Lowell Creek Flood Diversion
Seward, Alaska
Appendix B: Geotechnical

April 2021

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MEMORANDUM FOR CEPOA-PM-C (Attn: Mr. Brent Howard)

SUBJECT: Geotechnical Appendix, Lowell Creek Flood Diversion, Seward, Alaska

1. Enclosed is the draft Geotechnical Appendix for the Lowell Creek Flood Diversion Integrated Feasibility Report and Environmental Assessment, Seward, Alaska.

2. Questions should be addressed to Andrew Romero at 907-753-2784 or Coleman Chalup at 907-753-2686.

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Appendix B: Geotechnical

Integrated Feasibility Report and Environmental Assessment

Lowell Creek Flood Diversion

Seward, Alaska

Prepared By:

U.S. Army Corps of Engineers

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1.0 INTRODUCTION

1.1 Location and Purpose

The Lowell Creek Flood Diversion project consists of a diversion dam and tunnel located approximately 0.1 mile west of Seward, AK at the end of Lowell Creek Canyon. This facility diverts stream flow from the old channel through Bear Mountain and into Resurrection Bay, at the south end of Seward. The original constructed project authority is the Flood Control Act of 1936 (Public Law (P.L.) 74-738). The authorized project purpose is flood risk management, by preventing inundation of the City of Seward.

As of November 2007, in accordance with the Water Resources Development Act of 2007 (P.L. 110-114, Sec. 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 15 years after the enactment of this Act (November 2022), whichever is earlier.

1.2 Current Stage of Work

This appendix describes existing geologic and geotechnical information for the Lowell Creek Canyon and Resurrection Bay region and preliminary geotechnical recommendations for the Tentatively Selected Plan (TSP).

1.3 Feasibility Studies Requirements

The following planning objectives were identified for this feasibility study.

- Reduce risk to public health, life, and safety from flooding of Lowell Creek to the City of Seward, Alaska
- Reduce flood damages to property and critical infrastructure in the City of Seward
- Reduce the cost of emergency response and management of post-flood event cleanup

2.0 LOWELL CREEK DIVERSION DAM AND TUNNEL

The original structure was designed and constructed in the late 1930’s to early 1940’s. The initial design has functioned as intended for over seventy years of operation. However, the structure was not constructed to modern standards that have been implemented since the initial design. In addition, the City of Seward was built upon an alluvial fan which includes filled-in sections of the old Lowell Creek channel and flume structure. Due to the loss of channelization, potential flows overtopping the dam facility would disperse throughout the city, causing unpredictability in determining the flow path and associated damages (Figure 1).

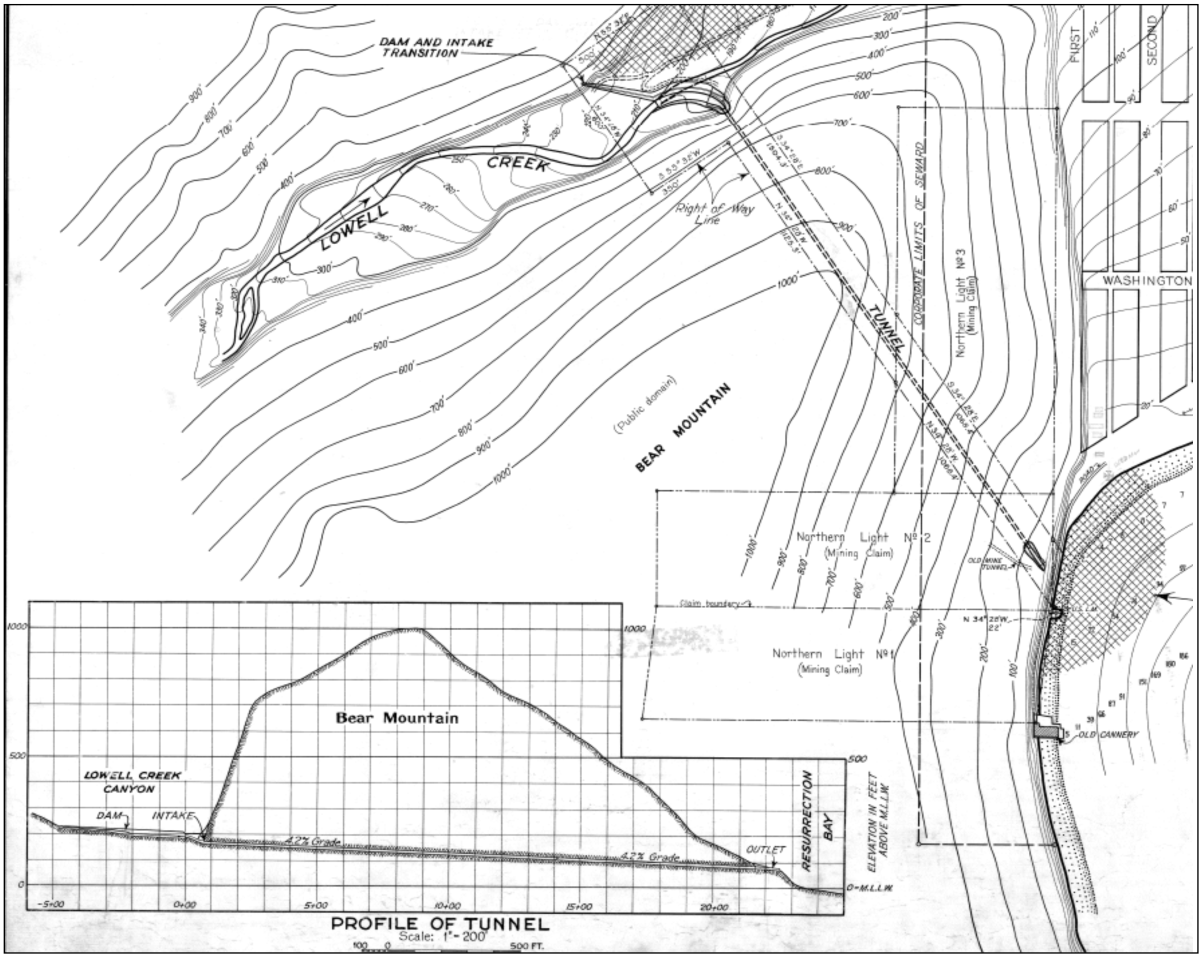

Figure 1. Project Overview.

2.1 Dam

The diversion dam consists of a 450 feet (ft) long rock-filled embankment with a crest elevation that varies from 203.2 to 225.7 ft North American Vertical Datum of 1988 (NAVD88) and a maximum height of 25 ft as measured from tunnel entrance end of invert to the spillway crest. Figure 2 presents a typical dam embankment cross section. The dam is designed to divert water into the tunnel and is not intended to impound water for long periods. The upstream slope at two horizontal to one vertical (2H:1V) is lined with a reinforced concrete slab. The downstream slope at two horizontal to one vertical (2H:1V) is lined with a cement grouted rock fill. The rock fill for the embankment was specified to range in size from 0.5 cubic ft to 27 cubic ft, of which not less than 25% of the stones shall be 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 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 is tied into the tunnel entrance, which is cast into 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 currently functional.

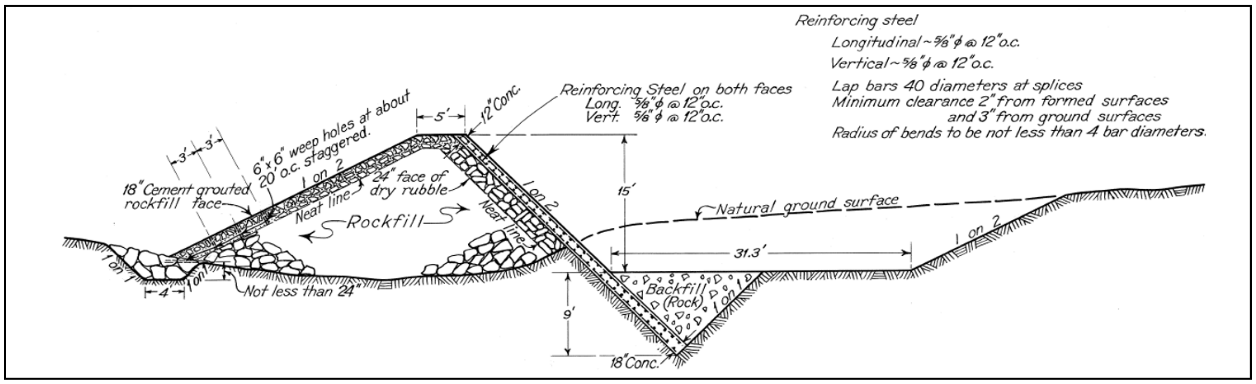

Figure 2. Typical Embankment Cross-Sections C-C.

The City of Seward placed a water line through the left abutment in 1985. During the installation of this water line, a section of the dam was removed to facilitate construction. During the rebuilding of this dam section, fill soil was used as core material for the dam. It is not known what composition or compaction requirements were required for the backfill material. Additional information will be gathered during the design phase, which could require borings, test pits, or geophysical methods.

2.2 Tunnel

The tunnel consists of a ten-ft diameter concrete lined horseshoe-shaped tunnel through Bear Mountain, which is 2,089 ft long with an average grade of 4.2%. Figure 3 presents typical tunnel cross sections. The tunnel was designed with a sharp drop at the intake transition; this accelerates the water to approximately 43 ft per second and facilitates debris transportation through the tunnel. The tunnel was constructed with drill and blast techniques. The bedrock was supported with timbers and lagging until placement of the tunnel concrete liner. It is believed the timber supports were left in place during liner construction and contact grouting within the timber holes was not performed after the liner was emplaced. The crown of the tunnel is assumed to have not been fully contact grouted and voids are still present within this section, reducing the liner's overall structural integrity. The tunnel was designed to be lined with concrete throughout the length of the tunnel and was originally armored with 40-pound railroad rails welded to the channel cross ties embedded in the invert. Sheet 2 of the 1945 original drawings has details that can be found in Appendix C: Hydraulic and Structural Design. The outside curve side of the tunnel is also rail lined at the intake. Spaces between rails have been filled with concrete during subsequent tunnel repairs. The tunnel capacity, assuming the spillway crest is filled to full capacity, has been calculated at 2,800 cubic ft per second (cfs).

