

### 3.3 GEOHAZARDS AND SEISMIC CONDITIONS

This section presents information available regarding geological hazards (geohazards) for the Mine Site, Transportation Corridor, and Pipeline. Geohazards include geophysical processes (earthquakes and volcanoes), surficial or geomorphological processes (landslides, and avalanches), and coastal hazards (flooding, beach and bank erosion, and tsunamis).

The EIS Analysis Area, across Southwest and Southcentral Alaska, lies within a region of active tectonic (geophysical) processes and contains various types of terrain, some with high potential geohazards. The area encompasses an array of landforms associated with dynamic geologic processes, some of which may occur throughout the entire EIS Analysis Area, whereas others may be limited to one geographic region. The potential for multiple types of geohazards across the EIS Analysis Area is dependent on the topography, regional location, natural materials present, and proximity to known geohazard sources (e.g., volcano, earthquake fault, avalanche chute). Earthquakes and associated geohazards are the most noteworthy earth processes with effects on both a regional and local scale. In contrast, surficial processes like landslides tend to affect localized areas.

---

#### SYNOPSIS

This section presents information available regarding geohazards (geologic hazards) in each of the proposed Mine Site, Transportation Corridor, and Pipeline regions. Each alternative is examined by major project component: Mine Site; Transportation Corridor; and Pipeline.

#### EXISTING CONDITION SUMMARY

Geohazards include geophysical processes (earthquakes and volcanoes), surficial or geomorphological processes (landslides, and avalanches), and coastal hazards (flooding, beach and bank erosion, and tsunami). Regional scale descriptions of the geohazards are presented in this section, followed by local descriptions enhanced with information gathered from geotechnical engineering studies where available. The proposed Project Area encompasses an array of landforms associated with dynamic geologic processes. Some of these processes may occur throughout the entire EIS Analysis Area, whereas others may be limited to one geographic region. The Project Area, across Southwest and Southcentral Alaska, lies within a region of active tectonic (geophysical) processes and contains various types of terrain, some with high potential of gravity-generated geomorphological processes. The potential for multiple types of geohazards across the proposed Project Area is highly dependent on the topography, regional location, natural materials present, and proximity to known hazard sources.

#### EXPECTED EFFECTS SUMMARY

##### Alternative 1 - No Action

Impacts to the project from geohazards, seismic events, and other geotechnical conditions would not occur as a result of this alternative. In addition, there would be no impacts to the environment as a result of natural hazards such as earthquakes and landslides causing damage to project facilities.

### **Alternative 2 - Donlin Gold's Proposed Action**

Impacts in the event of a major earthquake at the Mine Site, Transportation Corridor, and Pipeline components would mostly range from ground shaking effects that may or may not be noticeable to effects that could result in noticeable but repairable impacts on mine infrastructure. There would be a low probability of acute or obvious effects at certain structures and project phases (e.g., pit walls in post-Closure, effects from ground shaking, liquefaction, and slope instability for all phases of mining, and into post-Closure). Slope stability effects would mostly range from minor sloughing in low to moderate relief areas along the pipeline corridor that may not be measurable or noticeable, to noticeable effects along the pipeline route. Under these cases, the design is adequate for the expected range of geohazard conditions. There would be a low probability of acute or obvious effects in two cases: landslide movement near lower Contact Water Dam (CWD) during Construction, and northwest pit crest settlement and potential overtopping by Crooked Creek. The effects of other potential geohazards would range from tsunami effects that may not be noticeable or measurable, to noticeable but repairable effects from dam seepage and volcanoes. There would be a low probability of acute or obvious more significant effects on river water quality in the event of Horizontal Directional Drilling (HDD) frac-out. Mitigation measures are provided in Chapter 5, Impact Avoidance, Minimization, and Mitigation.

The duration of effects from earthquakes (including failure or liquefaction effects), slope instability, and other geohazards could range from infrequent but not longer than the span of the Construction Phase, to effects lasting through the life of the project. The extent or scope of effects would be mostly within the immediate vicinity of facility footprints. Geohazards are a usual or ordinary occurrence in Alaska, but are governed by regulation for certain structures (e.g., dams, pipelines).

