway bridge would be constructed across the floodway with sufficient overhead clearance in the floodway for equipment to manage debris loads. Approximately 9 acres of land outside the floodway would need to be acquired for construction of the bridge.

In Figure 22, the red lines represent the floodway containment dikes to prevent overflow to the remaining developed areas on the alluvial fan. The yellow line is a highway bridge to allow traffic to cross the floodway. Yellow zones show areas that need to be acquired for bridge construction.

Figure 22: Floodway Between Adams and Madison Street.

6. NUMERICAL MODEL STUDIES

The effectiveness of the suite of alternatives was investigated using a HEC-RAS model of Seward. The purpose of this modeling effort was to analyze downstream consequences from flow overtopping the diversion dam. The model domain extended from Lowell Creek Canyon downstream of the diversion dam to tidewater at Resurrection Bay and included the alluvial fan on which downtown Seward is built. The model does not attempt to route discharge through the tunnel under consideration. Instead, the tunnel discharge was removed from the inflow hydrograph leaving only the water that would flow over the crest of the dam and onto the alluvial fan. Tunnel discharges were developed as described in Section 4.2. The model also does not consider local changes in velocity near the dam. It was deemed to be an unnecessary refinement of the model for the intended purpose.

6.1. Elevation Data

Elevation data for the model grid is from a LiDAR survey of Seward collected in 2008. The horizontal datum for the survey is Alaska State Plane Zone 4, U.S. survey feet, and the vertical datum is NAVD88, U.S. survey feet. The survey extent covers the City of Seward from tidewater up Lowell Creek Canyon to just above the diversion dam for the tunnel. The canyon upstream of the dam is not included in the survey data.

6.2. Model Domain

The model domain covers the alluvial fan of Lowell Creek downstream of the diversion dam (Figure 23). This area was defined as a 2D area in HEC-RAS. The 2D area allows the model to determine the flow path over the alluvial fan where a 1D model would have followed an arbitrary flow path. The upstream boundary of the model grid is located approximately 250 ft downstream from the existing diversion dam. Hydraulic processes upstream of the dam and overtopping of the dam are not included in the model. The model extends to the tidewater coastline of Seward. Figure 23 presents the Lowell Creek HEC-RAS Model Domain Extents.

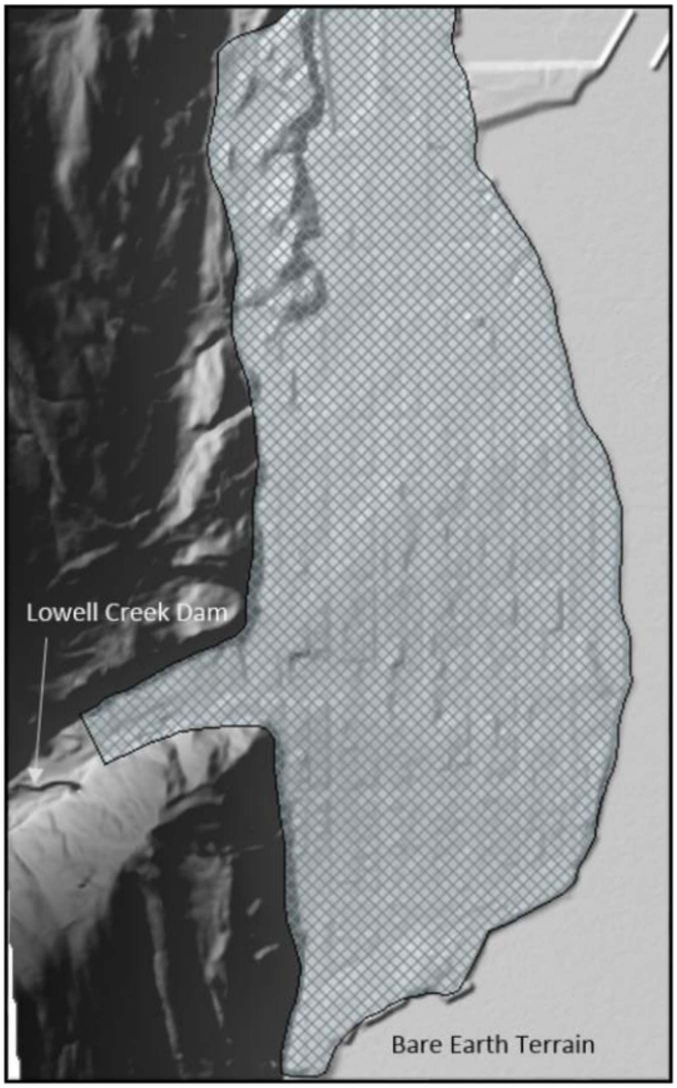

Figure 23. Lowell Creek HEC-RAS Model Domain Extents.

The limitation of the 2D area is that the model geometry is fixed. During an overtopping event, it is expected that velocities will be sufficient to mobilize debris and objects within Seward, which would create blockages to flow path and redirect flow to other parts of the alluvial fan. Also, if the tunnel is blocked, the full debris load of the flow would be delivered to the alluvial fan, which would also result in flow path uncertainty. This process is beyond the capability of HEC-RAS to predict. While blockage scenarios could be assessed by altering the terrain within the 2D area to represent blockages, these blockages would be arbitrary. All model runs were performed as clear water flow with no blockage.

6.2.1. Terrain and Roughness

The 2D area was based on a 25-ft square grid using the bare earth terrain model. Building footprints were created from a shapefile, with the mesh resolution increased around the buildings to a 10-ft spacing to allow for computations along the faces of the buildings (Figure 24). A total of 68 buildings, mostly along Jefferson Street and to the south, were modeled in this fashion.

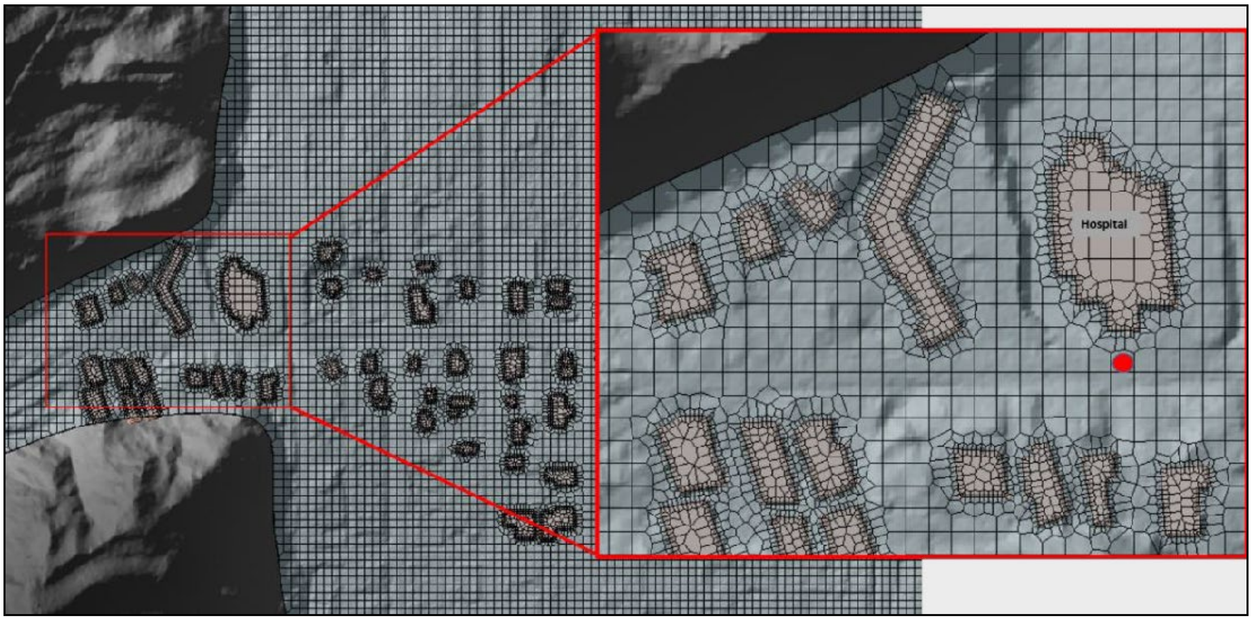

Figure 24. Grid Improvements at Buildings.

The 2D area was assigned a global Manning's n value of 0.1. Areas within the digitized building footprints were assigned a Manning's n value of 5.0. The high n-value was used to reduce velocity to nearly zero, thus simulating building inundation. The effect of this change in roughness was to constrain flow between buildings, thus increasing velocity in these areas (Figure 25). Altering the Manning's n values in this way is considered to be more representative of expected conditions than allowing free flow over the building footprints.

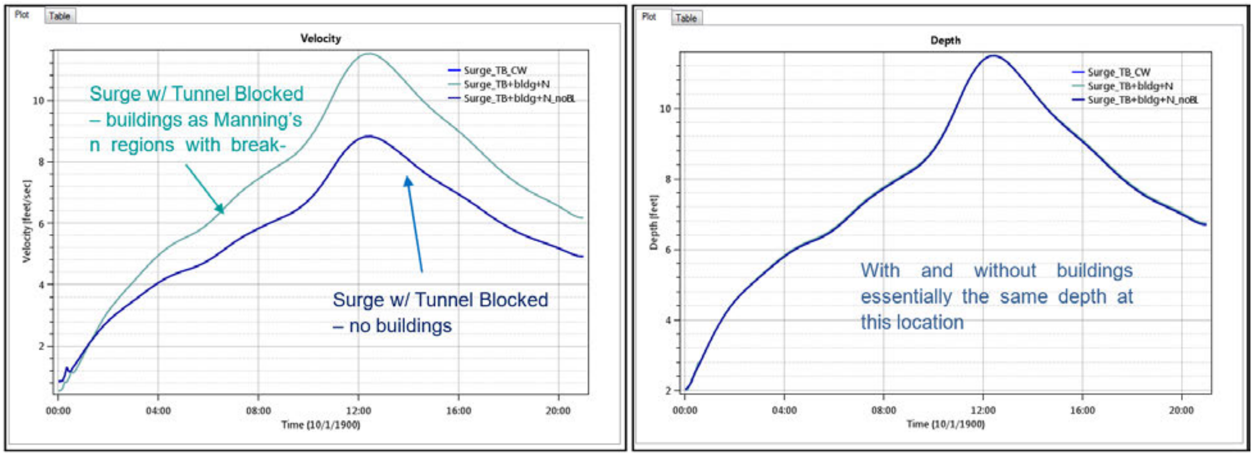

Figure 25. Velocity and Depth Comparison of Test Runs of the Model With and Without Buildings. (Output Results Are from the Red Dot in Figure 24).

6.2.2. Simulation Options and Tolerances

Initial modeling options in HEC-RAS were left to their default values. During the modeling process, some settings were modified to optimize run times due to the large number of scenarios to analyze and compare. Notable changes are that the number of iterations for the 2D area was reduced from the default of 20 to 10 to reduce simulation run times. Model simulations were run using the diffusion wave equation set. The difference in modeling results due to the change on parameters were on the order of a couple of tenths of a foot in water surface elevation, and a couple of feet per second in velocity. These were judged to be not significant.

6.2.3. Boundary Conditions

The upstream boundary condition is an overflow hydrograph based on the PMF hydrograph. The location of flow is downstream of the dam, so the process of dam overtopping and changes in the flow regime as overflow transitions from the downstream face of the dam to the natural grade of the canyon were not included in the model. Hydrographs for events smaller than the PMF were scaled by multiplying the entire hydrograph time series to the ratio of the event peak flow to the PMF peak flow. The hydrographs were then truncated by subtracting tunnel capacity based on event scenarios. All remaining flow was then developed as an overflow hydrograph and applied to the upstream boundary of the model. The upstream hydrographs are described in further detail in Section 6.3. Boundary conditions for modeling purposes will be referred to as overflow hydrographs.

The downstream boundary condition is tidewater; flows passing through the model domain reach the ocean. Due to the relatively steep terrain of the alluvial fan, tidal effects are considered to be negligible.

6.2.4. Surge Flow Routing

A water-budget approach was used such that total flow volume was not changed by the surge event. The inflow hydrographs were modified for the scenarios with surge flows as follows. Time-steps of 5-minutes were used. The total duration of the surge-related flows was assumed to be completed within 45-minutes of the occurrence of the upstream landslide. After the landslide, the first inflow value was assumed to be approximately 10% of the clear-water (not bulked by 1.11) inflow that would have otherwise happened at that time. This inflow was doubled, then tripled for the next two 5-minute time steps. At the 20-minute mark (4th time step), the inflow was calculated as the sum of the inflow that would have happened at this time step, without the surge event, plus the sum of the storage related flows at the landslide dam for all other surge-related time steps.

Storage related flow for each time step was calculated as outflow minus inflow at the landslide dam. Using this sum of the storage related flows assures that the water-budget approach is balanced. At the 25-minute mark (5^{th} time step), the inflow was 2.5 times the peak clear-water inflow, which is the peak of the surge-related flow. A linear transition was used from this peak inflow value to the inflow value at the 45-minute mark (9^{th} time step). This method does not account for the increase in volume of total flow contributed by the landslide debris that is carried by the surge-related flows.

6.3. Event Scenarios

Four categories of event scenarios were analyzed with the HEC-RAS model to look at downstream impacts. The scenario categories included the occurrence of tunnel blockages or surge releases. Multiple hydrographs were modeled for each scenario. The four scenarios are:

a. No Tunnel Blockage, No Surge Release
b. Tunnel Blockage, No Surge Release
c. No Tunnel Blockage, Surge Release
d. Tunnel Blockage, Surge Release

An array of frequency flows was used to analyze the system. The inflow conditions analyzed are shown in Table 8. The probability of a surge release has not been assessed in Table 8. The AEP shown for each flow condition is based on the probability of peak clear water flow. Consequently, separate values for bulked flow and surge release are designated the same probability of occurrence based on peak inflow. Only bulked flow and surge release flows were modeled; clear water flow values are shown for reference.