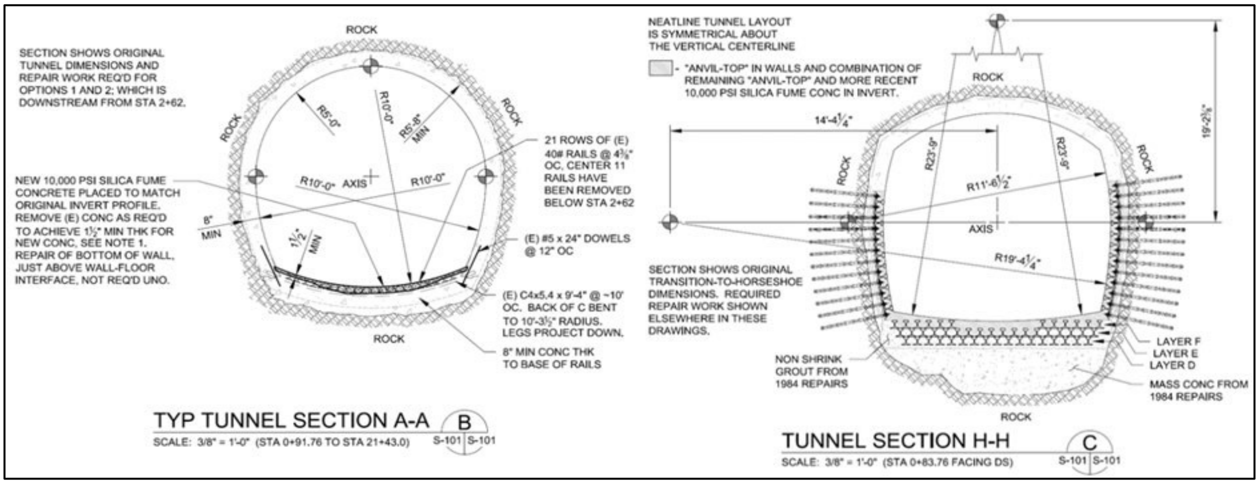

Figure 3. Typical Tunnel Cross-Sections.

The tunnel exits into a trapezoidal concrete flume 10 ft wide at the bottom and 109 ft long. The flume invert is 70.5 ft NAVD88, which allows for the accumulation of debris that is carried through the tunnel. The flume exits over a near-vertical rock cliff. At the toe of the cliff, the debris forms a channel which continues about 500 ft into tidewaters. A two-lane bridge crosses the channel about 100 ft from the toe of the mountain. The bridge has been known to become inundated with debris carried through the tunnel during large rainfall events.

2.3 Spillway

The emergency spillway is an uncontrolled weir with a discharge capacity of 1,700 cfs. The downstream side of the dam is sloped at three horizontal to one vertical (3H:1V) at the spillway and is protected by the same grouted rock fill as the main embankment. The spillway is a 40 ft long section of the dam with a crest elevation of 199.0 ft NAVD88. Figure 4 presents an embankment cross section at the spillway. Additional details can be found in Appendix C: Hydraulic and Structural Design.

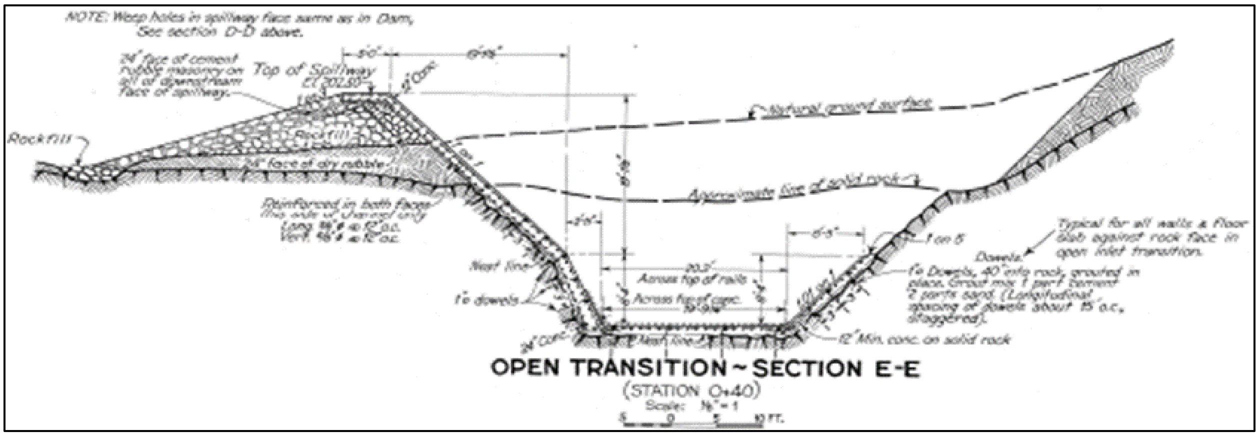

Figure 4. Typical Embankment Spillway Cross-Section.

3.0 SITE GEOLOGIC SETTING

Seward is located on the Kenai Peninsula at the north end of Resurrection Bay. The Kenai Mountains are composed primarily of sedimentary rocks that show a wide range of character and varying degrees of metamorphism. The main composition of these mountains is predominantly argillite, slate, and greywacke. The material was originally deposited as impure sand and mud. With time and pressure, the sediments were transformed into shale and sandstone with a certain fine-grained content. The rock has been further altered during folding of the mountains due to plate tectonics. The common geologic structure now appears as hard shale, argillite, greywacke, and impure quartzite, although locally metamorphism has proceeded far enough to convert the rock to slate or schist. Surface weathering in this area is significant with temperature fluctuation (freeze/thaw cycle) and high rain quantities noted for the surrounding geographic regime. With these factors the rock structure within this drainage basin produces great quantities of trap rock or shingle, which has a very flat angle of repose and is readily transported by water action. Figure 5 provides a geologic map for the project vicinity, indicating the major rock formations present. The City of Seward was built on the alluvial fan delta (Qf unit) made from sediments deposited by Lowell Creek. The location of some major landslide deposits within the upper Lowell Creek drainage basis are shown in Figure 5.

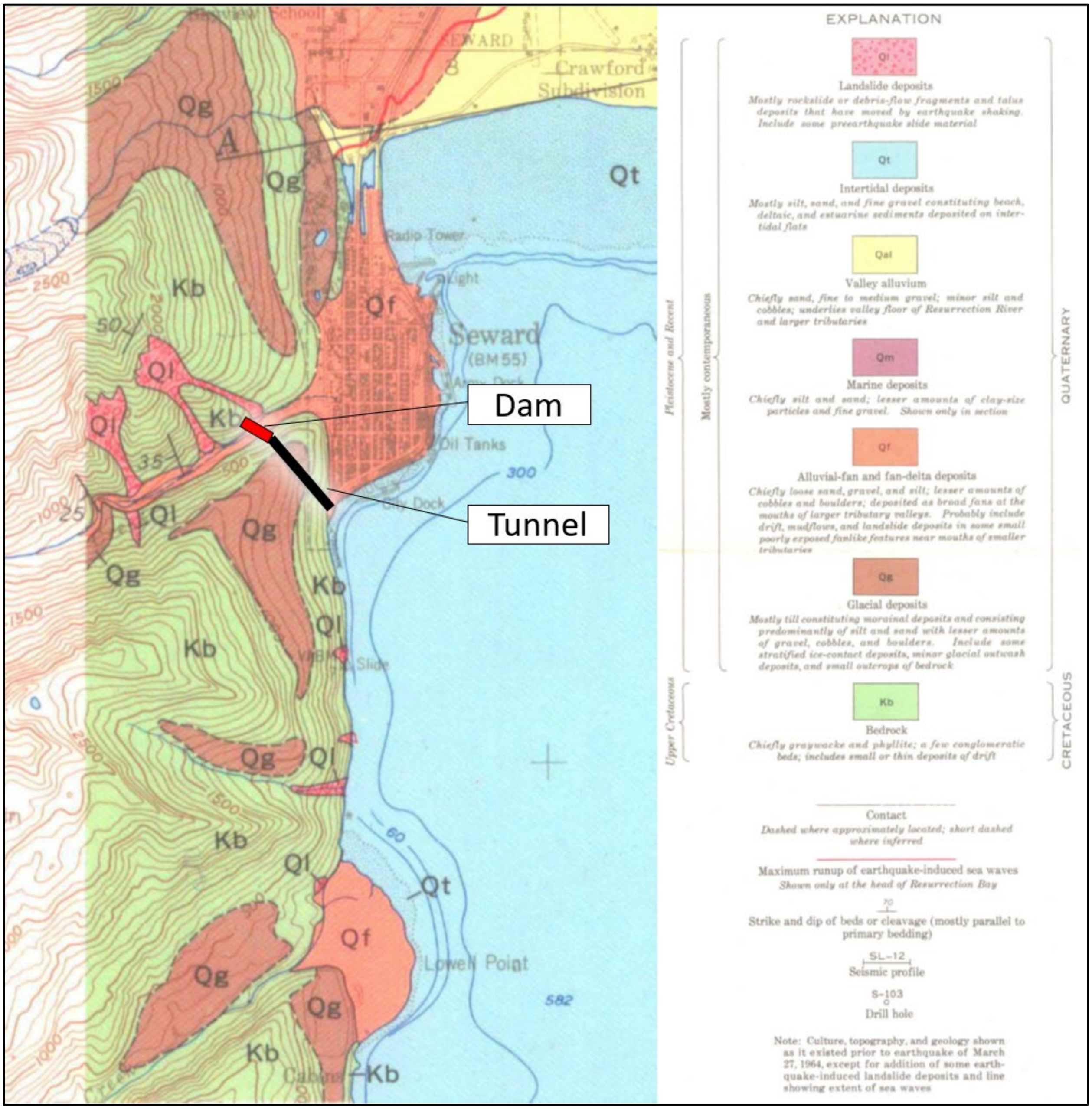

Figure 5. Geologic Map (*from* Lemke 1967).

The 1946 Operation and Maintenance Manual has a very brief description of the rock conditions during construction. It states "Rock encountered in construction of the tunnel consisted of shale, slate, and graywacke in varying degrees of hardness. Some earth seams were encountered at the lower end. Ground water occurs throughout the length of the tunnel and is relieved by numerous weep holes through the tunnel lining. The discharge capacity of the tunnel, with water depth of 8.3 ft, is approximately 3,150 cubic feet per second". Further analysis conducted during the feasibility study estimated the capacity of the tunnel to be 2,800 cfs.

The 1994 Flood Damage Reduction Revised Reconnaissance Report states:

> “The area is mapped as being part of the Upper Cretaceous Valdez Group described as undivided, dark gray, thin to thick bedded sandstone, siltstone and mudstone flysch (turbidite). The sandstone is fine to coarse grained and mainly composed of plagioclase, quartz and igneous rock fragments. The rock fragments constitute as much as 40 percent of the sandstone. Conglomeric sandstone, with sedimentary clasts, is widely distributed, occurring at the base of some sandstone beds. The Valdez Group beds are several thousand feet thick, and structure indicates a turbiditic depositional environment” (USACE 1994).

3.1 Bedrock Geology

Alternating units of greywacke and phyllite constitute virtually all of the bedrock in the immediate vicinity of Seward. Rock in the site area falls within the greywacke complex which predominantly consists of shale. It is through the shale member that the tunnel passes. The shale bedding is steeply dipping at about 65 degrees to the west and strikes roughly north-to-south. The rock cleaves parallel to the bedding planes. The shale appears quite competent for the tunnel. The main structural trend of the rocks in the Seward area is from near north to approximately north 20 degrees east. Bedding and cleavage commonly dip 70 degrees west or northwest to near vertical.

Small faults, shear zones, fractures, and joints are common. Rock strata are commonly offset vertically a few inches to several feet along these faults. The shear zones are mostly less than 5 ft wide and are commonly made up of angular pieces of greywacke or phyllite a few inches to a few feet long, though some shear zones are composed of finely ground rock fragments or a bluish-gray clayey gouge. The more massive graywacke sections are characterized in many places by a major and a secondary joint set system. North of Lowell Point, where the joints are well exposed, the major set strikes north 60-70 degrees west and dips approximately 85 degrees northeast and the secondary set trends northeastward. Most of the joints are filled with quartz veins but some are filled with calcite.

The rocks in the Kenai Peninsula bordering Resurrection Bay are of the greywacke complex which forms a crescent from the southern tip of the Kenai Peninsula northeast to Valdez, then eastward towards Yakutat. The greywacke series is composed of conglomerate beds and thick beds of shale with some thin limestone members.

Bedrock outcrops 1 and 2 identified in Figure 6 are located just upstream of the existing tunnel entrance. Outcrop 1 consists of thin to thickly bedded graywacke bearing N10-

13°E 72°W. The rock is hard and moderately jointed. Thick vegetation hides most rock on the left bank. Outcrop 2 is in two locations: the downstream portion is an outcrop of interbedded graywacke and shale bearing N-S 85°W, while the upstream portion is a fissile shale outcrop bearing N10°W 80°W (Figure 7).

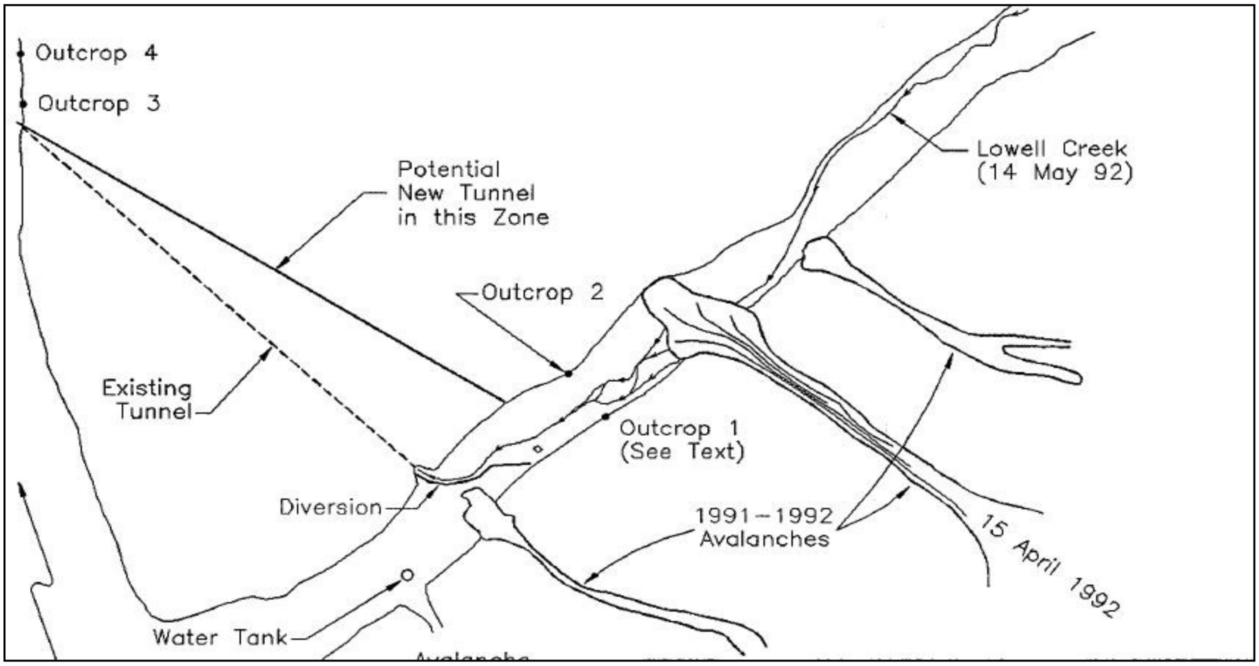

Figure 6. Bedrock Outcrop Map Upstream from Lowell Creek Tunnel Entrance.

Figure 7. Bedrock Outcrop 2 Photo Facing Downstream.

Bedrock outcrops 3 and 4 are located on the Lowell Point Road. Outcrop 3 is about 300 ft south of the existing outlet and consists of massive graywacke bearing N15°E 90°N. The bedding planes are undulating, giving the appearance of overturning. This area is moderately to lightly fractured, with occasional highly fractured zones. Outcrop 4 demonstrates the complexity of local geology. The outcrop consists of four separate areas about 800 ft south of the existing tunnel outlet. The outcrop starts with an exposure of interbedded shale and graywacke bearing N30°E I6°N near road level, being folded to an attitude of N17°E 40°W, 50 ft above the road and 50 ft north. The rock is further folded to an attitude of N15°E 84°E, 55 ft above the road and N63°E 77°S, 60 feet above the road, with dips varying 20 to 30 degrees within a short distance. Shale layers within this area are crushed and thinned between competent graywacke beds. The area may represent local recumbent folding with tight bedding plane faulting in the less competent shale.

3.2 Unconsolidated Deposits

The valley above the project is “V” shaped with very steep side slopes. Slope overburden in the lower 0.75 miles above the project is shallow colluvium. About 40% of the basin has vegetative cover consisting of low-growing alders, small shrubs, and small isolated patches of conifers near the creek channel. Snowfields in the upper part of the watershed cover about 10% of the basin. Most of the remainder appears to be solid rock, with near-precipitous slopes. Unconsolidated glacial and fluvial deposits overlie the bedrock except on the steep, higher slopes where rock is exposed. Remnants of lateral moraines flank the northeast margin of the Lowell Creek basin and are found throughout the upper portion of the drainage. Morainal deposits and side-channel alluvial fans provide a large source of sediment to the stream. According to the 1994 flood damage recon report (USACE 1994), lateral moraines in the upper Lowell Creek valley contain 1991 slide scars. The volumes of these slides are unknown.

The upper valley has very steep, essentially un-vegetated side slopes consisting of thick lateral moraines. These moraines continue upstream to remnant snowfields, where loose lateral and ablation moraines cover the valley. This material is very permeable and subject to continuous mass movement. During a site visit in October 2017, the team met with Matthew Balazs, a PhD candidate from University of Alaska, Fairbanks, who was investigating a lateral moraine located about 1.25 miles upstream of the project. This moraine was estimated to cover an area of approximately 1,000,000 square ft, estimated from satellite imagery. The height of the moraine is unknown; however, if the average thickness of the deposit were 30 ft, the volume of material in the moraine would be approximately 1,000,000 cubic yards (cy). Figure 8 presents photos of canyon deposits. The left photo is lateral moraine 1.25 miles upstream from facility. The, right photo, ¾ mile upstream from facility, shows a landslide on the right side of the photo which shifted the channel left.

Figure 8. Lowell Canyon.

The canyon has numerous active landslides and avalanche chutes that extend down the valley sides to the streambed. The streambed material appears to be much finer below these active slides than in the upper portion of the basin, where the bed material appears to be mainly cobbles with some gravel. The lower portion of the stream exhibits a dual or braided channel pattern, which is probably the stream's response to the greatly increased sediment load below the active talus slopes. The upper two-thirds of the basin has a single channel, with no major source of sediment evident. A major source of sediment for the stream appears to be the active slides and the side channel alluvial fans in the lower portion of the basin. Two landslide scarps in Lowell Creek were measured on aerial photos and were approximately 750,000 and 3,800,000 square ft in areal dimensions.

The 1986 U.S. Geological Survey (USGS) flood report identified Lowell Canyon as an area of high potential for landslides, debris flows, and debris avalanches. While there is some information on landslides in this region, there is a lack of data and mapping of potential landslides within this valley. The University of Alaska Fairbanks is currently studying and mapping potential landslides within this valley and some of the surrounding areas, but the information from this research is not currently available and is not expected until the end of 2020. This data would allow for a greater understating in the potential for and magnitude of a landslide caused surge release within the Lowell Creek basin. The active slides in the lower portion of the basin, along with the apparently over-steepened morainal deposits, make Lowell Creek susceptible to surge-release flooding and debris flow. A surge release flood occurs when a landslide temporarily impounds water and then fails, leading to a surge of water and debris with peak flows that are higher than what would naturally occur (Jones and Zenone 1988).