Table 8. Modeled Event Peak Inflow as Clear Water Flow, Bulked Flow, and Surge Release.

Peak Inflow			
Inflow AEP.	Clear Water Flow (cfs)	Bulked Flow (cfs)	Surge Release (cfs)
1.35E-01	1,500	1,700	3,800
2.75E-02	2,300	2,600	5,800
8.27E-04	4,500	5,000	11,300
1.52E-04	5,800	6,400	14,500
1.89E-05	7,600	8,400	19,000

The AEP of surge events was evaluated by multiplying the hydrologic frequency by the notal probabilities associated with surge release events. This has the effect of decreasing the frequency of surge release events an order of magnitude when compared to the event driving hydrologic frequency. Discussions of how surge release event frequency was incorporated into evaluation of project consequences can be found in the Appendix H: Risk Assessment and Appendix D: Economics.

The array of event permutations modeled for each alternative are shown above in Table 6 (Alternative 2), Table 7 (Alternative 3A), and Table 8 (Alternative 4A). Scenarios where the maximum spillway flow is 0 were not modeled as there was no overflow onto the alluvial fan.

6.3.1. Scenario a. No Tunnel Blockage, No Surge Release

Scenario a. is based on the PMF hydrograph without surge, as described in this Appendix. The PMF hydrograph flow values are scaled based on the peak flow value for the various AEP peak flows. For these scenarios, the tunnel or tunnels under consideration are assumed to bypass water up to the tunnel capacity. At this point, additional flow overtops the spillway/dam and flows down the remainder of the original stream channel paralleling Lowell Canyon Road and out onto the alluvial fan. Overflow hydrographs were created (Figure 26). The inflow hydrograph (blue line) was truncated by subtracting all flow up to the tunnel capacity (red line), resulting in overtopping flow (purple line). For Alternative 4A, where a second tunnel routes discharge, the second tunnel capacity is also subtracted (green line). Scenario a. only causes overtopping flow for Alternative 2, which uses the existing tunnel capacity. Alternatives 3A and 4A pass all inflow through the tunnel(s) in this scenario, so there are no Scenario a. overtopping flows for Alternatives 3A and 4A.

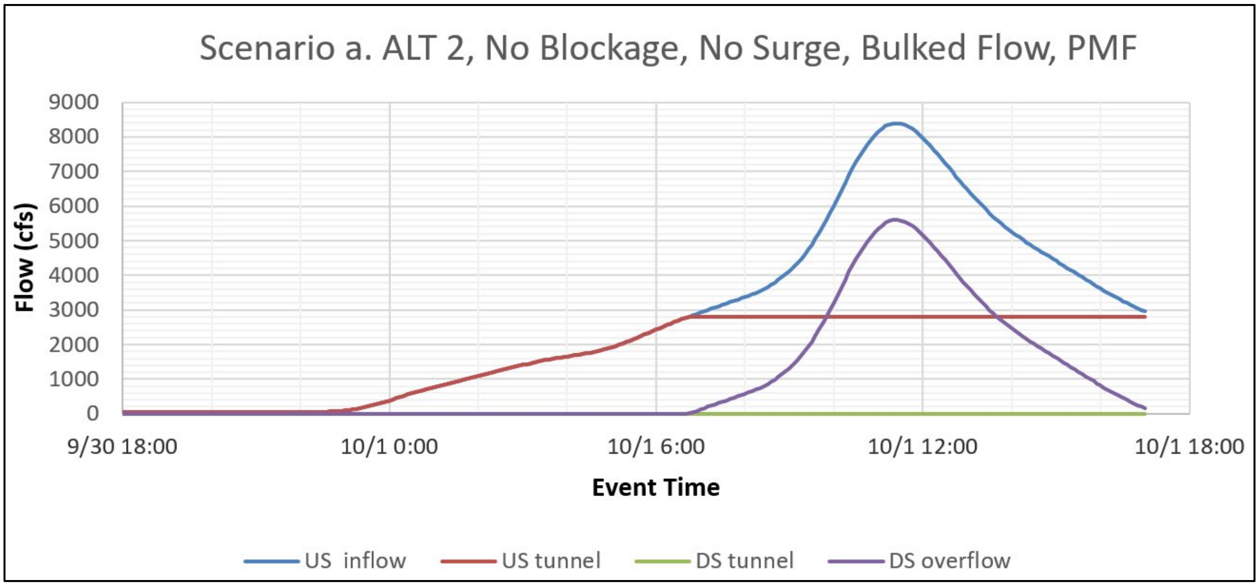

Figure 26. PMF Overflow Hydrograph for Scenario a., Alternative 2.

6.3.2. Scenario b. Tunnel Blockage, No Surge

Scenario b. is based on the PMF hydrograph without surge, as described in this Appendix. The PMF hydrograph flow values are scaled based on the peak flow value for the various AEP peak flows. The tunnel or tunnels under consideration are assumed to bypass water to the tunnel capacity until a blockage occurs. The blockage was modeled to occur at the peak flow of the hydrograph, or the flow when water begins to flow over the spillway when the tunnel reaches capacity. At this point, all flow overtops the spillway/dam and flows down the remainder of the original stream channel paralleling Lowell Canyon Road and out onto the alluvial fan. For Alternative 4A, a blockage is only assumed to occur on the upstream tunnel. Overflow hydrographs were created, as shown in Figure 27 (Alternative 2), Figure 28 (Alternative 3A), and Figure 29 (Alternative 4A). The inflow hydrograph (blue line) was truncated by subtracting all flow up to the tunnel capacity (red line), resulting in overtopping flow (purple line). For Alternative 4A, where a second tunnel routes discharge, the second tunnel capacity is also subtracted (green line).

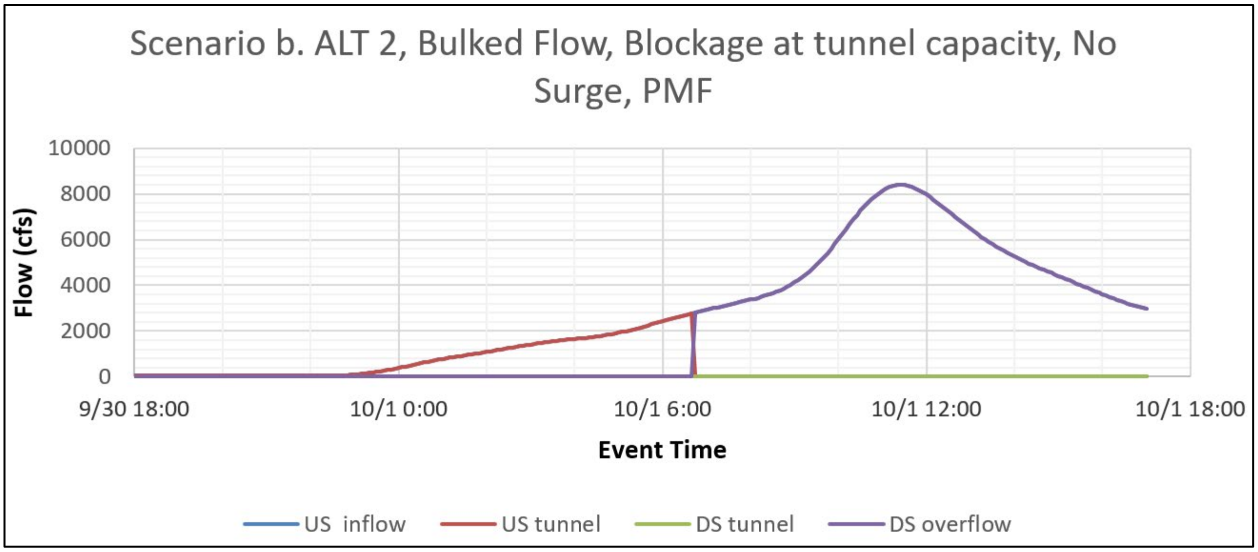

Figure 27. PMF Overflow Hydrograph for Scenario b., Alternative 2.

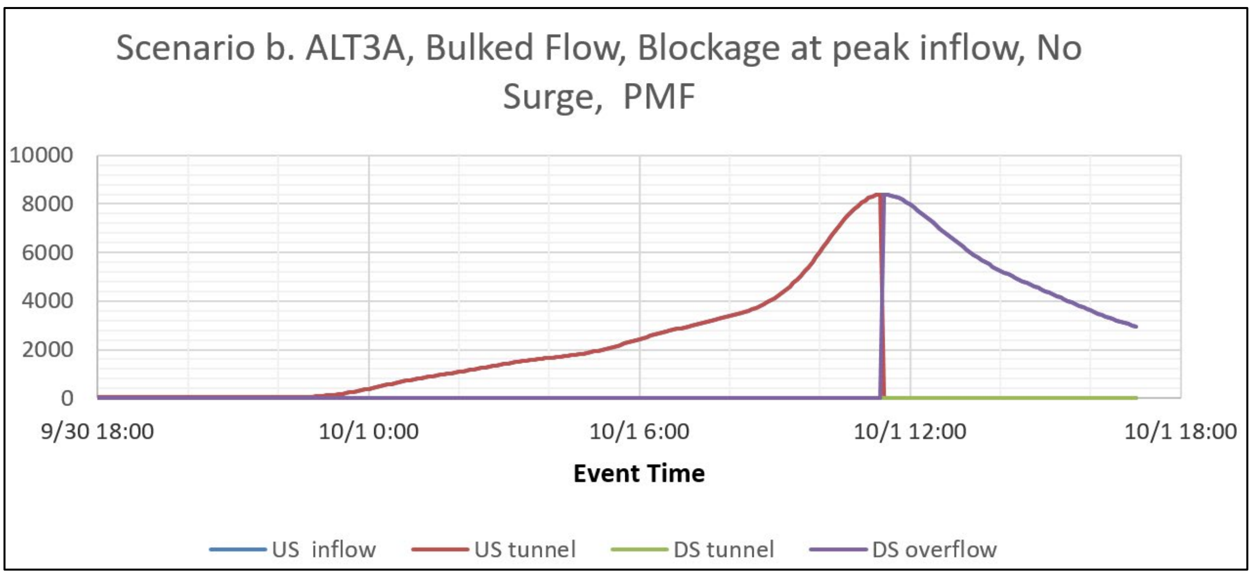

Figure 28. PMF Overflow Hydrograph for Scenario b., Alternative 3A.

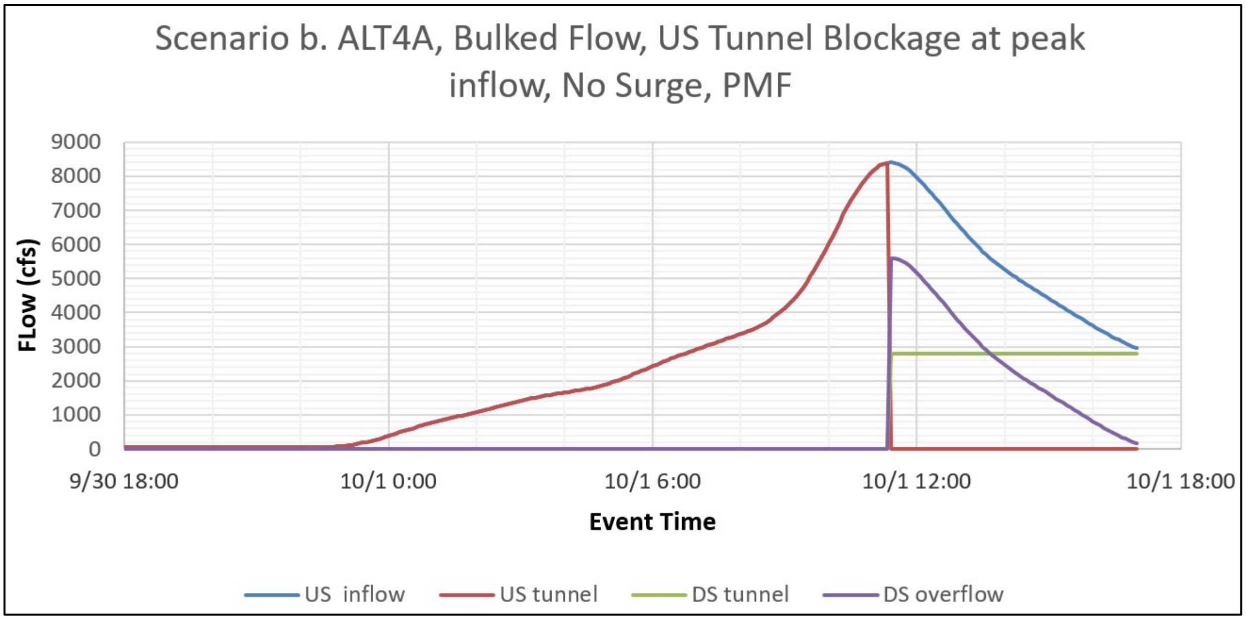

Figure 29. PMF Overflow Hydrograph for Scenario b., Alternative 4A.

6.3.3. Scenario c. No Tunnel Blockage, Surge Release

Scenario c. is based on the PMF hydrograph with surge, as described in this Appendix. The PMF hydrograph flow values are scaled based on the peak flow value for the various AEP peak flows. The tunnel or tunnels under consideration are assumed to bypass water up to the tunnel capacity. At this point, additional flow overtops the spillway/dam and flows down the remainder of the original stream channel paralleling Lowell Canyon Road and out onto the alluvial fan. During the event, a surge release is modeled to occur, as described in this Appendix. Overflow hydrographs were created, as shown in Figure 30 (Alternative 2), Figure 31 (Alternative 3A), and Figure 32 (Alternative 4A). The inflow hydrograph (blue line) was truncated by subtracting all flow up to the tunnel capacity (red line), resulting in overtopping flow (purple line). For Alternative 4A, where a second tunnel routes discharge, the second tunnel capacity is also subtracted (green line).