Figure 9 presents photos of weathered rock faces and slope failures upstream of the diversion facility. Large amounts of material can come down the slides or come from over-steepened glacial deposits during intense rainfall, temporarily blocking the stream. The subsequent breaching of the slide material and release of impounded water can result in a much greater flood peak than would otherwise have occurred due to rainfall alone. In addition, the 1988 study evaluated the potential for surge-release floods in Lowell Canyon to be high. The assignment of potential for debris-laden surge-release floods was based on known past events and the assignment of potential for landslides or avalanches was based on the work by Lemke (1967), field reconnaissance in October 1986, and analysis of historical aerial photographs (Jones and Zenone 1988).

Figure 9. Lowell Canyon Weathered Rock Face and Slope Failures Upstream from Diversion Facility.

Damming and surge-release flooding occurred in the Seward area in October 1986 on Godwin, Lost, Box Canyon, Japanese, and Spruce Creeks, and are described within the main Integrated Feasibility Report and Environmental Assessment. Avalanche scars were observed in the Lowell Creek basin after the 1986 flood. Remnants of a debris flow were found below the lateral moraine deposits on the northeast margin of the basin. A snout-shaped front of flow containing boulders and debris was observed on the right bank about 1 mile above the project. According to the 1994 flood damage reconnaissance report (USACE 1994), there is a remnant of a massive debris slide from the left bank which ran up the right side, blocking the stream. The slide remnant is approximately ¾ miles upstream and the slide may have approached 1,000,000 cy in volume. According to the report, this landslide appears to have dammed the stream for some time, as the channel above the slide cuts through a 10 ft thick deposit of clay and silt. The toe of this slide is not identified as landslide material on the USGS maps in the 1986 Flood Report (Jones and Zenone 1988) or in Lemke (1967).

3.3 Seismic Loading

3.3.1 Background

The dam is located at latitude 60.103º, longitude −149.454º on the south coast of Alaska. Alaska is the most seismically active state in the U.S. An average of one magnitude 8 or greater earthquake has occurred every 13 years in Alaska, one M7−8 earthquake every two years, and six M6−7 earthquakes every year (Koehler et al. 2012). Crustal deformation in Alaska is dominated by the subduction of the Pacific Plate and the Yakutat microplate beneath the North American Plate (Koehler et al. 2012). Figure 10 shows the location of earthquakes with a Moment Magnitude (M_w) greater than 5.5 that have occurred between 1900 and 2004 in Alaska (Wesson et al. 2007).

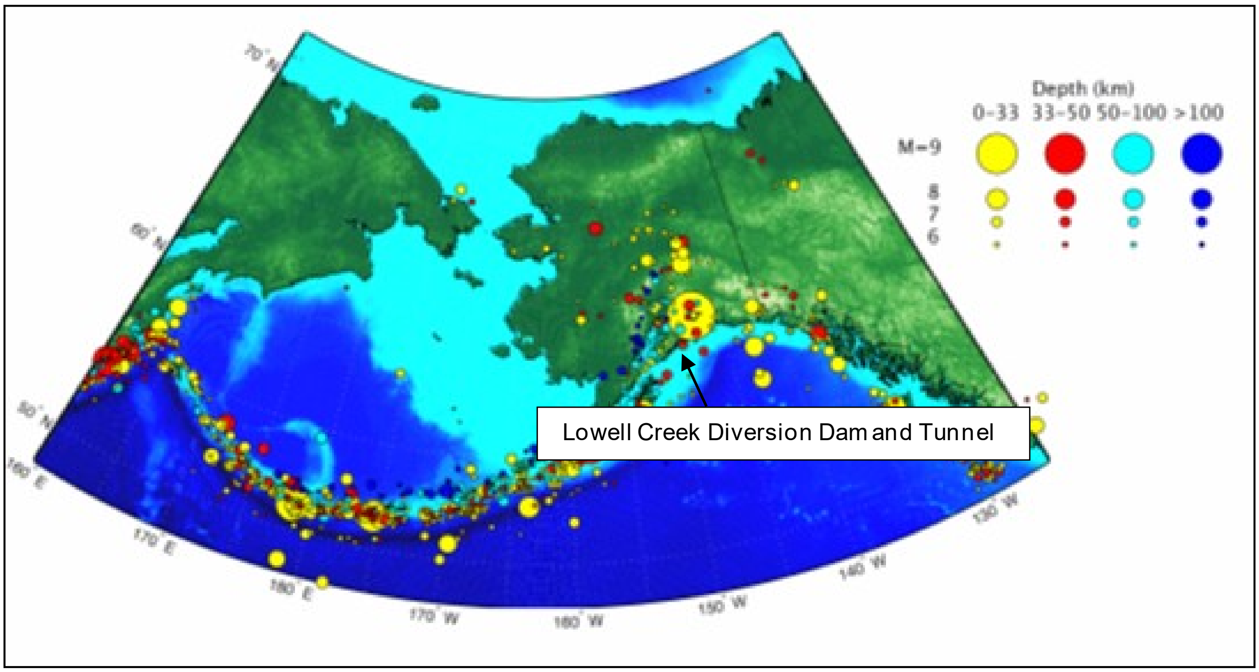

Figure 10. Alaska Earthquakes with $M_w \geq 5.5$ from 1900 to 2004 (*from* Wesson et al. 2007).

Most of the seismicity in Alaska is associated with the Alaska-Aleutian megathrust fault which runs along the Aleutian arc as illustrated in Figures 10 and 11. The fault is where the northwestward-moving Pacific Plate is subducted beneath the North American Plate (Wesson et al, 2007). The Alaska-Aleutian subduction zone is the source for the 1938 M8.3 Alaska Peninsula earthquake, 1946 M7.8 Unimak earthquake, 1957 M8.6 Fox Islands earthquake, 1964 M9.2 Prince William Sound earthquake, and the 1965 M8.7 Rat Islands earthquake (Koehler et al, 2012). The 1964 Prince William Sound M_w 9.2 event is the second largest earthquake in the world ever recorded. Other significant sources of seismicity include the Denali fault in south-central Alaska and a series of northwest-striking right-lateral strike-slip faults that run along the panhandle of southeast Alaska. These faults form the north-northeast boundary of the Pacific Plate at depth. The 2002 Denali fault M_w7.9 event is the largest earthquake to occur on land in the U.S. since the 1906 San Francisco earthquake. The Denali fault ruptured over a distance of 340 kilometers with up to 8 meters of offset from the event (Wesson et al. 2007).

Figures 11 and 12 show the location of active and suspected active faults of various ages in Alaska and in the vicinity of the project site, as obtained from a database of Alaska faults and folds compiled by Koehler et al. (2012). The age of fault movement ranges from known events in historical time (last 150 years) to Quaternary faults which have been recognized at the ground surface and have evidence of movement in the past 1.6 million years. Fault sources within 450 kilometers of the dam are presented in Koehler et al. (2012). The majority of Quaternary faults and folds in Alaska remain poorly characterized and information on the location, style of deformation, and slip rates for most faults is limited.

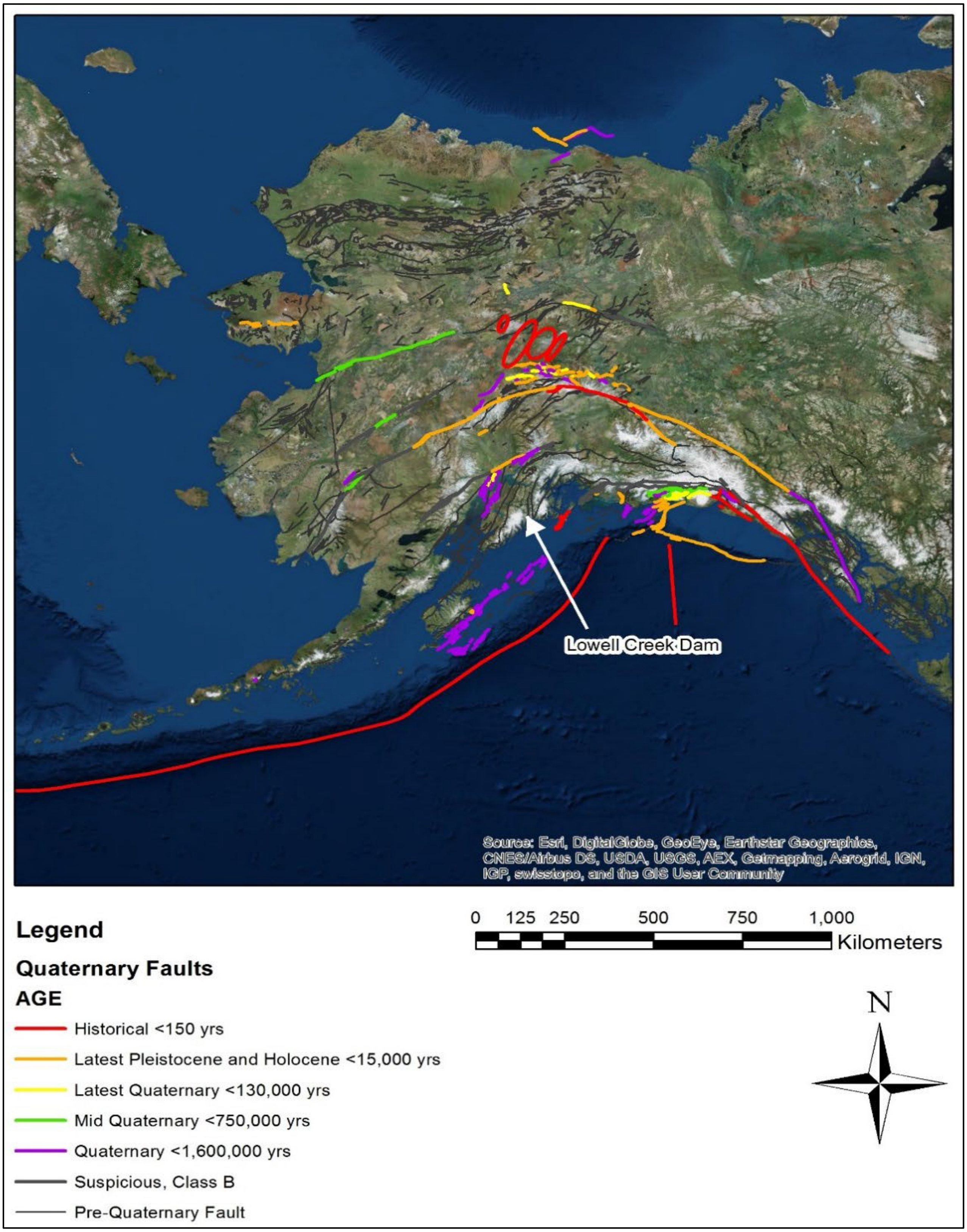

Figure 11. Location of Fault Sources in Alaska (Koehler et al. 2012).

The Aleutian arc is the most active seismic feature in the state and extends approximately 3,000 kilometers from the Gulf of Alaska in the east to the Kamchatka Peninsula in the west (Figure 12; USGS 2017). Nearly all of the plate boundary along the subduction zone has ruptured during the last century (Wesson et al. 2007). The Pacific Plate is moving northwest at a rate of approximately 60 millimeters per year at the eastern edge of the arc to approximately 76 millimeters per year near the western terminus. Motion along the eastern Aleutian arc, which is closest to the project, is characterized by arc-perpendicular convergence and Pacific Plate subduction (USGS 2017). Most of the seismicity here results from thrust faulting along the interface between the North American Plate and the subducting Pacific Plate. The closest megathrust earthquake to the dam was the 27 March 1964 M9.2 Prince William Sound earthquake. The rupture length for this event was approximately 700 kilometers long extending from Prince William Sound to the southern end of Kodiak Island. Damage was reported in Kenai, Moose Pass, and Kodiak, with the largest property damage occurring in Anchorage as a result of shaking from the main shock and subsequent landslides (USGS 2017).

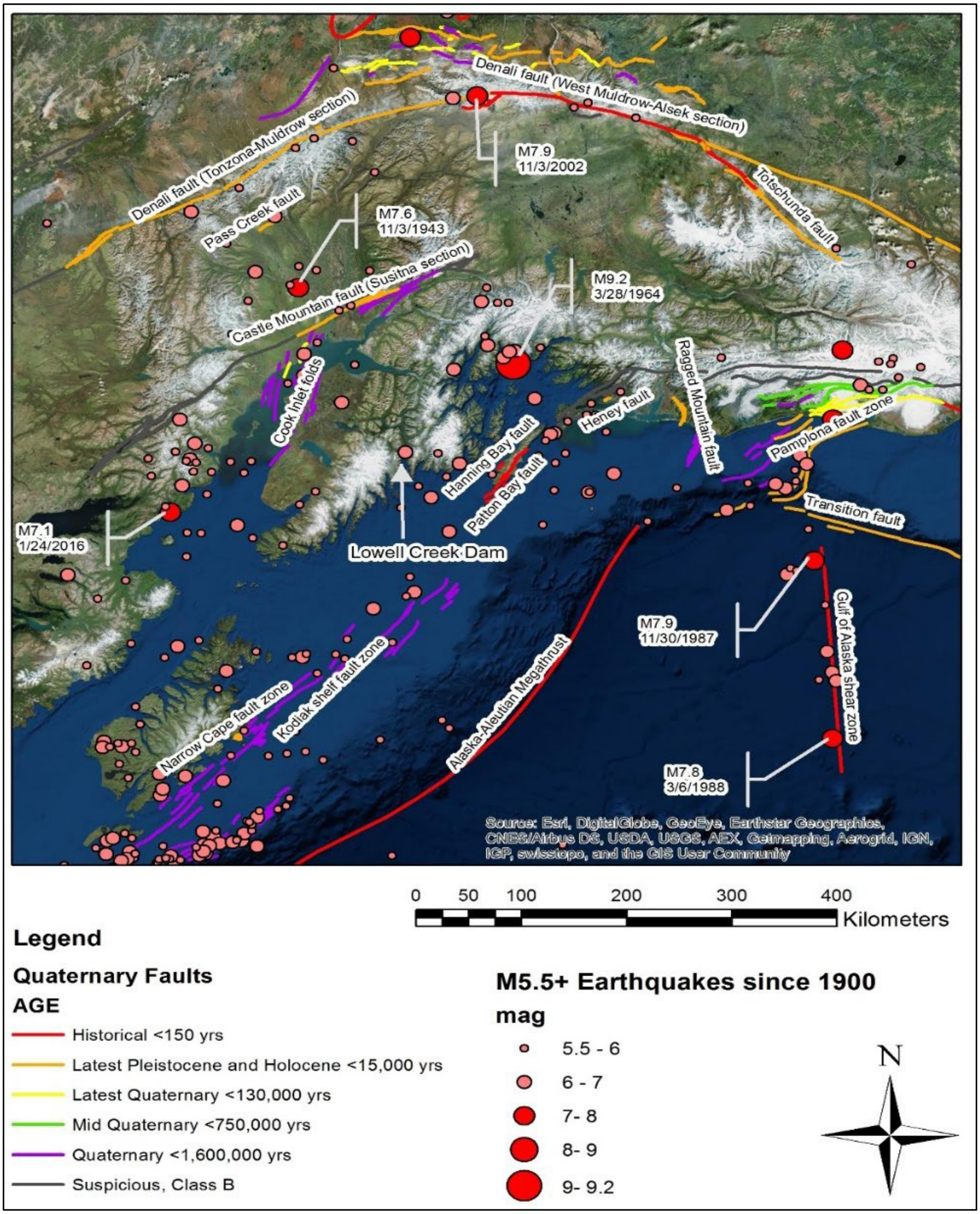

Figure 12. Faulting in the Dam Vicinity (Koehler et al. 2012, 2013) and M5.5+ Earthquakes since 1900 (USGS 2017).

The effects of the Prince William Sound earthquake on the town of Seward are described in Lemke (1967) and summarized as follows. Strong ground motion lasted three to four minutes in Seward. During the earthquake, a 50–400 ft wide strip of land along the Seward waterfront slid into Resurrection Bay as a result of large-scale submarine land sliding. Waves generated by the slide and later tsunami waves inundated the shore. Wave run up was as much as 30 ft above mean lower low water and caused significant damage to the town. Damage from the strong ground motions were comparatively minor. Tectonic subsidence of about 3.5 ft resulted in low areas being inundated at high tide. The earthquake reactivated old slides and triggered new ones in the mountains. Snow avalanches were triggered in Lowell Creek Canyon. Two such avalanches in the lowermost mile of the canyon reached the creek bed and piled up snow, rock fragments, and broken trees as high as 30 ft. Landslides within Lowell Creek Canyon from this event were found in the record obtained by the Alaska District. Additionally, there is no noting of damage to the tunnel caused by the earthquake and subsequent repairs may have fixed any damage that may have occurred.

There are no seismic monitoring instruments at the project. According to a Lowell Creek Tunnel repair report dated August 2001, the 1964 Alaska earthquake did not affect the project. Table 1 provides a list of Class A faults located within 450 km of the project site, with Class A faults defined as the existence of a Quaternary fault formed from tectonic movement.

Table 1. Class A Faults in within 450 km of Lowell Creek Dam (Koehler et al. 2012).

Fault Name	Activity (Years since last movement)	Distance (km)	Length (km)	Direction from Site
Patton Bay fault	<150	86		E
Hanning Bay fault	<150	92	14	E
Cook Inlet Folds	<130,000 to 1,600,000	98		NW
Kodiak shelf fault zone	<1,600,000	133	1405	S
Castle Mountain fault, Susitna section	<15,000	168	68	NW
Heney fault	<15,000	205	12	E
Alaska-Aleutian Megathrust	<150	228		S
Tenfathom fault	<1,600,000	271	65	E
Ragged Mountain fault	<15,000	272	104	E
Kayak fault	<1,600,000	273		E
Wingham fault	<1,600,000	280	24	E
Pass Creek fault	<15,000	286	49	NW
Transition fault	<15,000	304	697	E
Pamplona fault zone	<1,600,000	306		E
Narrow Cape fault zone	<15,000	337		SW
Denali fault, Tonzona-Muldrow section	<15,000	342	267	NW
Chugach-St. Elias fold and thrust belt	<130,000 to 1,600,000	347		E
Albatross Bank fault zone	<1,600,000	367		SW
Denali fault, West Muldrow-Alsek section	<150	369	1040	N
Denali fault, Farewell section	<15,000	381	455	NW
Susitna Glacier fault	<150	389	131	NE
Gulf of Alaska shear zone	<150	410	447	SE
Totschunda fault	<15,000	444	699	NE

The Patton Bay and Hanning Bay faults on Montague Island are the closest Quaternary faults to the site. Both faults are reverse faults that were reactivated during the major 27 March 1964 earthquake. These faults, identified and mapped by Plafker (1967), strike between N37°E and N47°E and dip northwest at 50°–85°. The blocks northwest of the faults were upthrown relative to the southeast side of the fault, and both sides of the faults were upthrown relative to sea level. The Patton Bay fault is a system of en echelon faults. The fault has been traced over a distance of 22 miles on land and at additional 17 miles on the sea floor. The maximum dip slip offset on the fault is 26 ft. The Hanning Bay fault is 4 miles long with a maximum dip slip offset of 20 ft. Plafker (1967) concluded that these faults lie within a tectonically important zone of crustal attenuation and maximum uplift associated with the earthquake and that there is no evidence to suggest that they form major tectonic boundaries.

Other faults close to the project include the Cook Inlet folds, the Castle Mountain fault, and the Kodiak Shelf fault zone as shown in Figure 12 and Table 1. The Cook Inlet basin lies on a northeast-trending forearc located between the Chugach and Kenai Mountains to the southeast, and the Alaska Range and Aleutian volcanic arc to the northwest. Deformation of the Cook Inlet basin started between Eocene and early Oligocene time. Deformation in the upper Cook Inlet is transpressional and resulted in a series of folds, faults, and eroded horst blocks (Haeussler & Saltus 2011).

The Castle Mountain Fault consists of two segments, the western Susitna segment, and the eastern Talkeetna segment. There is no evidence of post-Pleistocene surficial displacement on the Talkeetna segment (Wesson et al. 2007). However, an M_S 5.2 earthquake was interpreted to indicate slip at a depth of 13–20 kilometers along this segment by Lahr et al. (1986). Holocene surface displacement is evident on the Susitna segment (Wesson et al. 2007).