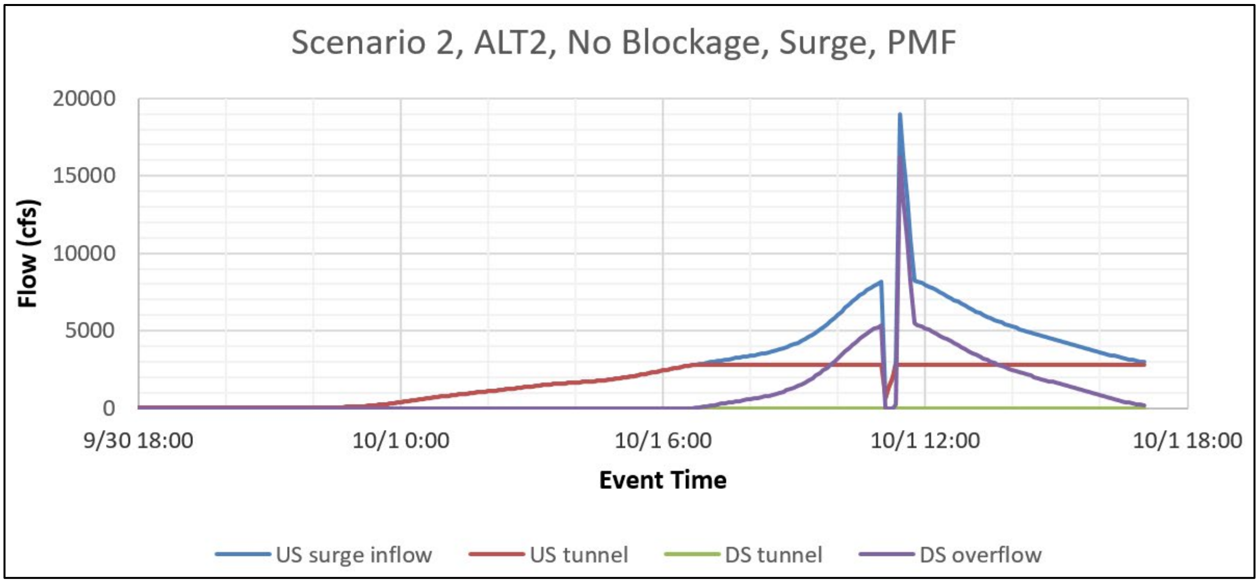

Figure 30. PMF Overflow Hydrograph for Scenario c., Alternative 2.

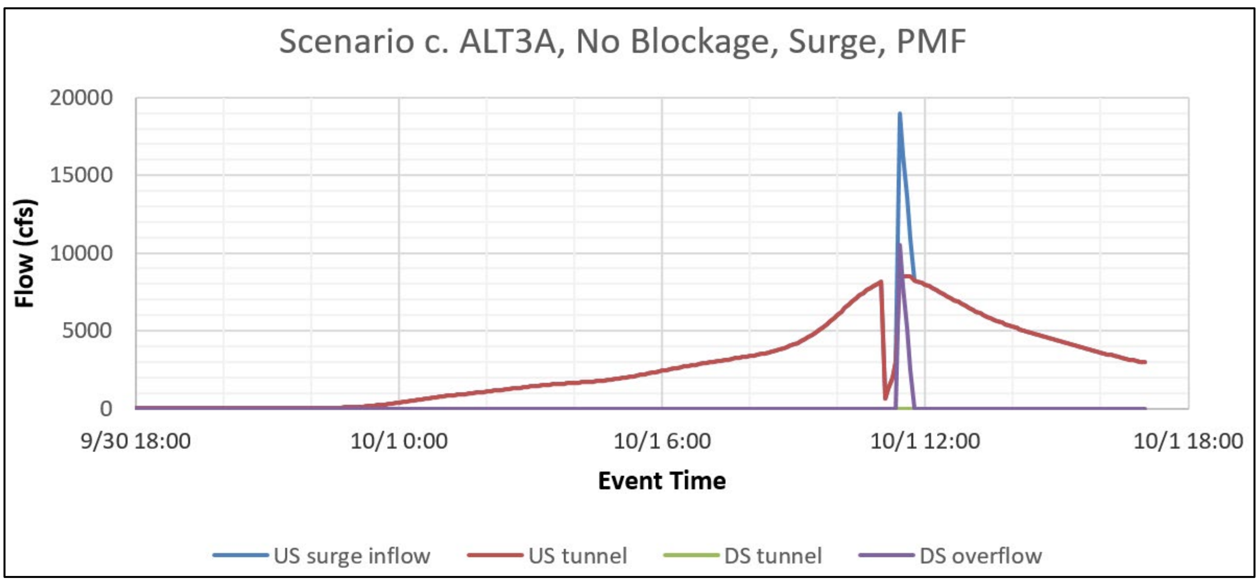

Figure 31. PMF Overflow Hydrograph for Scenario c., Alternative 3A.

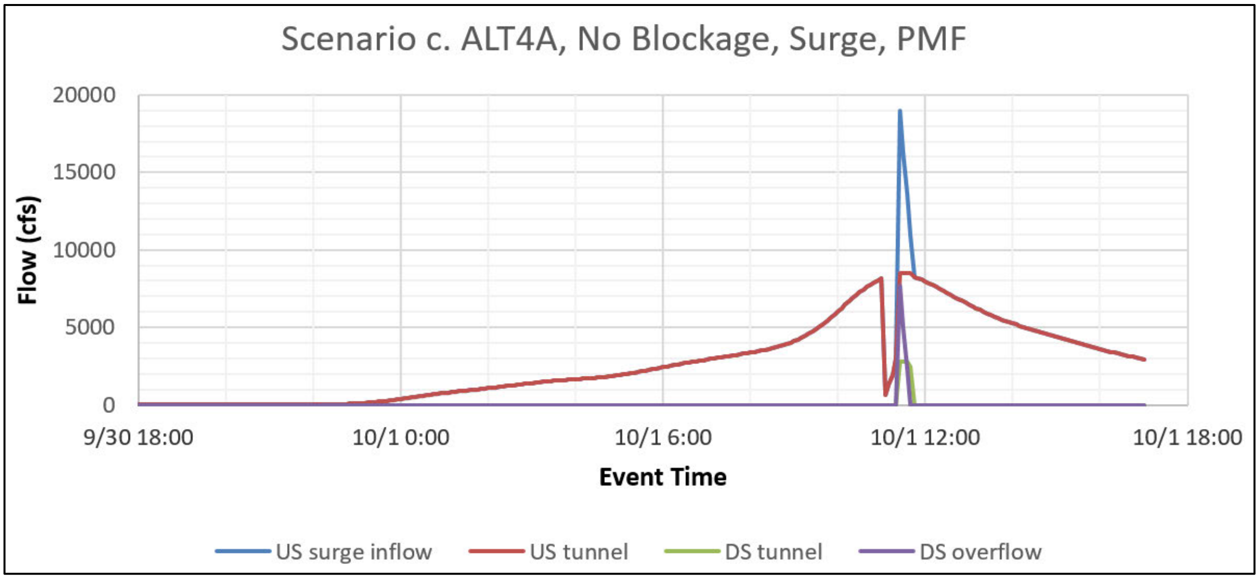

Figure 32. PMF Overflow Hydrograph for Scenario c., Alternative 4A.

6.3.4. Scenario d. Tunnel Blockage, Surge Release

Scenario d. is based on the PMF hydrograph, as described in this Appendix. The PMF hydrograph flow values are scaled based on the peak flow value for the various AEP peak flows. The tunnel or tunnels under consideration are assumed to bypass water up to the tunnel capacity until a blockage occurs. At this point, all flow overtops the spillway/dam and flows down the remainder of the original stream channel paralleling Lowell Canyon Road and out onto the alluvial fan. The blockage was modeled to occur at the peak flow of the hydrograph, or the flow when water begins to flow over the spillway when the tunnel reaches capacity. For Alternative 4A, a blockage is only assumed to occur on the upstream tunnel. During the event, a surge release is modeled, as described in this Appendix. Overflow hydrographs were created, as shown in Figure 33 (Alternative 2), Figure 34 (Alternative 3A), and Figure 35 (Alternative 4A). The inflow hydrograph (blue line) was truncated by subtracting all flow up to the tunnel capacity (red line), resulting in overtopping flow (purple line). For Alternative 4A, where a second tunnel routes discharge, the second tunnel capacity is also subtracted (green line).

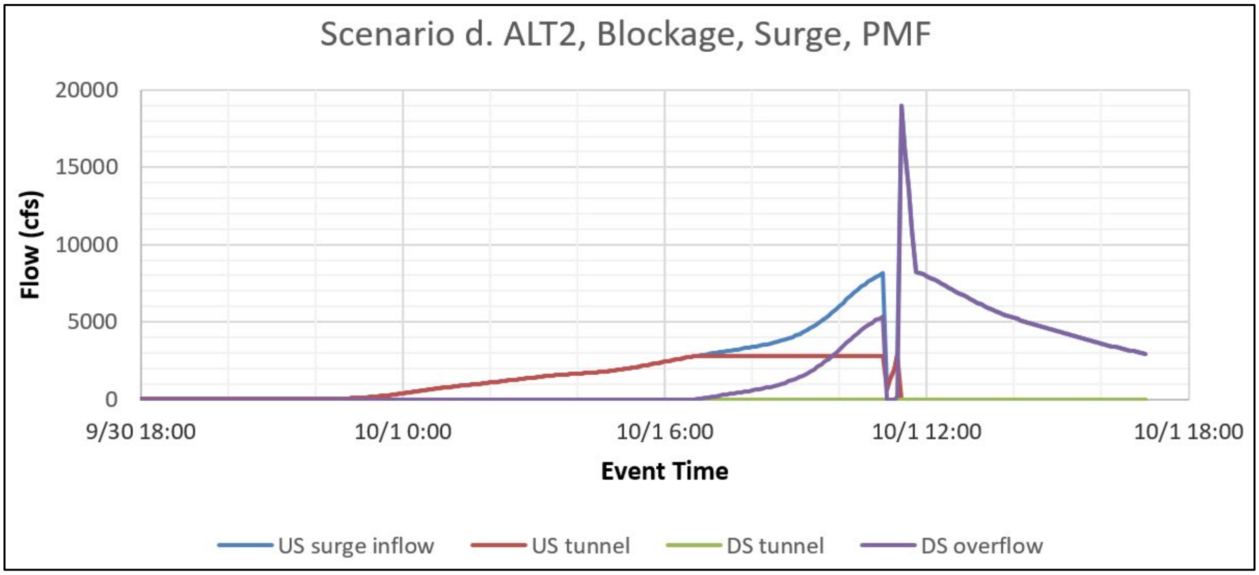

Figure 33. PMF Overflow Hydrograph for Scenario d., Alternative 2.

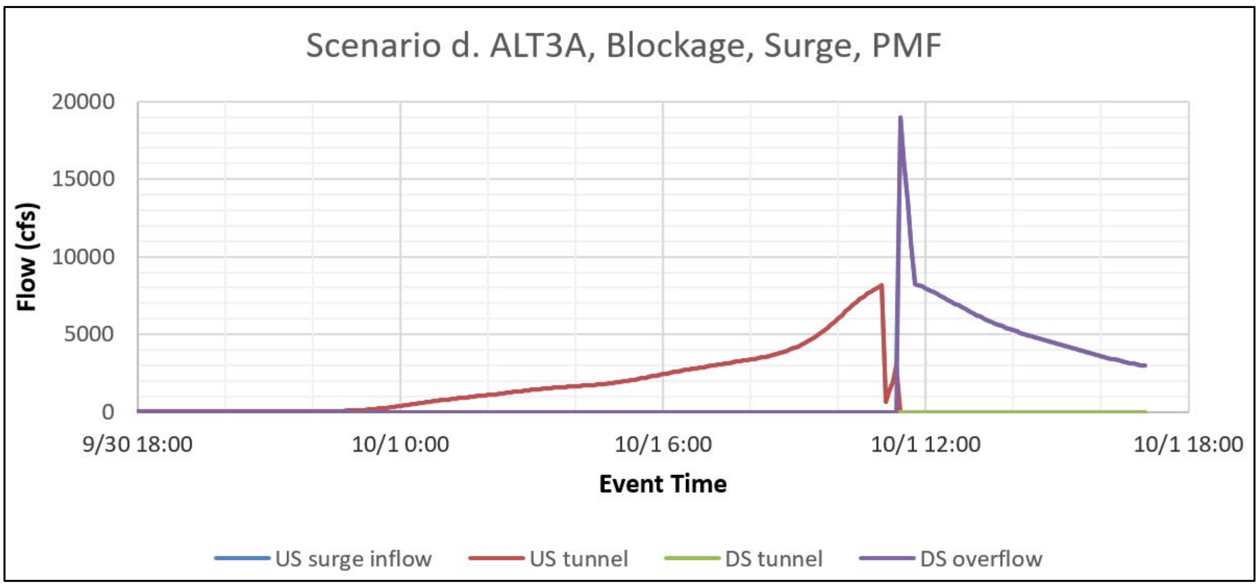

Figure 34. PMF Overflow Hydrograph for Scenario d., Alternative 3A.