The Kodiak Shelf Fault zone includes the Kodiak Island and the Narrow Cape faults. These are high angle left lateral strike slip faults that strike about N45°E, sub parallel to the subduction-zone trench (Freymueller et al. 2013; Wesson et al. 2007).

3.3.2 Previous Seismic Evaluations

Only limited documentation of design of the embankment and diversion tunnel have been located and no information on seismic considerations during the original design have been found. Based on current review, no subsequent seismic evaluations have been performed.

3.3.3 Site Classification

The Lowell Creek Diversion Dam embankment is located within Lowell Creek Canyon, approximately ¼ mile upstream of the delta upon which the City of Seward is built. There is limited design and construction information available for the embankment. Based on cross sections of the embankment on design drawings, the rock fill embankment was placed on bedrock in some areas. The bedrock consists primarily of sedimentary rocks composed of argillite, slate, and graywacke with varying degrees of metamorphism.

Shear wave velocity (v_S) is the velocity that shear waves, such as those produced by an earthquake, will have as they pass through the soil or bedrock. For the purposes of estimating ground motions, the dam's foundation soil and rock profile can be classified based on the average shear wave velocity in the uppermost 100 ft (30 meters, v_s^{30}) of the site profile. The v_s^{30} is currently the preferred parameter for characterizing (classifying) the site conditions in developing estimates of ground shaking using ground motion attenuation relationships.

Directly acquired shear wave velocity data is not available for the site. Allen and Wald (2007) developed an indirect methodology to derive maps of v_s^{30} based on topographic slopes. Based on the topography at the dam site, the v_s^{30} is estimated to be approximately the B and C boundary (760 meters per-second) shown on the reference scale in Figure 13. The use of downhole vertical shear velocity profiling and or alternative geophysical surveys will be conducted from the surface to further refine the material's v_s^{30} value and provide accurate information for design purposes.

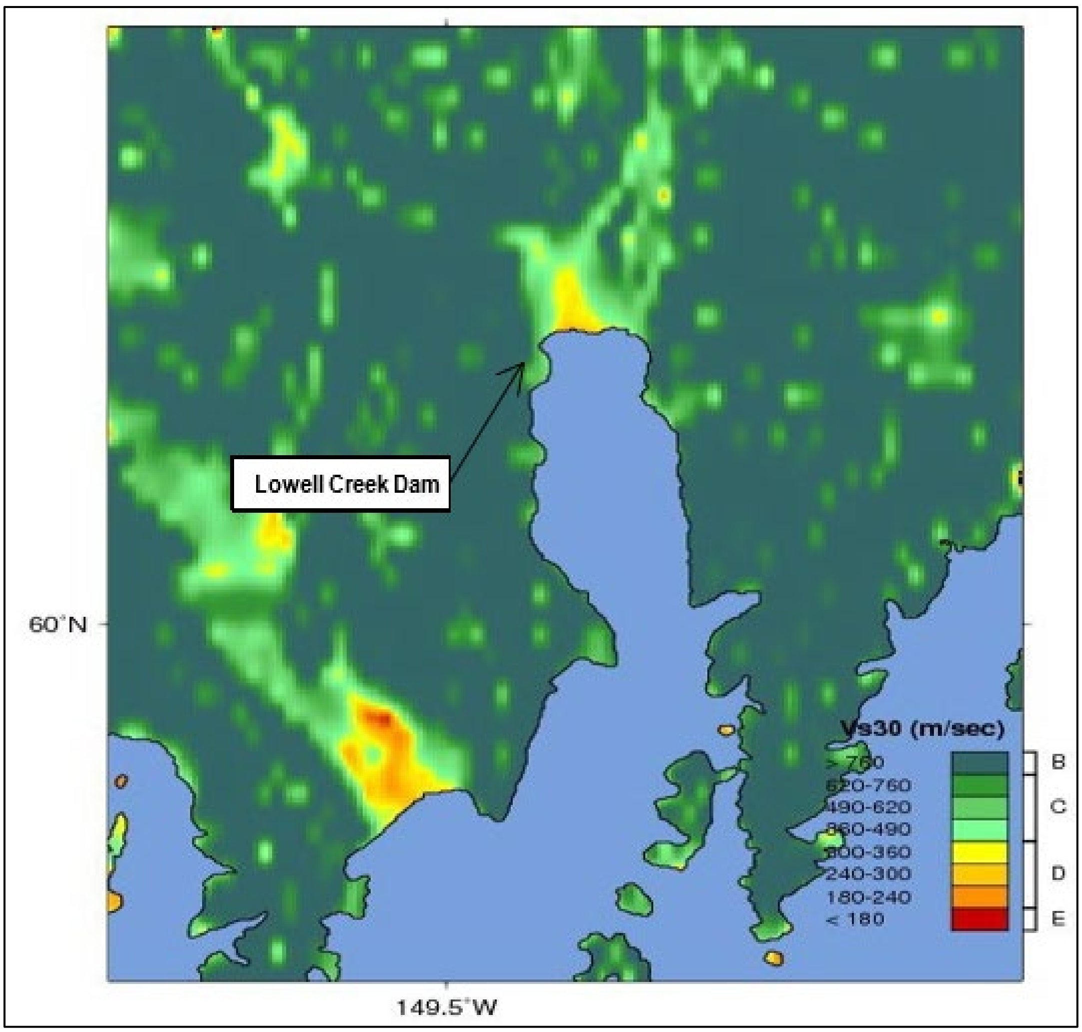

Figure 13. V_S^{30} Based on High-Resolution Topographic Data (*from* Allen & Wald 2007)

3.3.4 Seismic Hazard Curve

USACE design guidelines (ER 1110-2-1806) utilize an operating basis earthquake (OBE) and a maximum design earthquake (MDE). The probabilistically determined operating basis earthquake is considered to be an earthquake that has a 50% probability of exceedance in 100 years (i.e., 144-year return period) and is estimated from a probabilistic seismic hazard analysis (PSHA). The MDE is the maximum level of ground motion for which a structure is designed or evaluated.

For “critical” structures which are part of a high hazard project and whose failure will result in loss of life, the MDE represents the expected ground motions that could be produced by the maximum credible earthquake (MCE). The MCE is defined as the greatest earthquake magnitude that can reasonably be expected to be generated by a specific seismic source. The MCE determination includes both the expected maximum magnitude and the source-to-site distance. The MCE is an informed judgment based on seismological and geological evidence from a Deterministic Seismic Hazard Analysis.

The expected ground motion from the MCE may be produced either by an individual seismic source or by a composite of several seismic sources that could produce different shaking levels for different ground motion frequencies. The MCE is typically associated with the 84th percentile expected ground motion for major active faults and may be associated with the median (50th percentile) expected ground motion for potentially active faults (with slip rates of ~ 0.1 millimeter per year or less).

By definition, it is not possible to assign a return period to the ground motions produced by the MCE. However, the results of a PSHA are commonly used to estimate the approximate return period of the MCE. For structures that are part of a significant hazard project and whose failure will not result in loss of life, the probabilistically determined MDE is generally an earthquake that has a 10% probability of exceedance in 100 years (i.e., 950- year return period) (ER 1110-2-1806 7.a).

For the purposes of this risk assessment, a probabilistic return period and corresponding ground motion value were selected by the risk assessment team, as this was sufficient for determining the potential failure mode and consequences. A site-specific PSHA may need to be performed during the design phase for a more accurate characterization of seismic ground motions applicable to the TSP. The mean seismic hazard curve for the peak horizontal ground acceleration (PGA) shown in Figure 14 is based on a regional PSHA for Alaska (Wesson et al. 2007). This analysis provides a general estimation of the seismic hazard rate and frequency for the region around the City of Seward, Alaska.

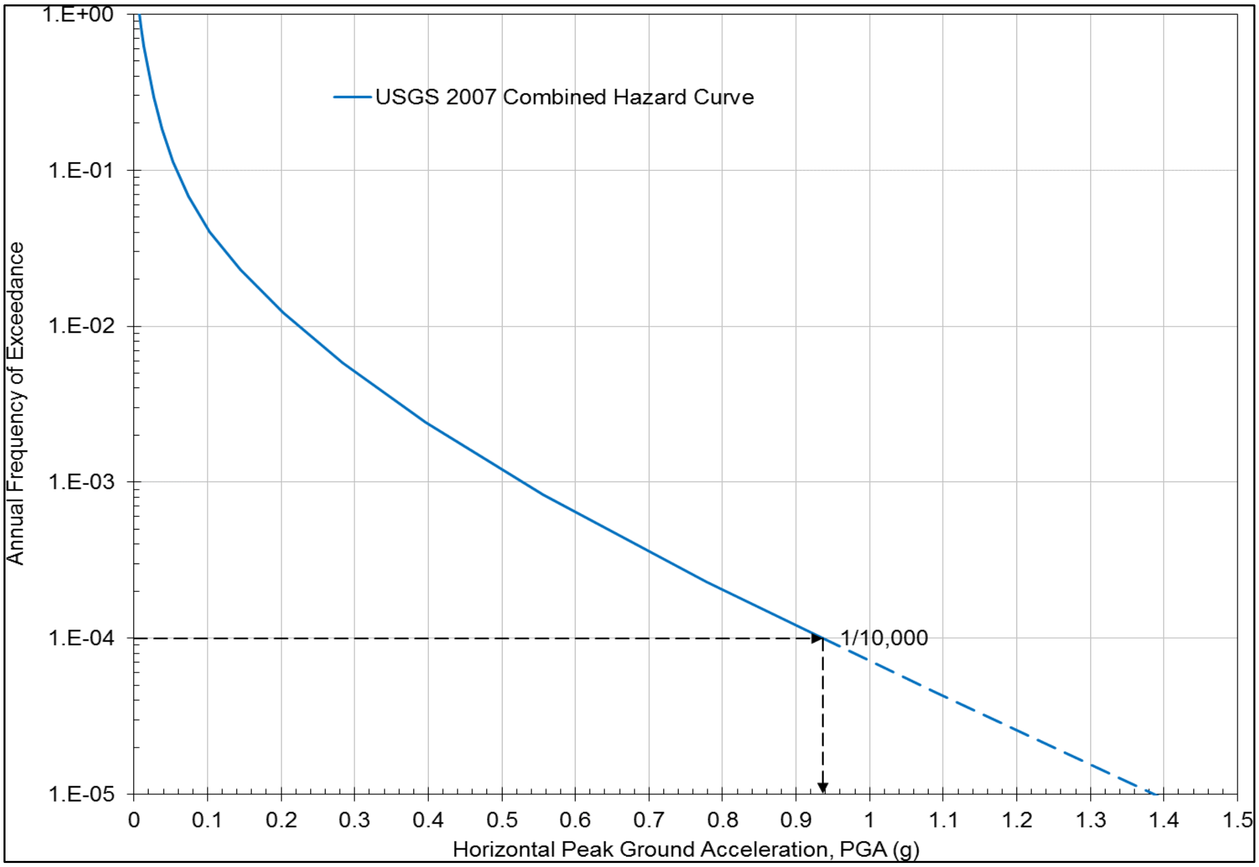

Figure 14. Seismic Hazard Curve for PGA (Wesson et al. 2007).