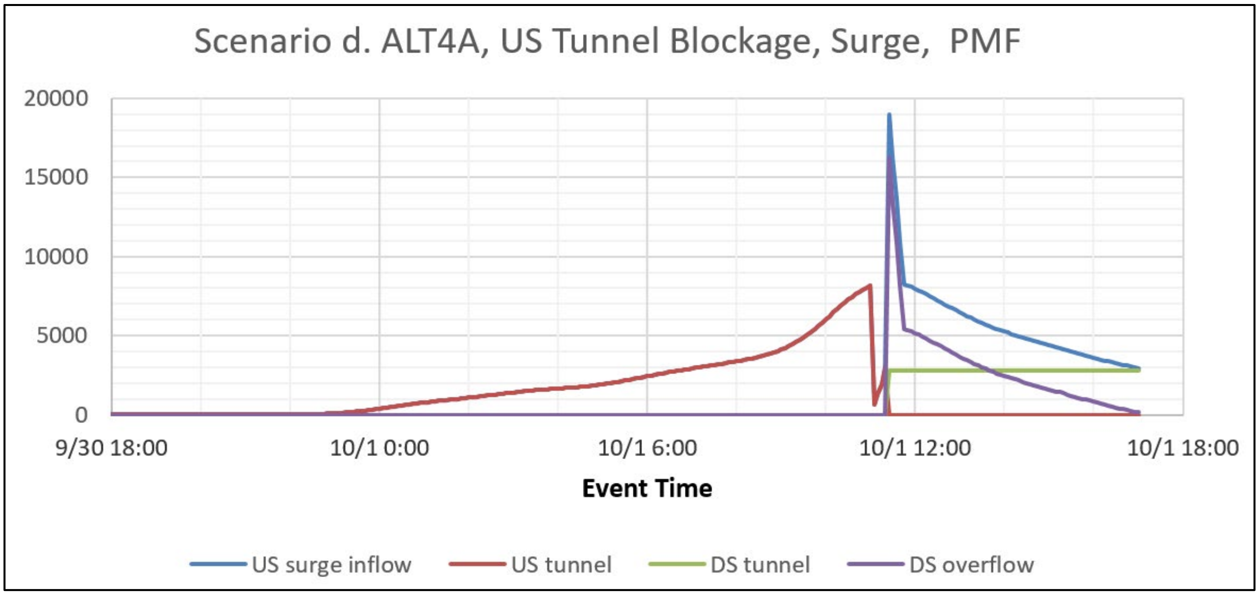

Figure 35. PMF Overflow Hydrograph for Scenario d., Alternative 4A.

6.3.5. Event Matrix

Testing four scenarios for three alternatives with five hydraulic loading events leads to a potential of 60 model runs to evaluate the alternatives. Some scenarios were found to pass all flow through the tunnel resulting in no overflow over the spillway and no impact on the City of Seward. Nineteen of these cases were found, and the remaining 41 hydraulic cases were evaluated in HEC-RAS. The following tables show the overflow conditions modeled. In these tables, maximum spillway flow shows the peak flow that was modeled through Seward. The events modeled for Alternative 2 are shown in Table 9. The events modeled for Alternative 3A are shown in Table 10, and Table 11 shows the events modeled for Alternative 4A.

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Table 9. Overflow Events Modeled for Alternative 2.

Alternative and Scenario	Inflow AEP	Peak Inflow (cfs)	Maximum US Tunnel Flow (cfs)	Maximum DS Tunnel Flow (cfs)	Minimum US Tunnel Flow (cfs)	Maximum Spillway Flow (cfs)	Trigger for Blockage (cfs)	Trigger for Surge
2. a. Repaired (E) Tunnel No Blockage No Surge	1.35E-01	1,700	1,700	N/A	Controlled by Inflow Hydrograph values less than 2,800 cfs	0	N/A	N/A
	2.75E-02	2,600	2,600			0		
	8.27E-04	5,000	2,800			2,200		
	1.52E-04	6,400	2,800			3,600		
	1.89E-05	8,400	2,800			5,600		
2. b. Repaired (E) Tunnel Blockage No Surge	1.35E-01	1,700	1,700	N/A	0	1700	Q = 1,700	N/A
	2.75E-02	2,600	2,600			2600	Q = 2,600	
	8.27E-04	5,000	2,800			5000	Q = 2,800	
	1.52E-04	6,400	2,800			6400	Q = 2,800	
	1.89E-05	8,400	2,800			8400	Q = 2,800	
2. c. Repaired (E) Tunnel No Blockage Surge	1.35E-01	3,800	2,800	N/A	Controlled by Inflow Hydrograph values less than 2,800 cfs	1,000	N/A	Non-Surge Peak Inflow
	2.75E-02	5,800	2,800			3,000		
	8.27E-04	11,300	2,800			8,500		
	1.52E-04	14,500	2,800			11,700		
	1.89E-05	19,000	2,800			16,200		
2. d. Repaired (E) Tunnel Blockage Surge	1.35E-01	3,800	2,800	N/A	0	3,800	Concurrent with Surge	Non-Surge Peak Inflow
	2.75E-02	5,800	2,800			5,800		
	8.27E-04	11,300	2,800			11,300		
	1.52E-04	14,500	2,800			14,500		
	1.89E-05	19,000	2,800			19,000		

US = Upstream; DS = Downstream

Table 10. Overflow Events Modeled for Alternatives 3A.

Alternative and Scenario	Inflow AEP	Peak Inflow (cfs)	Maximum US Tunnel Flow (cfs)	Maximum DS Tunnel Flow (cfs)	Minimum US Tunnel Flow (cfs)	Maximum Spillway Flow (cfs)	Trigger for Blockage (cfs)	Trigger for Surge
3A. a. Enlarge (E) Tunnel to 18 ft No Blockage No Surge	1.35E-01	1,700	1,700	N/A	Controlled by Inflow Hydrograph values less than 8,500 cfs	0	N/A	N/A
	2.75E-02	2,600	2,600					
	8.27E-04	5,000	5,000					
	1.52E-04	6,400	6,400					
	1.89E-05	8,400	8,400					
3A. b. Enlarge (E) Tunnel to 18 ft Blockage No Surge	1.35E-01	1,700	1,700	N/A	0	1,700	Q = 1,700	N/A
	2.75E-02	2,600	2,600			2,600	Q = 2,600	
	8.27E-04	5,000	5,000			5,000	Q = 5,000	
	1.52E-04	6,400	6,400			6,400	Q = 6,400	
	1.89E-05	8,400	8,400			8,400	Q =8,400	
3A. c. Enlarge (E) Tunnel to 18 ft No Blockage Surge	1.35E-01	3,800	3,800	N/A	Controlled by Inflow Hydrograph values less than 8,500 cfs	0	N/A	Non-Surge Peak Inflow
	2.75E-02	5,800	5,800			0		
	8.27E-04	11,300	8,500			2,800		
	1.52E-04	14,500	8,500			6,000		
	1.89E-05	19,000	8,500			10,500		
3A. d. Enlarge (E) Tunnel to 18 ft Blockage Surge	1.35E-01	3,800	3,800	N/A	0	3,800	Concurrent with Surge	Non-Surge Peak Inflow
	2.75E-02	5,800	5,800			5,800		
	8.27E-04	11,300	8,500			11,300		
	1.52E-04	14,500	8,500			14,500		
	1.89E-05	19,000	8,500			19,000		
US = Upstream; DS = Downstream								

Table 11. Overflow Events Modeled for Alternative 4A.

Alternative and Scenario	Inflow AEP	Peak Inflow (cfs)	Maximum US Tunnel Flow (cfs)	Maximum DS Tunnel Flow (cfs)	Minimum US Tunnel Flow (cfs)	Maximum Spillway Flow (cfs)	Trigger for Blockage (cfs)	Trigger for Surge
4A. a. Add new 18 ft Tunnel & Repair (E) Tunnel No Blockage No Surge	1.35E-01	1,700	1,700	0	Controlled by Inflow Hydrograph values less than 8,500 cfs	0	N/A	N/A
	2.75E-02	2,600	2,600					
	8.27E-04	5,000	5,000					
	1.52E-04	6,400	6,400					
	1.89E-05	8,400	8,400					
4A. b. Add new 18 ft Tunnel & Repair (E) Tunnel Blockage No Surge	1.35E-01	1,700	1,700	1,700	0	0	N/A	N/A
	2.75E-02	2,600	2,600	2,600		0	N/A	
	8.27E-04	5,000	5,000	2,800		2,200	Q = 5,000	
	1.52E-04	6,400	6,400	2,800		3,600	Q = 6,400	
	1.89E-05	8,400	8,400	2,800		5,600	Q = 8,400	
4A. c. Add new 18 ft Tunnel & Repair (E) Tunnel No Blockage Surge	1.35E-01	3,800	3,800	0	Controlled by Inflow Hydrograph values less than 8,500 cfs	0	N/A	Non-Surge Peak Inflow
	2.75E-02	5,800	5,800	0		0		
	8.27E-04	11,300	8,500	2,800		0		
	1.52E-04	14,500	8,500	2,800		3,200		
	1.89E-05	19,000	8,500	2,800		7,700		
4A. d. Add new 18 ft Tunnel & Repair (E) Tunnel Blockage Surge	1.35E-01	3,800	3,800	2,800	0	1,000	Concurrent with Surge	Non-Surge Peak Inflow
	2.75E-02	5,800	5,800			5,800		
	8.27E-04	11,300	8,500			11,300		
	1.52E-04	14,500	8,500			14,500		

US = Upstream; DS = Downstream

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6.4. Model Output

Model simulation runs were performed, and grids (.HDF files) were produced for analysis in LifeSim. Consequence analysis was a function of resultant overflow depths and velocities through Seward. It must be noted that the model results represent clear water results with no changes to flow paths, which are likely to occur during high flow events. While the model results show specific locations where the flow was modeled to occur, all locations within the alluvial fan are subject to overflow risk as these flow paths could be blocked by debris resulting in a different flow routing. A good representation of risk in Seward is shown in Figure 36. In general terms, depths and velocities are highest in the canyon immediately downstream of the diversion dam, as shown on Figures 37, 38, 39, and 40. Several individual houses, multi-unit residences, and the community hospital are located in this area. Depths in this area were found to exceed 10 ft adjacent to some of these buildings, and velocities between the buildings were in the range of 15 ft per second during PMF overtopping events. As the overflow exits the canyon, and it spreads out over the alluvial fan through downtown Seward. Depths and velocities decrease, and a major concentration of flow continues down Jefferson Street to Resurrection Bay. Branching flows were modeled to the south and during larger overflow events, to the north of Jefferson Street. As stated before, these paths are based on fixed-bed geometry. Engineering judgment, and general knowledge of flood-events on alluvial fans indicates that debris movement will shift from these paths as an event progresses and flow paths could occur anywhere on the alluvial fan, as indicated in Figure 36.

[redacted]

[redacted]

[redacted]

7. STRUCTURAL DESIGN

Structural design of the system closely followed the design of the existing project. Existing project features were relocated and scaled as needed to develop concept level designs for the new flood diversion system. Detailed design requirements for the tunnel system will be in accordance with EM 1110-2-2901, Tunnels and Shafts in Rock. Guidance from ER 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects; Bureau of Reclamation Design Standard No. 3, Water Conveyance Facilities, Fish Facilities, and Roads and Bridges (BoR 2014); FHWA Technical Manual for Design and Construction of Road Tunnels; and other design standards will be applied to system elements as applicable.

7.1. Tunnel Design

Tunnel design assumes a horseshoe tunnel similar to the existing tunnel. Concrete thickness has been assumed to be equal to the same number of inches that the tunnel diameter is in feet (thus 18-inch-thick concrete for an 18-ft-diameter tunnel). Armoring was assumed to be accomplished with 2-inch x 4-inch steel flat bars allowing for better weldability than would be the case using railroad rails. In all cases, tunnels are assumed to be contact-grouted after the concrete placement has been completed to ensure full support around the circumference of the tunnel. The primary components of refurbishing the existing tunnel are to re-establish steel armor protection in the tunnel invert and contact grouting the crown. Figure 41 shows details involved with the repair of the existing tunnel.

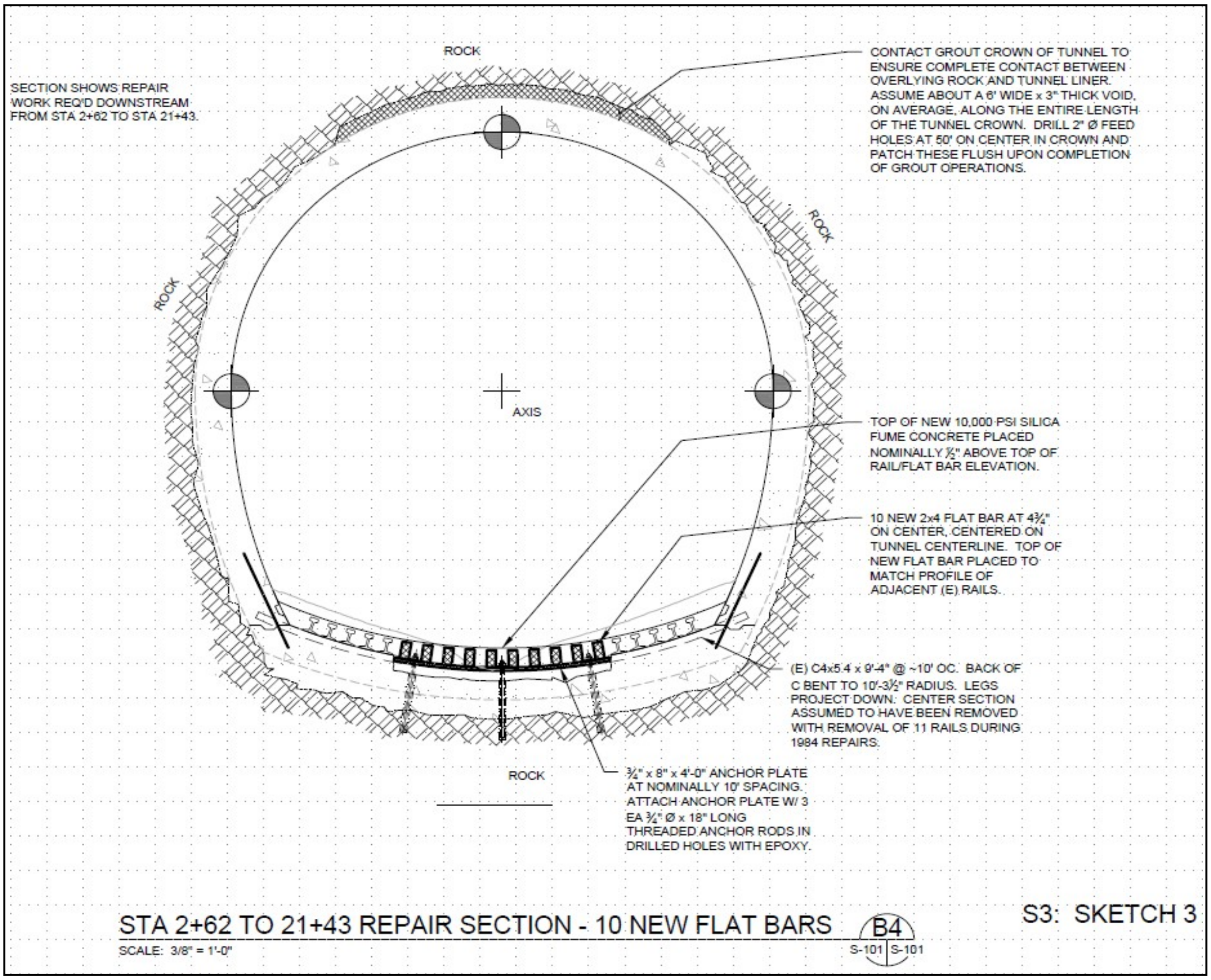

Figure 41. Refurbish Existing Tunnel Cross Section.

7.2. Extended Outfall Design

Outfalls have been designed as pre-cast concrete open-channel flumes placed on drilled piers with pier caps, similar to those typically used in bridge construction. Piers are concrete-filled steel pipes with a rebar cage. The pre-cast flume sections have bent tube-steel struts across the top of the walls to facilitate lifting and placing as well as reinforcing the side walls of the flume for lateral loads. Armoring is field-welded and encased in concrete to form a replaceable wear surface, which also will allow for a uniform slope. The system has been designed for a mounded gravel live load to prevent flume failure should a blockage occur.

Seismic loads perpendicular to the length of the flume have been accounted for. However, further analysis must be done to account for seismic loads along the length of the flume. A rigid connection to the supporting rock where the flume is tied to Bear

Mountain would prevent the piers from seeing lateral loads for seismic forces in this direction, which would make for a large load over a small area. For the 150-ft-long outfall extension under consideration, these large forces may be manageable, but this has not been evaluated at this time. It is expected that his work will be done during preconstruction engineering and design (PED). Figures 42, 43, 44, 45, and 46 present drawings of the outlet and flume designs that were considered.

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DRILLED SHAFTS			
PILE No.	APPROXIMATE O.A.L. (FT)	APPROXIMATE TOTAL EMBEDMENT (FT)	APPROXIMATE ROCK EMBEDMENT (FT)
P03	80	50	38
P04	100	63	34
P05	115	83	28

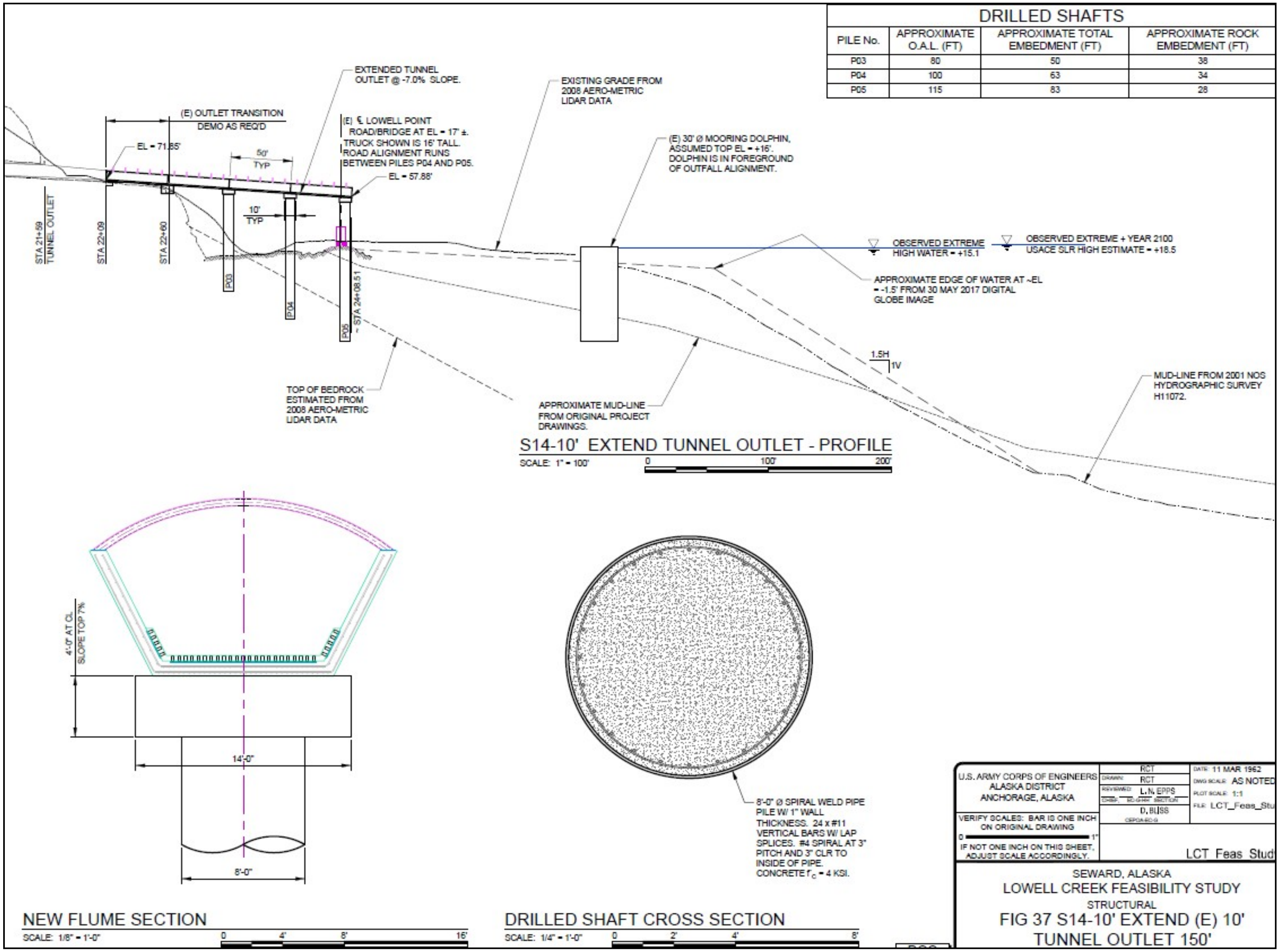
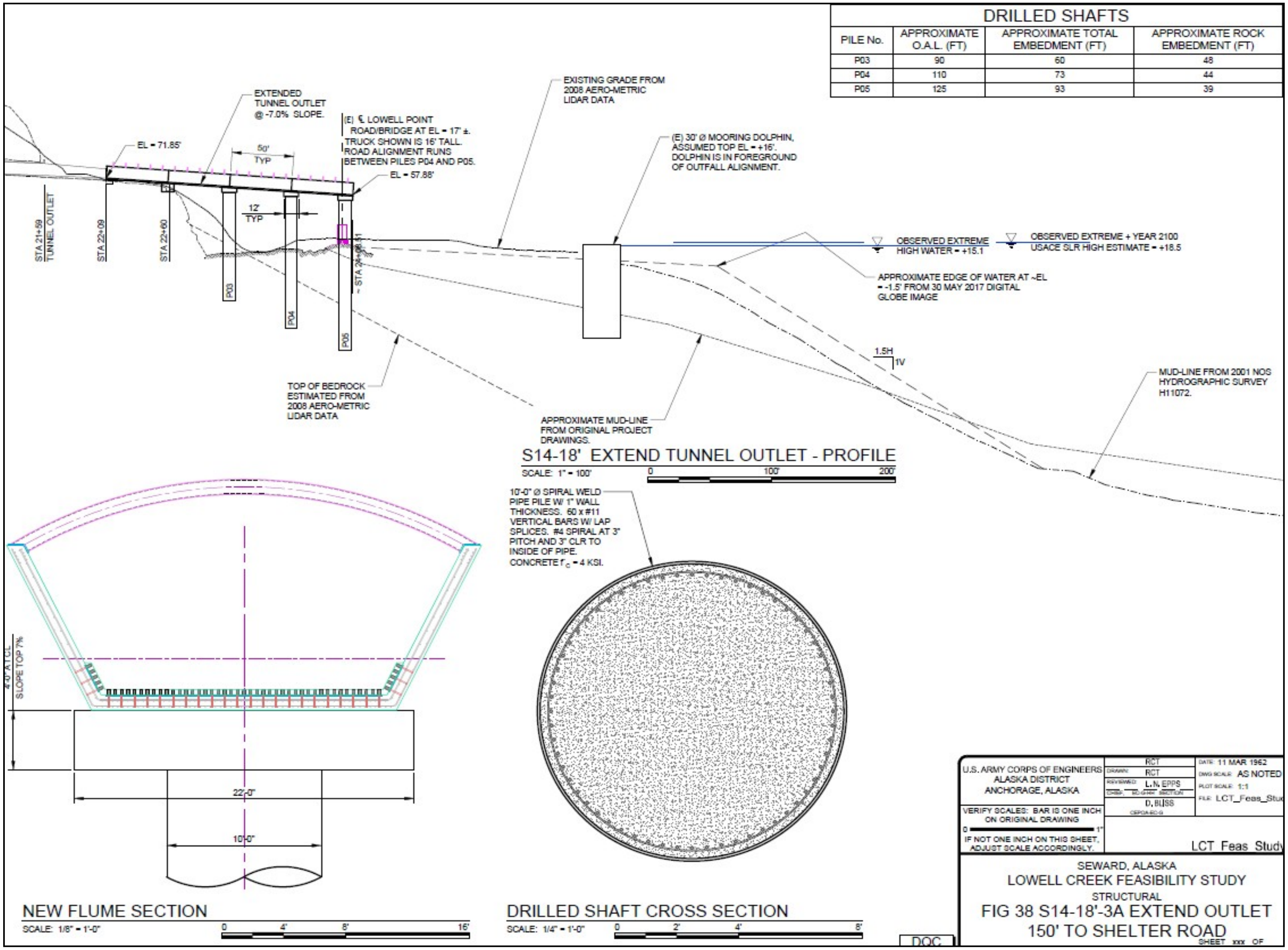


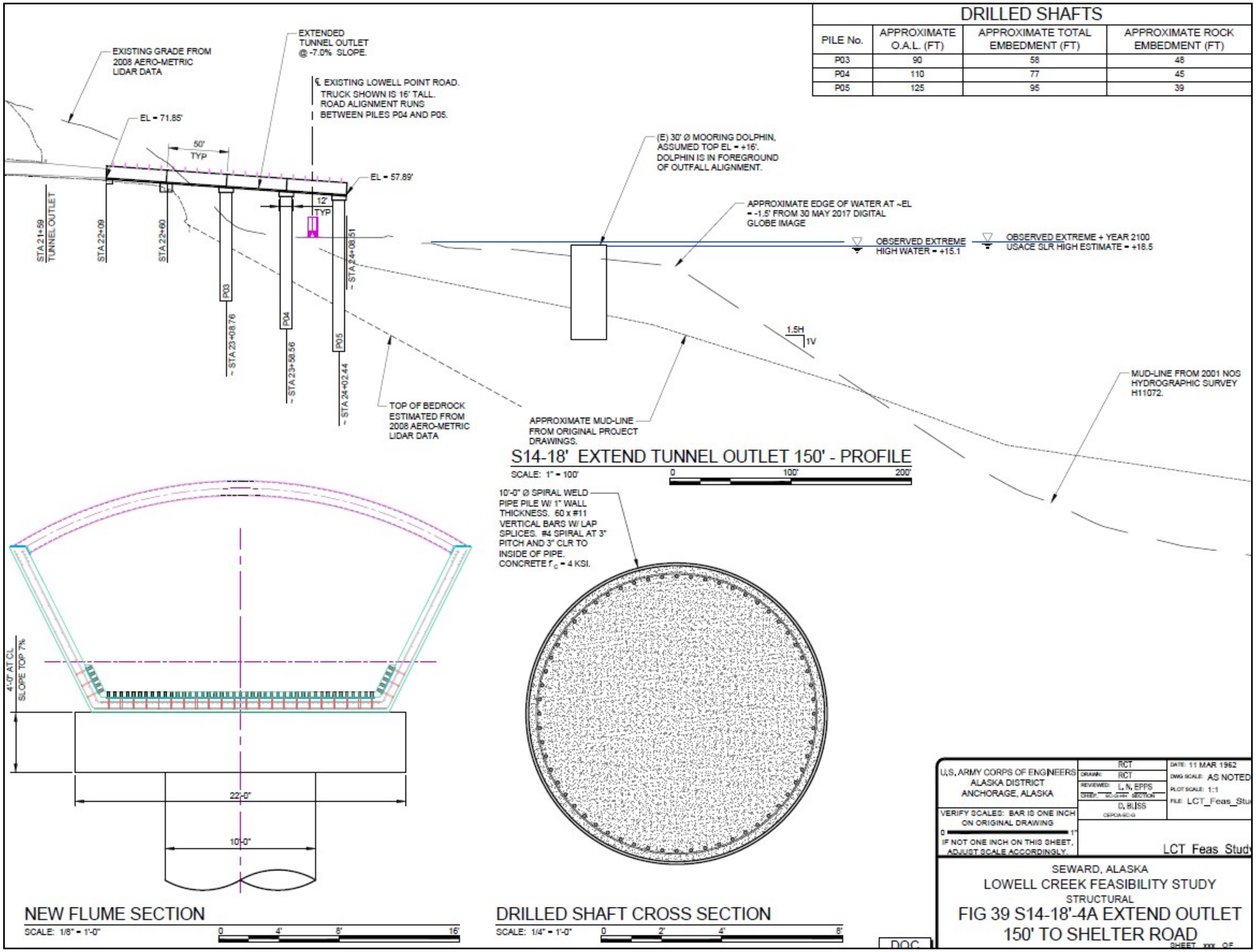
Figure 42. Extend Existing 10-ft Tunnel Outlet 150 ft to Shelter Road (Alternative 2).