Time-independent probabilistic seismic hazard maps of Alaska and the Aleutians were developed by the USGS in 2007. The 2008 and 2014 updates to the National Seismic Hazard Map do not include Alaska. The 2007 seismic source model for Alaska considers uncharacterized and unrecognized fault sources, active faults with known parameters, and the Alaska-Aleutian megathrust associated with the subduction of the Pacific Plate. There is considerable uncertainty for annual exceedance probabilities (AEP) less than 1/10,000. The extrapolation of the mean hazard curve to remote annual exceedance probabilities (i.e., less than 1/10,000 AEP) is shown as a dashed line in Figure 14. The PGA corresponding to selected common values of return periods were interpolated from this mean hazard curve and are shown in Table 2.

Table 2. Peak Horizontal Ground Acceleration Summary (Wesson et al. 2007).

Earthquake	Return Period (years)	PGA (g)
OBE	144	0.265
MDE for "non- critical" structures	950	0.52
International Building Code "maximum considered earthquake"	2,475	0.68
Intermediate earthquake	4,950	0.8
International Committee on Large Dams Bulletin 72 (2010) earthquake	10,000	0.94

3.3.5 Deaggregation

The USGS has not published a web application to perform the seismic hazard deaggregation using the most recent Alaska PSHA (Wesson et al. 2007). The deaggregated seismic hazard for the PGA is shown in Figure 15 using the 1998 USGS *National Seismic Hazard Mapping Project* Hazard Maps for Alaska. Deaggregation corresponding to only the B and C boundary (at 760 meters per-second, Figure 13) v^{30} site class is available. The deaggregation suggests that the primary contributors to the seismic hazard at the site include an M_w 6.2 earthquake from shallow random sources at a distance of 11.4 kilometer, an M_w 7.6 earthquake from the subduction zone at a distance of 38.6 kilometers, and an M_w 9 earthquake from the subduction zone at a distance of approximately 38 kilometers. An updated evaluation of significant seismic hazard sources and associated ground motions will be provided as part of a current PSHA during the project design phase.

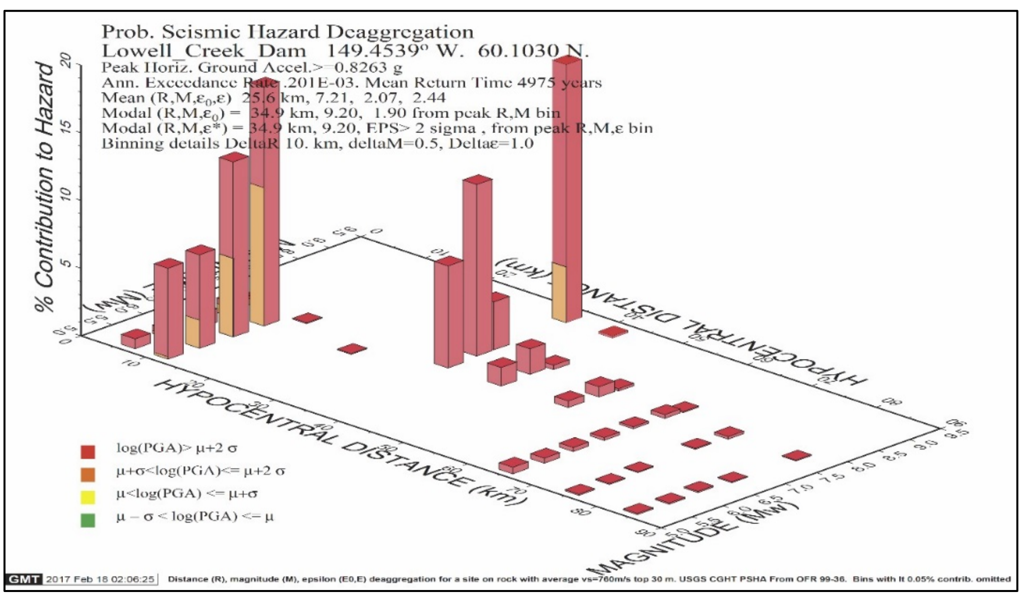

Figure 15. Probabilistic Seismic Hazard Deaggregation for PGA with 1% Probability of Exceedance in 50 Years (1/4,975 AEP) (USGS 1998).

4.0 ALTERNATIVES SITE INVESTIGATION REQUIREMENTS

Current geologic and geotechnical information for Lowell Creek Diversion Dam and Tunnel is not sufficient to support detailed design work. The dam and tunnel facility will require investigation and testing of the material at and around the dam structure. Geotechnical down-hole soil analysis and material testing will need to be performed to bedrock. Analysis of groundwater flows will also be conducted during the site investigation. Rock core sampling and testing for this facility will also need to be performed, along with detailed surface geologic mapping, for characterization and analysis of the rock mass structure. Probabilistic seismic hazard analysis will have to be performed for the project site, through downhole vertical shear velocity profiling and surveying. Testing of sediment physical properties pertinent to stream flow conditions may be required as well.

4.1 Alternative 1: No-Action

This option would not require any geotechnical analysis of the project, but additional data could be collected to aid in the long-term maintenance and repair of the tunnel and dam structures. A geologic/landslide study should be performed for the inlet, to develop the information/data to determine the potential for landslide and slope failures. Rock core sampling along the length of the tunnel would provide information on the current state of the rock structure as it impacts stability of the tunnel liner. From this information, the stability and erodibility of the surrounding material could be further understood, when tunnel liner failure occurs. Coring analysis of the liner and surrounding rock mass will be required for this and all other alternatives.

4.2 Alternative 2: Improve Existing Flood Diversion System

This alternative focuses on removing elements that contribute to the risks associated with flooding and public safety in the Lowell Creek basin. This alternative does not change the size of the existing tunnel and therefore does not impact the risk associated with flows above a 0.01 annual exceedance probability. What this alternative does do is remove trees large enough to get caught in the tunnel (NS3) upstream of the dam, constructs a canopy above the tunnel entrance to prevent blockage from a landslide (S18), rehabilitates the tunnel liner (S3), installs instrumentation (NS1), constructs a flow bypass for maintenance within the tunnel (S26), and improves the outfall of the tunnel (S14).

Construction of a canopy will require an in-depth investigation of the slope surface above the tunnel entrance. Rock coring will be conducted around the existing tunnel entrance where the canopy location is suggested. The material is expected to consist of loose soil and rock which will require field mapping. Data from the coring samples will consist of rock identification, mineral composition, geological structuring, and material strength. The flow bypass location has not been determined and a suitable path through the embankment foundation will need to be determined and the base material tested for its consistency and structuring. The outfall structure will require a geotechnical boring investigation of the underlying sediments to bedrock, in addition to rock core sampling on the slope around the outfall structure. Further investigation of the existing embankment and the surrounding area will also need to be performed. The focus of these investigations will be on the material construct/composition, soil density/compaction, and soil permeability.

Considerations will be made for precipitation gages, flow gage, thermometer, and earthquake monitoring (seismograph, geophone, or accelerometer) at the tunnel and dam structures in addition to other instrumentation (e.g. staff gages, video, or continual photo monitoring system). Core sampling within the tunnel of the liner and surrounding rock mass will also be required. In addition to evaluating the tunnels geologic surroundings, a PSHA would be performed on the project area. This data would allow for determining the viability of the existing tunnel in the long term, and analysis any potential issues that may occur due to an earthquake.

An additional consideration is that the alternative may require dredging of the material deposited at the outfall, consisting mostly of coarse sediment. A disposal location within Resurrection Bay will be required if dredging is deemed necessary. Chemical testing for potential contamination within the material to be deposited will also be needed. Dredging of the material will require periodic maintenance, to be done by mechanical dredging due to the location of the channel and size of sediments. A split hopper barge will transfer the material offshore to the disposal site. This material could be mined depending on its physical properties, by use of dump-truck and drainage/dewatering facilities. Mining of material is dependent on local interest and permitting.

4.3 Alternative 3: Enlarge Existing Flood Diversion System

This alternative focuses on reducing elements that contribute to the risk in Lowell Creek. This alternative expands the size of the existing tunnel from its current 10 ft to 18 or 24-ft (S4), which has an impact on risk associated with flows up to 8,400 cfs (probable maximum flood (PMF)). This alternative also incorporates the site investigation requirements and measures stated for Alternative 2, Section 4.2.

With the expansion of the tunnel in consideration, a geotechnical investigation including field geologic mapping and core sampling of the rock surrounding the tunnel will need to be conducted. The data collected from this investigation is needed for analysis of the surrounding geologic structure, rock strength and discontinuities, and overall integrity of the material. This will also establish the geologic stratigraphy and lithology, surrounding and impacting the tunnel structure.

4.4 Alternative 4: Construct New Flood Diversion System

This alternative focuses on eliminating elements that contribute to the risk from Lowell Creek. This alternative would add a new 14, 16, 18, or 24-ft diameter tunnel and a diversion dam upstream of the existing facility (S1 & S8), this design would have an effect on the risks associated with flows up to 8,400 cfs (PMF). Figure 16 shows a general proposed location and layout for this alternative. This alternative also incorporates measures from Alternative 2, with the development for a canopy over each tunnel inlets, and the same site investigation requirements stated in Section 4.2.

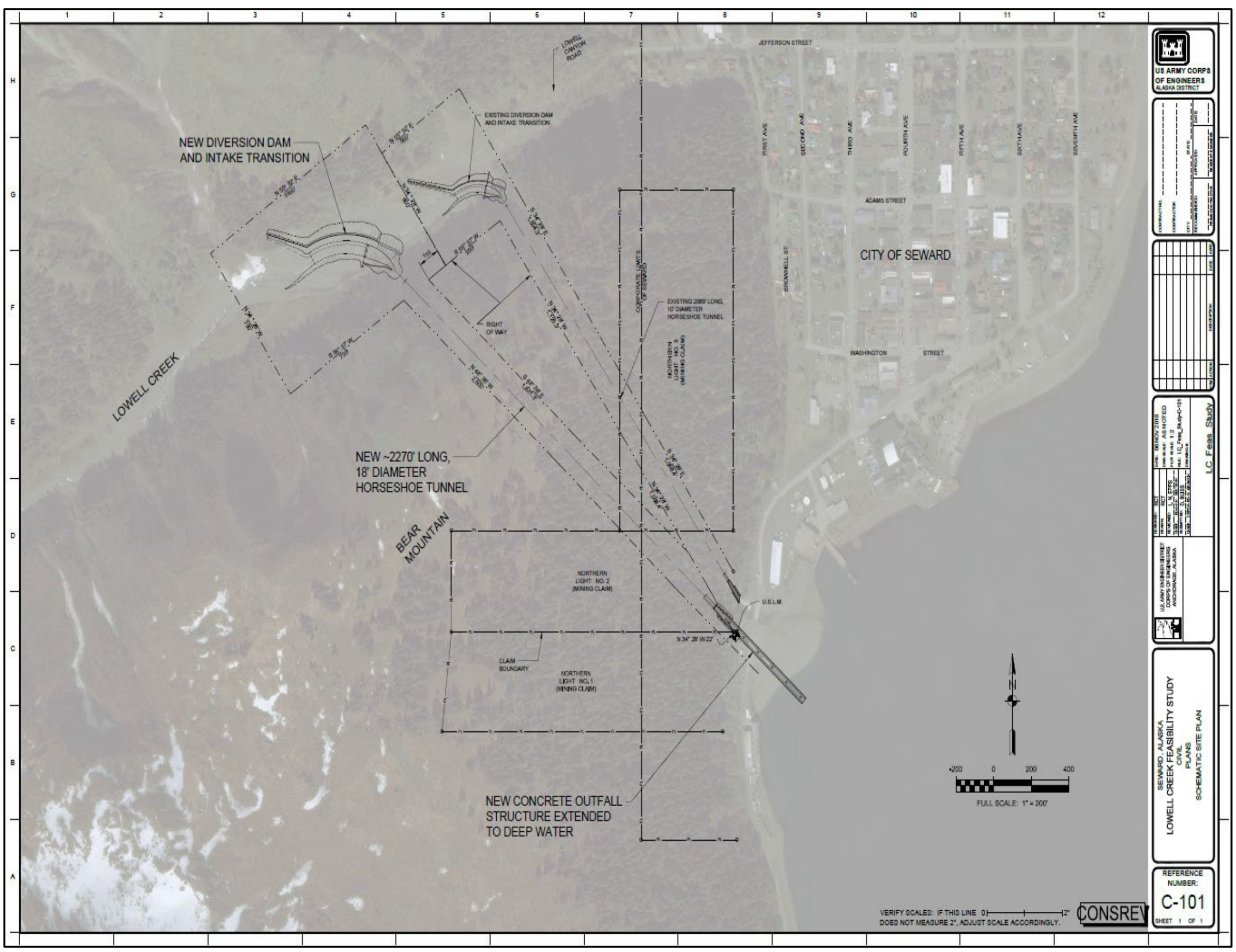

Figure 16. Alternative 4 Conceptual Drawing.

It is assumed that new tunnel construction will be by drill and blast methods; however, other methods such as Tunnel Boring Machines (most likely Micro-Tunneling) or

Sequential Excavation Method techniques will be acceptable. If drill and blast is utilized, smooth wall (or controlled) blasting techniques may be required to minimize overbreak and blasting effects outside the excavation lines. A stabilizing shotcrete liner with reinforcing wire mesh netting would be installed prior to forming and placing the final layer(s) of concrete for the liner. The use of rock bolts may be required to decrease the loading of unstable surrounding rock mass on the tunnel liner, transferring the loading to the more stable interior rock mass. The final concrete liner is assumed to be a single layer reinforced concrete design with weep holes. The length of the constructed sections will be defined by the geologic structuring and loading of the surrounding rock, based on the geotechnical and geological analysis performed on the existing tunnel and surrounding rock mass. Prefabricated concrete paneling may also be used for the tunnel liner.

Contact grouting will be accomplished after the concrete liner is placed to ensure full contact at the tunnel crown and invert connections. The invert will have a final armor of high strength concrete. The current proposal is for a trapezoidal flume shape with arched crown, but consideration may also be given to a round-shaped tunnel with a precast prestressed concrete liner. The invert will have a final armor of high strength concrete. The portal location for the new tunnel outlet will also be reviewed during design to determine if there is sufficient distance from the existing tunnel portal.

A geotechnical investigation including a geophysical survey and borings conducted to bedrock with associated soil and rock materials testing is required at the proposed dam location. Analysis of groundwater flow will be extended to the proposed dam location. Field geologic mapping and borings with rock coring will be required through Bear Mountain, at and above the proposed tunnel location to an approximate length of 1000 ft into the mountain in line with the projected tunnel alignment.

Field geologic mapping and borings with soil sampling and rock coring will also be needed around the inlet and outfall locations, as tunnel portals and the surrounding slopes often present the most challenging excavation and rock support conditions for a tunneling project. Core sampling within the original tunnel liner and surrounding rock mass will still be required, as the original tunnel will still function as an overflow diversion structure. This task should be performed before the design work for the new tunnel is completed. With this data the rock mass classification, theoretical loadings, bearing capacities, potential groundwater movement and joint conditions could be estimated for the design and analysis to determine the tunnel standup time and liner strength need for construction of the new tunnel.

As stated in Section 4.2, a PSHA will be performed for this project. In doing so, analysis of the current tunnels long term viability can be determined and the development for the design of the embankment structure and tunnel liner design can be refined.

Investigations may also be required related to the potential for landslides and surge releases. These would attempt to verify that the current estimate of 19,000 cfs of surge flow is sufficiently high so that the estimated risk reduction will be achieved. Information from an upcoming University of Alaska, Fairbanks paper mapping landslides in Lowell Creek basin will be utilized.

Lowell Creek water would be diverted around the proposed tunnel location, through ether a corrugated or smooth metal piping, or HDPE (high-density polyethylene) piping during the new tunnel's construction. An access road would need to be developed over this system. The creek water would continue to flow through the established bypass/overflow used during the repairs to the existing tunnel. Once the repairs have been finished on the existing tunnel, it could be used as the outflow until the new tunnel is complete. During the tunnel's construction, dump trucks would be used to remove excess material from the valley or outlet; the length of travel will vary depending on the direction of construction/tunneling. Material from tunnel excavation will be disposed of based on its physical properties. There may be the potential for the excavated material to be used as beach nourishment for the City of Seward. If a beneficial use for the material cannot be determined the material may need to be deposited within Resurrection Bay depending on an environmental review/analysis.

Additional information specific to the outlet portal location will be developed and evaluated by a subject matter expert prior to any geotechnical investigation on site to ensure the right area is investigated. The outlet portal is a critical feature for the success of this projects and will be further evaluated during the PED process. During PED a more precise location for the downstream portal, how close it will be to the existing tunnel outlet and expected conditions and/or need for rockfall/ landslide protection will be evaluated.

4.5 Alternative 5: Construct Debris Retention Basin

This alternative would involve the construction of a sediment retention basin upstream of the current Lowell Creek Diversion Dam and Tunnel (S15). Sediment retention would be through a series of check dams. The basin would retain the sediment produced during a flooding event behind these dams. Then maintenance would need to be performed following each event, which will require the development of a roadway further up into the valley. The material removed from the retention structures will need to be disposed of.

There is a potential for the sediment/material to be used as beach nourishment. If this does not become a viable option, the material could be deposited into Resurrection Bay depending on an environmental analysis of the material and proposed disposal sight. This alternative would require a similar geotechnical investigation of the underlying material as for Alternative 4, Section 4.4, for the structure. The geotechnical investigation would include soil boring analysis and material testing to bedrock at the proposed location.

Groundwater analysis and collection of stream flow and infiltration rates would also be performed and evaluated at this location and continuing downstream to the existing structure. Core sampling of the liner and surrounding rock throughout the inside of the existing tunnel is required for long-term maintenance of the tunnel. Flow bypasses would need to be developed for each check dam that was constructed. Material for the dams could potentially come from the sediment/materials located within Lowell Creek Canyon, depending on its composition and the technique of roller compacting concrete will be used to construct the check dams. The material will need to be processed dependent on the concrete mix design, for the construction of the dams.

4.6 Alternative 6: Floodplain Structure Relocation and Removal

This alternative would require additional geotechnical investigation through the city of Seward, if historical borings could not be located, for the development of a floodway through the city. Relocation of the structures within the canyon may require additional geotechnical borings for a building if a new one need to be constructed. The use of existing buildings for the relocation would negate the need for further geotechnical investigation.

4.7 Preferred Alternative

From the analyzed data, the development of a larger primary tunnel and dam structure provides the most significant benefit to the city of Seward. The construction of an 18-ft tunnel would be able to provide capacity for a 2,800cfs event and would allow for the transportation of debris within the water column. In addition, a second flood diversion system would allow for maintenance activities within one of the tunnels while the other passes the channels flow and vice versa. This alternative would require a more extensive geotechnical investigation compared to the other designs, analysis of the surrounding rock, underlying material and geologic structuring, but would incorporate many of the task that have to be performed for the other alternatives. In the event where flows were able to surge over the diversion structure the other structure would be able to divert the flow through it, preventing inundation to the city of Seward reducing the overall risk to the city.

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