DRILLED SHAFTS

PILE No.	APPROXIMATE O.A.L. (FT)	APPROXIMATE TOTAL EMBEDMENT (FT)	APPROXIMATE ROCK EMBEDMENT (FT)
P03	90	60	48
P04	110	73	44
P05	125	93	39

Figure 43. Extend Alternative 3A Tunnel Outlet 150 ft to Shelter Road (Alternative 3B Similar).



DRILLED SHAFTS			
PILE No.	APPROXIMATE O.A.L. (FT)	APPROXIMATE TOTAL EMBEDMENT (FT)	APPROXIMATE ROCK EMBEDMENT (FT)
P03	90	58	48
P04	110	77	45
P05	125	95	39

Figure 44. Extend Alternative 4A Tunnel Outlet 105 ft to Shelter Road (Alternative 4B Similar).

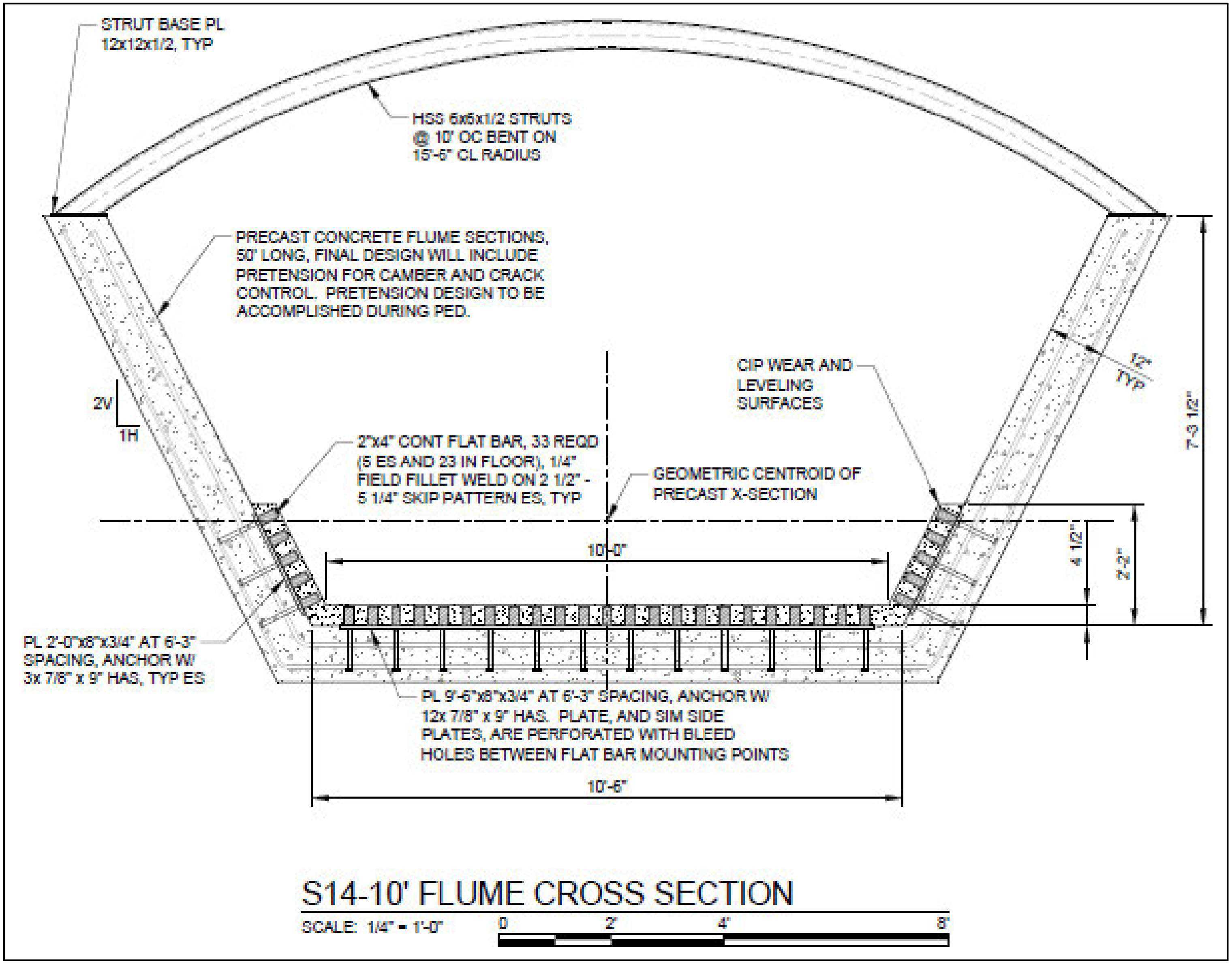


Figure 45. 10-ft Flume Cross Section.

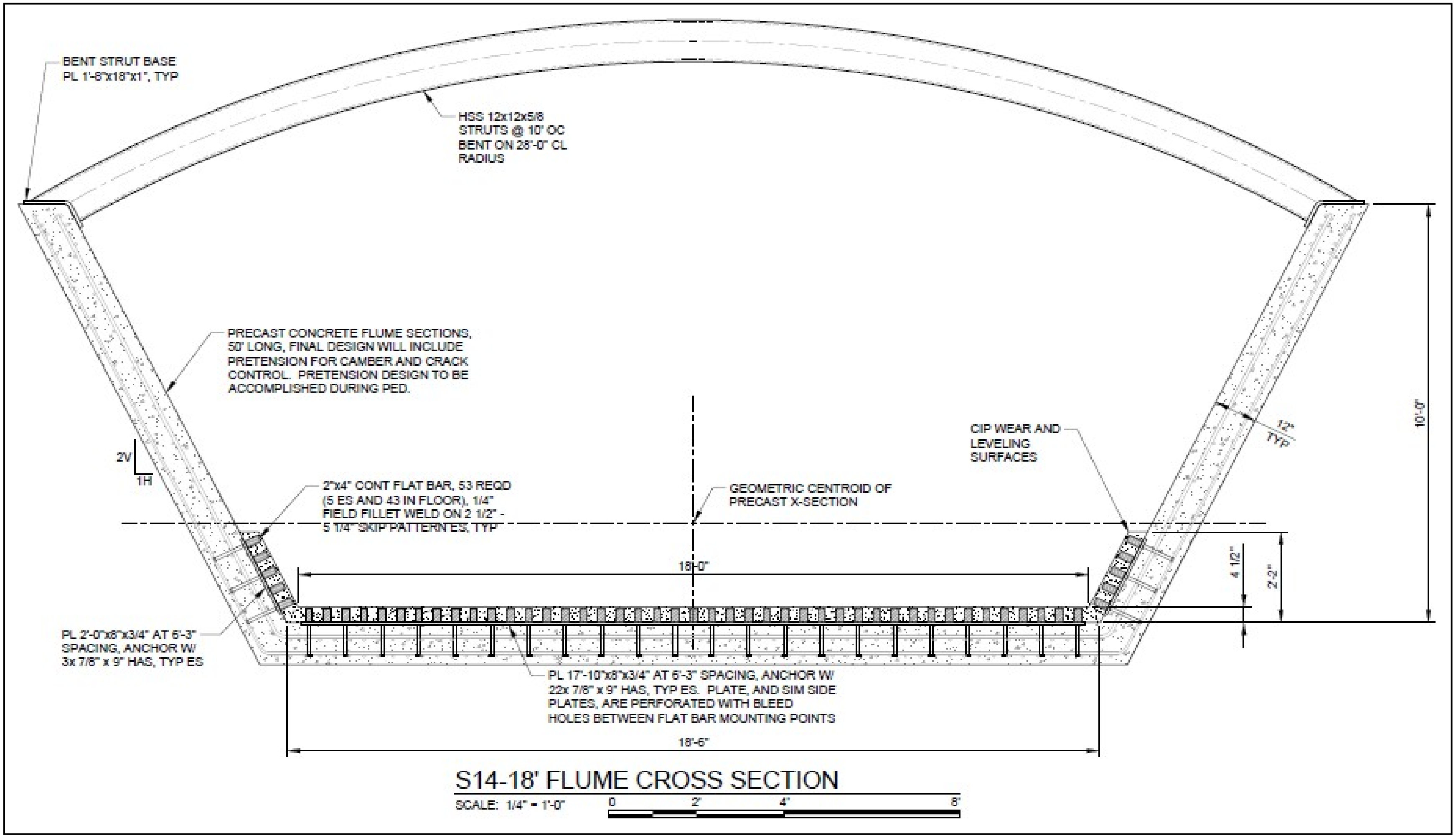


Figure 46. 18-ft Flume Cross Section.

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7.3. Diversion Dam and Intake Transition Design

The diversion dam and intake transition designs are largely based on the existing configuration. The steep canyon sidewalls, the width of the canyon bottom at the dam sites, and the lack of knowledge regarding depth to bedrock combine to necessitate an assumption of 40 ft of excavation and concrete placement at the toe of the dam. Excavation to this depth is prudent to prevent the undermining of the structure due to head-cutting during overtopping events. Any new intake transition design will require physical modeling to confirm performance. In the initial design configuration, the diversion dam height above the adjacent streambed has been kept similar to that of the existing system. See Figure 47 for a plan of a new dam and intake transition as required for Alternative 3.

7.4. Tunnel Inlet Portal Canopy Design

The tunnel inlet portal canopy is designed as a steel-frame structure with concrete footings tied into bedrock and a combination of site-cast and precast concrete decking. Design live load capacity was set at 600 psf to provide substantial resistance to landslide-related loading. No composite action was assumed between the steel girders and the deck slabs; however, this could be evaluated during PED to assess if it would provide some cost reduction or increase the structure’s load capacity. At this time, no architectural treatment has been included; however, it is assumed that a large structure of this type in a natural setting should consider aesthetics for the final design. See Figures 48 and 49 for details of the tunnel inlet portal canopy.

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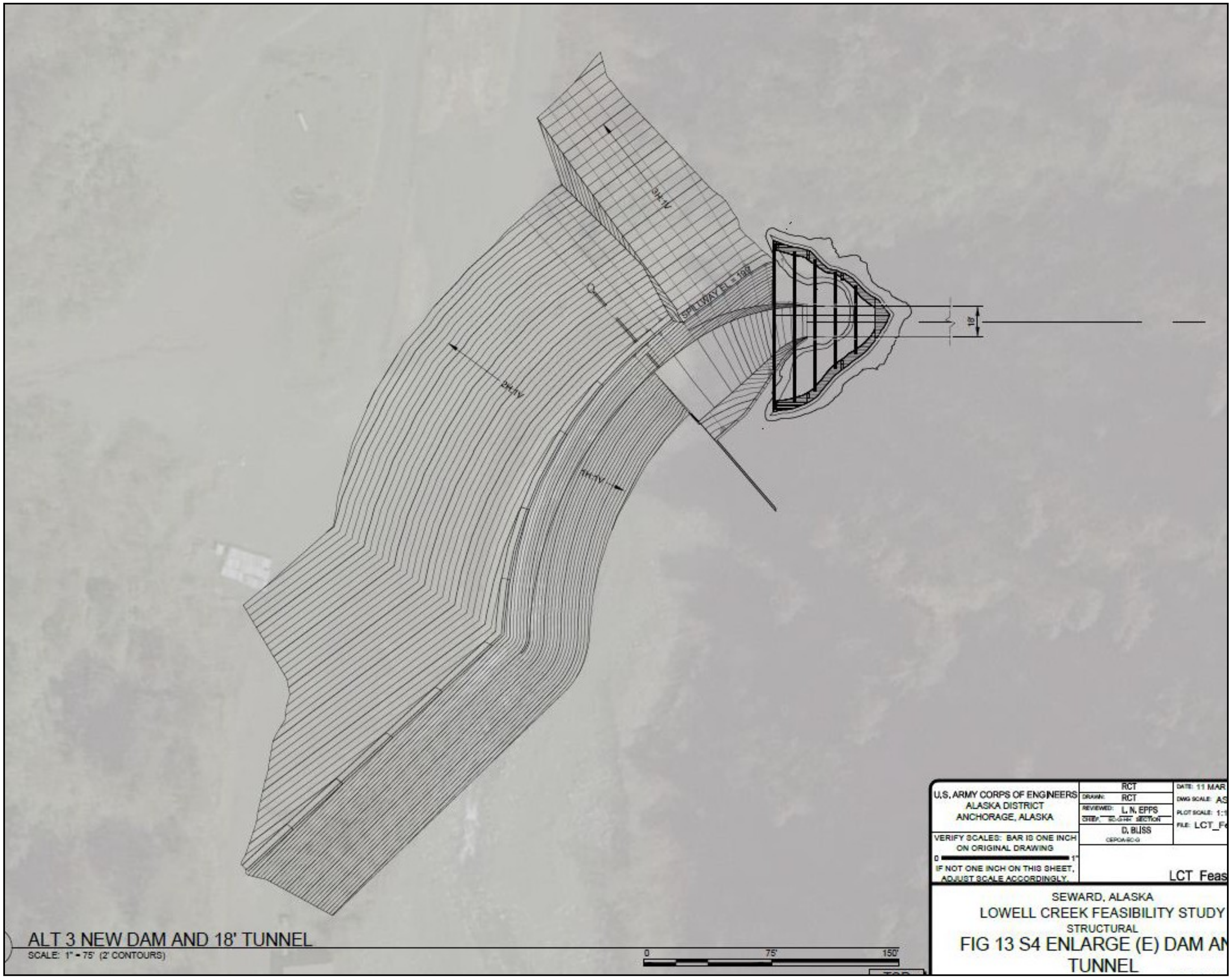

Figure 47. New Dam and Intake Transition for Enlarge Existing Tunnel.

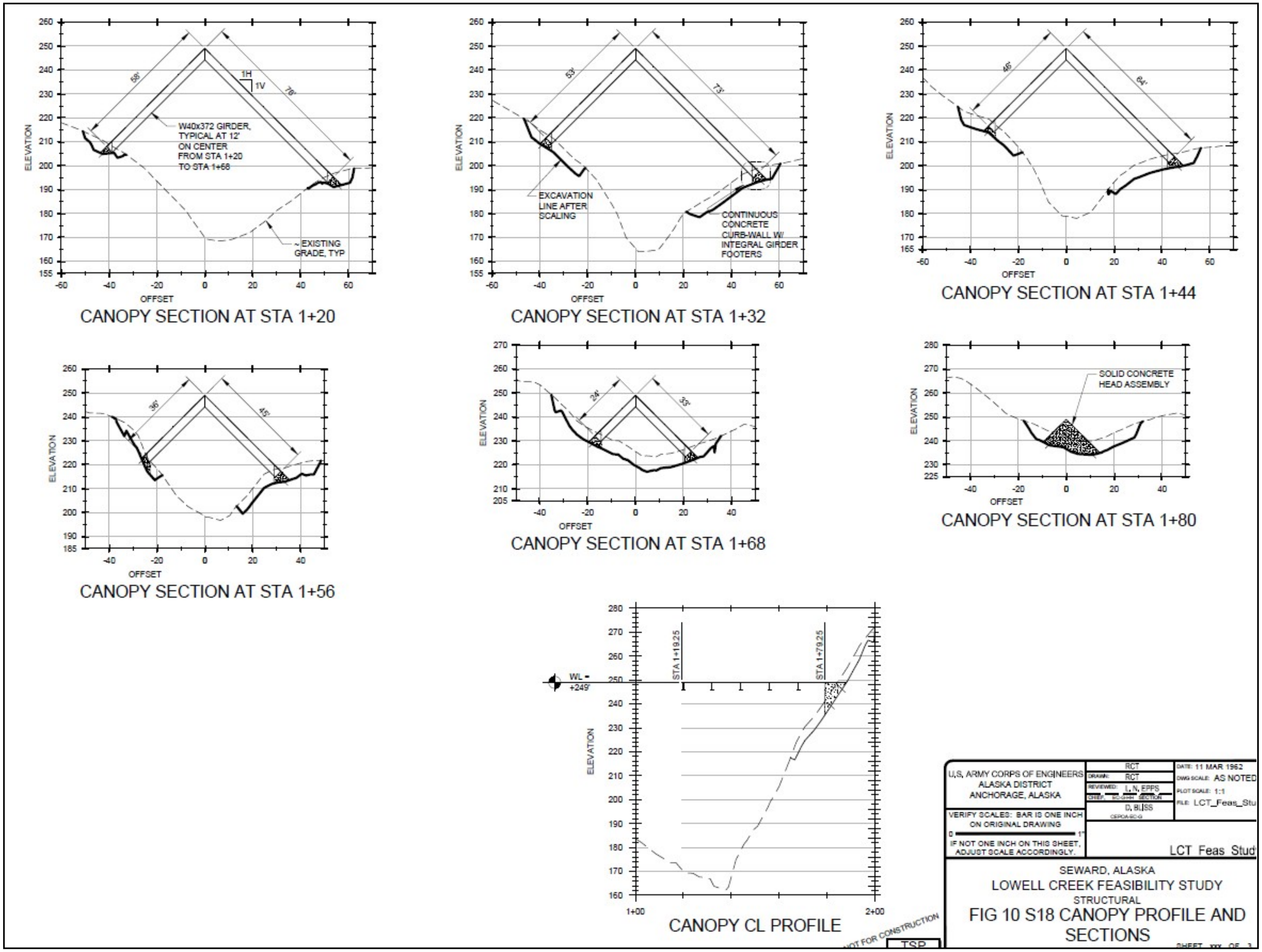

Figure 48. Tunnel Entrance Portal Canopy Details.

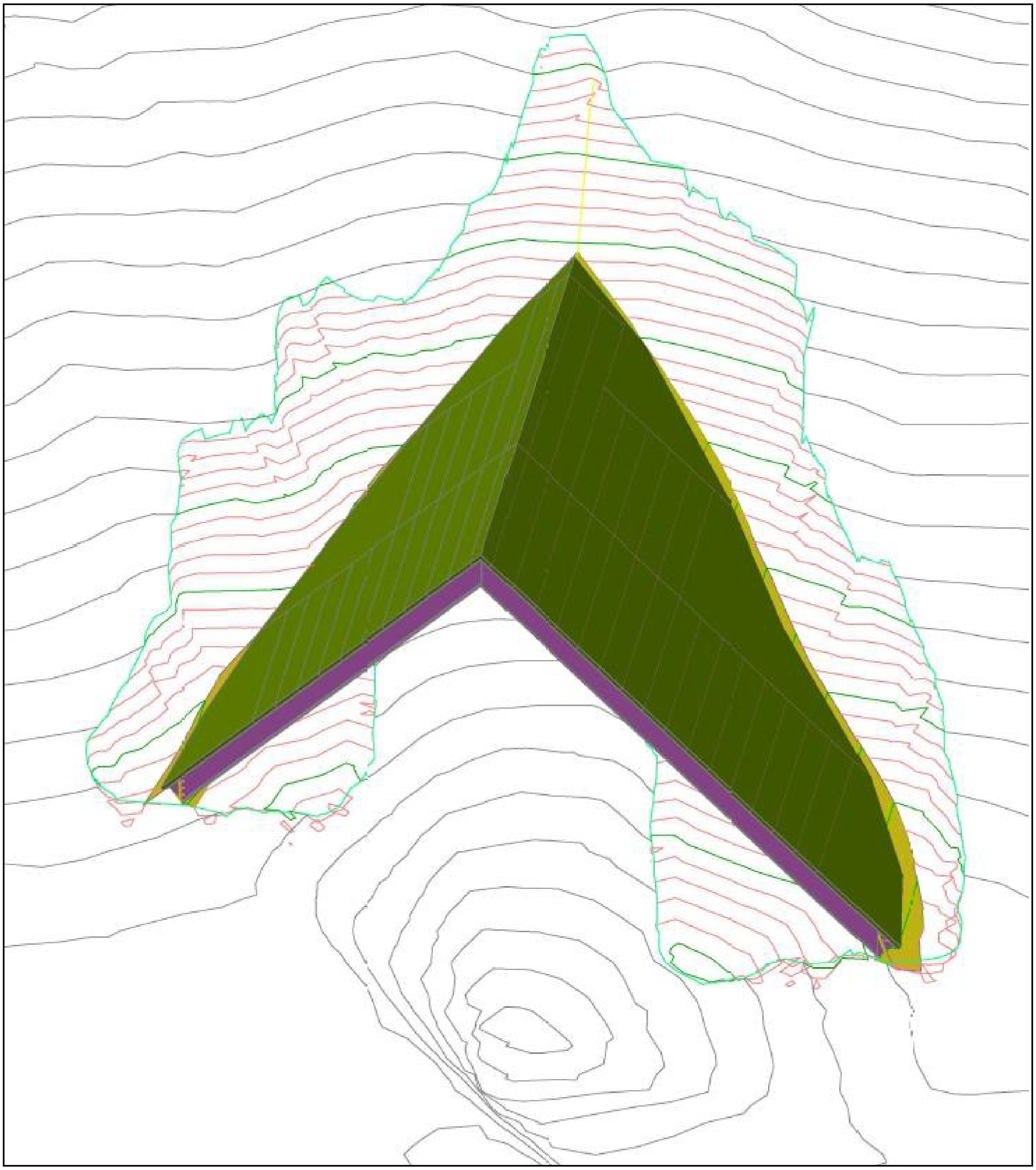

Figure 49. Entrance Portal Canopy Oblique View.

8. PROJECT IMPLEMENTATION

This section will describe a brief description of construction methods that are expected to be employed to construct the project.

8.1. Diversion Dam and Intake Transition

It is assumed that the diversion dam will be constructed of roller-compacted concrete; however, the intake transition will require formed and carefully controlled concrete screeding and finishing. The details of combining these construction methods will need to be further evaluated during design.

8.2. Tunnel

It is assumed that tunnel construction will be by drill and blast methods and that a stabilizing shotcrete liner will be installed prior to forming and placing the concrete liner. Contact grouting will be accomplished after the concrete liner is placed to ensure full contact at the tunnel crown.

8.3. Outfalls

Outfall construction will be similar to simple-span, pre-cast concrete bridge construction with land-based equipment being necessary to complete the structure. Multiple cranes may be necessary to lift the heavy flume elements into place.

9. OPERATIONS AND MAINTENANCE

This section will describe the operations and maintenance activities required to maintain a functional project. The concrete tunnel lining and the upstream face of the diversion dam are expected to deteriorate over time, as has been experienced with the existing project.

9.1. Improve Low Flow Diversion System

The water needs to be diverted reliably to perform repairs. Historically this has required the construction of a temporary detention berm and pond upstream of the tunnel entrance. Water from this pond has then been routed through corrugated pipes and routed downstream of the existing dam and fed into the existing storm drain manhole below the existing dam. Cold weather causes freezing in the exposed corrugated piles, requiring the use of ground thawing, or similar, equipment to keep water flowing. Current practice limits maintenance activities to late winter and early spring months when low flow conditions exist. A concrete sump will be installed above the existing dam to improve this scenario. The sump will include a headwall and gates feeding permanent diversion piping down the Jefferson street alignment to tidewater. The idea is to be able to reasonably and reliably divert winter flows to allow tunnel maintenance. Water diversion costs are included in the cost of concrete repairs, as described below.

9.2. Concrete Repairs

Repairs to the tunnel lining will be focused on the invert where water and debris have been flowing as well as completing contact grouting of the tunnel crown. Repairs would be cast in place concrete overlays controlled to maintain the design slope and grade of the tunnel invert. Successful repair operations in the past have employed temporary grade control beams and a screed that produces the invert profile by traversing the temporary rails. Concrete was delivered to the repair areas by various means, including winch operated carts, small wagon driven by gas-powered All-Terrain Vehicle (i.e., ATV/quad), and diesel-powered tracked vehicles (Yanmar).

The assumed cost for maintaining the concrete surfaces of the alternatives is based on a review of maintenance activities over the history of the existing project. After construction, maintenance was performed in 1945 to improve the rail reinforcement of the concrete. The cost of this effort is not known. Since 1945, USACE records show that $19,714,235 (adjusted to 2020 dollars) has been spent on project maintenance, primarily consisting of concrete repairs to the tunnel invert and intake transition. Over a 75-year period of record, this produces an average annual cost of maintenance of $262,856 (Table 12). The cost of concrete repairs was adjusted for each alternative to reflect differing levels of effort required to maintain different areas of concrete (Tables 13, 14, 15, 16, 17, and 18).

Table 12. Basis for Concrete Maintenance Costs.

Adjustments to Derive Annual Maintenance Costs		
Descriptions	**Factor**	**Annual Cost**
Approximate Annual Repair Contract (Maintenance) Costs in 2020 Dollars:		$ 262,856
Add 20% for PED and Supervision, Inspection, and Overhead:	20%	$ 315,428
Add Annual Inspection and Report Cost:		$ 325,428
Adjust Annual Cost by Engineering Judgment for Aging Condition of Tunnel:		$ 400,000
Consider that our "Refurbish Existing Tunnel" will put steel armoring back in the invert of the tunnel, which is where a large portion of the maintenance dollars is spent. By engineering judgment, estimate that refurbishing the existing tunnel (or building a new armored tunnel) will cut annual maintenance costs in half.	50%	$ 200,000

Table 13. Alternative 2 Concrete Maintenance Costs.

Adjustments to Derive Annual Maintenance Costs		
Descriptions	**Factor**	**Annual Cost**
Existing Tunnel (~2100 ft long x 10 ft diameter):	100%	$ 200,000
Outfall Extension (~150 ft long x 10 ft wide):	7%	$ 15,000
Total Alt. 2 Concrete Maintenance Cost:		$ 215,000

Table 14. Alternative 3A Concrete Maintenance Costs.

Adjustments to Derive Annual Maintenance Costs		
Descriptions	**Factor**	**Annual Cost**
Existing Tunnel (~2100 ft long x 18 ft diameter):	180%	$ 360,000
Outfall Extension (~150 ft long x 18 ft wide):	13%	$ 26,000
Total Alt. 3A Concrete Maintenance Cost:		$ 386,000

Table 15. Alternative 3B Concrete Maintenance Costs.

Adjustments to Derive Annual Maintenance Costs		
Descriptions	**Factor**	**Annual Cost**
Existing Tunnel (~2100 ft long x 24 ft diameter):	240%	$ 480,000
Outfall Extension (~150 ft long x 24 ft wide):	17%	$ 35,000
Total Alt. 3B Concrete Maintenance Cost:		$ 515,000

Table 16. Alternative 4A Concrete Maintenance Costs.

Adjustments to Derive Annual Maintenance Costs		
Descriptions	**Factor**	**Annual Cost**
New Tunnel (~2270 ft long x 18 ft diameter):	195%	$ 390,000
Existing Tunnel (~2100 ft long x 10 ft diameter, little use):	25%	$ 50,000
Outfall Extension (~150 ft long x 18 ft wide, new tunnel):	13%	$ 26,000
Total Alt. 4A Concrete Maintenance Cost:		$ 466,000

Table 17. Alternative 4B Concrete Maintenance Costs.

Adjustments to Derive Annual Maintenance Costs		
Descriptions	**Factor**	**Annual Cost**
New Tunnel (~2270 ft long x 24 ft diameter):	259%	$ 519,000
Existing Tunnel (~2100 ft long x 10 ft diameter, little use):	25%	$ 50,000
Outfall Extension (~150 ft long x 24 ft wide, new tunnel):	17%	$ 35,000
Total Alt. 4B Concrete Maintenance Cost:		$ 604,000

Table 18. Alternative 5 Concrete Maintenance Costs.

Adjustments to Derive Annual Maintenance Costs		
Descriptions	**Factor**	**Annual Cost**
Existing Tunnel (~2100 ft long x 10 ft diameter, less wear but not refurbished):	150%	$ 150,000
New Debris Basin (0.25% of Current Replacement Cost of $16,650M):		$ 42,000
Total Alt. 5 Concrete Maintenance Cost:		$ 192,000

9.3. Early Warning System

The early warning system consists of three continuously operating gages in the Lowell Creek Basin, a discharge gage to measure the quantity of water exiting the tunnel, and two Snow Telemetry (SNOTEL) sites within the basin to measure rainfall and snowpack accumulation. Overall, the system is assumed to cost approximately $100,000 annually to maintain and operate.

9.3.1. Discharge Gage

The current discharge gage on the system measures water depth and velocity at the tunnel exit every 15 minutes. Data is maintained by the USGS and made publicly available on the National Water Information System webpage. Maintenance of the discharge gage includes providing station power, calibrating the sensors, quality checking the data, and performing site maintenance as necessary to keep the data collection platform functional. The current gage costs $50,000 annually to operate.

9.3.2. SNOTEL Sites

SNOTEL sites are maintained throughout the state through a cooperative agreement with the Natural Resource Conservation Service (NRCS). A typical SNOTEL site consists of sensors to read air temperature, solar radiation, wind speed, soil temperature, rainfall precipitation, total precipitation (rainfall and snowfall), snow depth, and snow water equivalent. These sensors allow a system operator to detect rainfall or snowpack conditions in the basin that would lead to high flow events. SNOTEL sites would be distributed through the basin to provide a good representation of average basin-wide conditions, and sites would be selected to capture critical elevation ranges. The following costs do not include initial site installation costs. Operation and maintenance activities include performing manual snow surveys at each site two times per year, replacement of sensor fluids annually, maintaining site power, animal control, and quality checking data from all of the sensors. Since the sites are remote, measurement and maintenance activities require helicopter support. Coordination with NRCS gives an estimate of $25,000 per year per SNOTEL site or a total of $50,000 per year for the two SNOTEL sites. As the study progresses, consultation with the study partners will determine the optimum course of action.

9.3.3. Early Warning System Effectiveness

The early warning system would function by providing real time data of conditions in the basin which would alert the City's staff if conditions in the basin above the project indicate a landslide occurred creating the potential for a surge release event. The combination of real-time rainfall and streamflow data would detect if a sudden drop in discharge occurred during a rainfall event which is an indicator of a landslide occurring in the basin. The warning could be set up to alert operators by phone to visually check the basin status and make a decision based on event conditions if an evacuation notice is warranted downstream of the diversion dam.

The effectiveness of the warning system is dependent upon how much advance notice personnel can provide for an evacuation. The data available from the surge release event that occurred on Spruce Creek was used to estimate how much time could be available between the occurrence of a landslide that interrupts flow and the release of water stored behind the landslide. Gage records show that peak flow occurred on 11 October 1986. The time of day of the peak flow is not recorded; for the purpose of estimating the delay time, the event is assumed to be uniformly to have started at any time between 0000 and 2400 on October 11 with the average estimate of occurrence being at 1200 (Figure 50). The USGS report summarizing the 1986 rainfall flood events (USGS WRI 87-4278) includes data on local rainfall intensity. For the purpose of this exercise, it is assumed that the rainfall that induced the landslide occurred between 1800 and 2400 on October 10 with the landslide blockage of discharge occurring in this period with an average estimate of occurrence at 2100. While it is also possible that the landslide occurred during the earlier period on October 10, some rainfall would be needed prior to landslide initiation to saturate the soils and initiate movement.

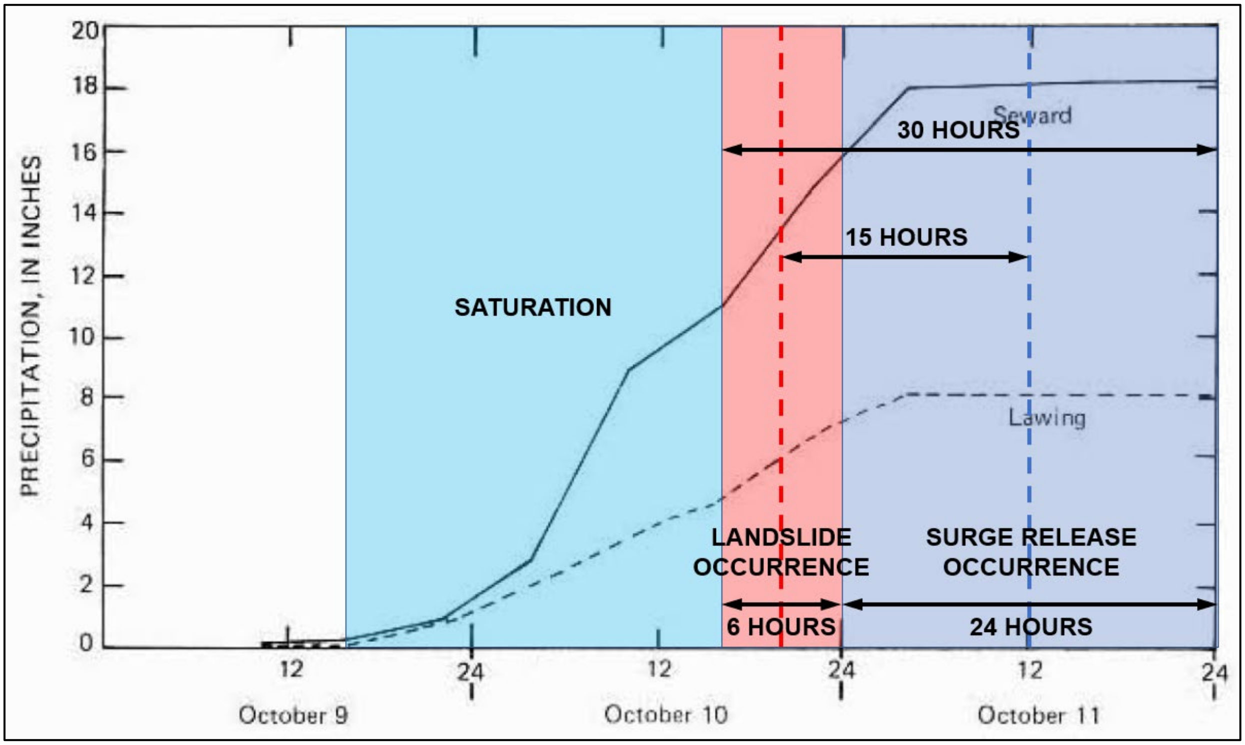

Figure 50. Hyetograph and Analysis of 1986 Precipitation and Gage Data.

Analysis of the records and assumptions provides a potential warning time of 0–30 hours between landslide occurrence and surge release resulting in peak flow with an average value of 15 hours. It is possible that the landslide could have occurred earlier during the event providing a greater delay between landslide and surge release.

With instrumentation in the Lowell Creek Basin, relationships between precipitation intensity and debris volumes observed at the tunnel outfalls could be studied and warning time estimates could be refined over time as observations are made.

LifeSim analysis of the basin indicates that a minimum warning time of 8 hours is needed to improve the effectiveness of the warning system. Warnings issued less than 8 hours before an event can potentially increase risk in Seward because more people could be in transit when high flows overtop the dam and affect the community. Due to the uncertainty of the system's ability to provide adequate warning time, the warning system was removed from consideration in the alternative plans. The stream gage at the outlet of the system was retained in the alternative plans to assist with debris removal operations at the outfalls.

9.4. Project Inspections

Project inspections to assess the condition of the structures in the flood diversion system are an important part of determining maintenance needs. The existing project is inspected annually by engineers from the Alaska District. The inspection includes a visual inspection of the inside of the tunnel, and the dam faces with measurement of distressed areas to track concrete abrasion over time. Inspections are needed to determine when concrete maintenance is required. Inspections would take a four-person team approximately one day to inspect the entire project and approximately three days of office time to compile the information and write an inspection report. Annual inspections will be required for all alternatives considered in this study.

9.5. Sediment Handling

The outfalls of the project must be maintained to prevent material buildup that would jeopardize adjacent facilities or block the system. It is expected that the system will deposit approximately 25,000 cy of material annually at the outfall. Over time, this material would accumulate and create a new alluvial fan at the location of the new outfall in the same manner that an alluvial fan is accreting at the location of the current outfall. Sediment handling is expected to be similar to what has taken place with heavy equipment pushing and moving the sediment towards deep water. Annual costs for these efforts have been provided by the City of Seward and are estimated to be $556,000. This value was increased to $580,000 to account for the occurrence of large events like the 1986 flood that deposit large single event volumes. A frequency analysis of these costs was performed and described in the Economics Appendix to include the uncertainty of larger event debris volumes and the effort that would be required to keep the outfalls clear. Including this uncertainty in larger event operations increases the average annual cost to keep the outfalls clear to $758,000 in the project’s current condition.

Alternatives 2, 3 and 4 do not change the volume of sediment deposited on the alluvial fan, but the outfall extensions alter the material placement allowing for more efficient handling procedures. This impact was modeled in the Economics Appendix by altering the frequency costs for sediment management. Events with percent exceedance frequencies ranging from 400% (three month average return interval) to 2% (50 year average return interval) used the handling cost of the 400% event, which reduced the average annual cost to maintain the outfalls to $178,000.

Alternative 5 intercepts some of the debris before it passes through the tunnel and it is assumed the 50% of the sediment handling will occur upstream from the tunnel(s), and the remainder will be at the outfall(s). Using the annual quantity of 25,000 cy and an upstream handling cost of $17 per cy yields $213,000 per year at the debris basin and $379,000 at the outfalls, which is 50% of the existing condition average annual cost. Combining these values yields a total sediment handling estimate annual cost of $592,000.

9.6. Assumed Total Maintenance Costs

The maintenance costs of the alternatives investigated for this study are summarized below in Table 19. Costs are expressed in 2020 dollars. Maintenance costs were estimated based on engineering judgment, historical information, and input from the National Infrastructure Maintenance Strategy — "Infrastructure Maintenance Budgeting Guideline."

The Alternative 5 existing tunnel maintenance cost is a middle ground between best-estimate of current costs and best-estimate with full refurbish of tunnel invert. Alternative 5 would experience less debris passing through the tunnel. However, significant debris events are the big driver for tunnel damage, and maintenance of the concrete surfaces of the diversion system is expected.

Table 19. Assumed O&M Costs of Alternative Flood Diversion Systems.

Alternative 2	Cost	Comment
Existing Tunnel	$ 200,000	Based on historic repair costs
Extended Outlet	$ 15,000	Extrapolated from historic repair costs
Protect Tunnel Inlet	$ 15,000	0.25% of Current Replacement Cost
Low Flow Diversion	$ 30,000	0.25% of Current Replacement Cost
Stream Gage	$ 25,000	$25k local share of Co-op agreement with USGS
Sediment Handling	$ 178,000	
Total	**$ 463,000**	
Alternative 3A	**Cost**	**Comment**
Enlarged Tunnel	$ 360,000	1.8 x Alt 2
Extended Outlet	$ 26,000	1.8 x Alt 2
Protect Tunnel Inlet	$ 15,000	Same as Alt 2
Low Flow Diversion	$ 30,000	Same as Alt 2
Stream Gage	$ 25,000	Same as Alt 2
Sediment Handling	$ 178,000	
Total	**$ 634,000**	
Alternative 3B	**Cost**	**Comment**
Enlarged Tunnel	$ 480,000	1.8 x Alt 2
Extended Outlet	$ 35,000	1.8 x Alt 2
Protect Tunnel Inlet	$ 15,000	Same as Alt 2
Low Flow Diversion	$ 30,000	Same as Alt 2
Stream Gage	$ 25,000	Same as Alt 2
Sediment Handling	$ 178,000	
Total	**$ 763,000**	
Alternative 4A	**Cost**	**Comment**
New Dam & Tunnel	$ 390,000	Same as Alt 3
Existing Tunnel	$ 50,000	25% of Alt 2 Costs (little use)
Extended Outlet	$ 26,000	Same as Alt 3
Protect Tunnel Inlets - New & Existing	$ 30,000	2 x Alt 2
Stream Gage	$ 25,000	Same as Alt 2
Sediment Handling	$ 178,000	
Total	**$ 699,000**	
Alternative 4B	**Cost**	**Comment**
New Dam & Tunnel	$ 519,000	Same as Alt 3
Existing Tunnel	$ 50,000	25% of Alt 2 Costs (little use)
Extended Outlet	$ 35,000	Same as Alt 3
Protect Tunnel Inlets - New & Existing	$ 30,000	2 x Alt 2
Stream Gage	$ 25,000	Same as Alt 2
Sediment Handling	$ 178,000	
Total	**$ 837,000**	
Alternative 5	**Cost**	**Comment**
Existing Tunnel	$ 150,000	75% of historic repair costs
New Debris Basin	$ 42,000	0.25% of Current Replacement Cost
Stream Gage	$ 25,000	Same as Alt 2
Sediment Handling	$ 592,000	
Total	**$ 809,000**	

10. REQUIRED FURTHER DESIGN STUDIES

This section describes future design efforts needed to complete the PED of a new project.

10.1. Geotechnical Investigation

A site investigation of any new project feature location needs to be performed before the creation of plans and specifications for construction. A thorough drilling program will be needed to establish foundation requirements for all project features, including new diversion dams, tunnels, and outfall structures.

10.2. Refined Numerical Study

The numerical model study of alternatives supports the decision-making process and provide sufficient information to make an informed decision between alternative plans. These models were simplified to focus on the consequence areas of concern and do not include the existing or proposed tunnel or dam. A detailed engineering study of the project components should be performed to refine the design and validate that tunnel capacity and project survivability goals are achieved. The refined design results should be validated with a physical model study. Numerical modeling of this level should be performed in a research facility with access to high performance computing assets such as the USACE Engineer Research and Development Center (ERDC).

10.3. Physical Model Study

A detailed physical model study in a hydraulic laboratory should be performed to validate the tunnel and flume capacity. Also, overtopping flow and scour resistance of the diversion dam need to be evaluated in greater detail. A scale model of the project would provide the best means to validate numerical model assumptions and results to ensure that design parameters have been met. Physical models of this type are investigated at the ERDC Laboratory in Vicksburg, MS.

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12. ATTACHMENTS

12.1. Original Contract Drawings