**OTHER ALTERNATIVES** – This section discusses differences of note between Alternative 2 and the following alternatives, but does not include a comprehensive discussion of each alternative's impacts if they are the same as or similar to Alternative 2 impacts.

#### **Alternative 3A - LNG Powered Trucks**

There could be slightly more seismic effects due to an LNG plant that is expected to be designed to withstand ground shaking, and slightly fewer seismic and slope concerns due to the reduction in port fuel tanks.

#### **Alternative 3B - Diesel Pipeline**

There could be slightly more effects from seismic, bluff stability, and seafloor geohazard concerns at the Tyonek dock and tank farm; liquefaction along 19 to 20 miles of additional pipeline; depending on selected option, and slope stability issues at two additional steep open-cut river crossings and five additional material sites. There could also be a slight increase in effects on river water quality in the event of frac-out, due to one additional HDD river crossing (Beluga River or Susitna River, depending on selected option).

#### **Alternative 4 - Birch Tree Crossing (BTC) Port**

There could be slightly greater effects from two additional bridges that are expected to be designed to withstand ground shaking; and from slope stability issues along the 46 mile longer road, and at approximately 2.5 times as many material sites, as Alternative 2.

### **Alternative 5A - Dry Stack Tailings**

There could be a slight increase in effects from ground shaking, liquefaction, and slope instability effects in the part of the dry stack that rises above the upper dam as compared to the Tailings Storage Facility (TSF) under Alternative 2 during Operations. There would be reduced seismic and slope stability concerns during closure, as the dry stack would become a stable landform, while the TSF would not.

### **Alternative 6A - Dalzell Gorge Route**

There could be more effects along this route, which has seven more miles with high-hazard slopes than Alternative 2, in which site-specific design is expected to withstand active debris flows.

---

## **3.3.1 REGULATORY FRAMEWORK**

Regulations and applicable guidance governing dam safety, administered under authority of the Alaska Department of Natural Resources (ADNR); pipeline safety administered under authority of ADNR, the State Pipeline Coordinator's Office (SPCO), and the U.S. Department of Transportation (USDOT); and, horizontal directional drilling administered under authority of ADNR and the Alaska Department of Environmental Conservation (ADEC) are applicable to various Mine Site and Pipeline project components. The applicability of regulations and guidance in these areas is provided in the following sections.

### **3.3.1.1 DAM SAFETY**

Dam safety in the state of Alaska is regulated by the Alaska Department of Natural Resources (ADNR) primarily under Alaska Statute (AS) 46.17 "Supervision of Safety of Dams and Reservoirs" and 11 AAC 93 "Dam Safety." Enforcement powers granted to ADNR under Dam Safety regulations include requirements for ADNR approval to construct, enlarge, repair, alter, remove, maintain, operate, or abandon a dam or reservoir. ADNR can inspect dams and enter private lands for this purpose without notice if there is reason to believe that a dam or reservoir may be unsafe or presents an imminent threat to life or property. ADNR may order the owner to take action to protect life and property if it determines the dam or reservoir is unsafe, and may take supervisory control of the dam from the owner in emergency situations.

ADNR also has financial assurance requirements associated with dam safety (11 AAC 93.171 and 172), and may enter into cooperative agreements with other state and federal agencies for the purpose of reclamation and management of tailings dams in accordance with AS 27.19.060. Financial assurance must be established to pay for costs of reclamation and post-Closure monitoring and maintenance, or for breaching a dam and restoring the stream channel and land to natural conditions (see Appendix A, Financial Assurance, and Appendix AA, Additional Regulatory Framework Information).

ADNR (2005) has published Guidelines for Cooperation with the Alaska Dam Safety Program, which is administered by ADNR in accordance with dam safety regulations, and applies to both water and wet tailings dams. The guidelines define classifications of dams based on potential danger to lives and property. These classifications are the main parameter for determining the level of

attention that a dam requires throughout the life of the project. The hazard potential classification represents the basis for the scope of the design and construction effort, and dictates the requirements for certain inspections and emergency planning. ADNR uses three classifications for dams based on the potential impacts of failure or improper operation of a dam:

- Class I (high). Probable loss of one or more lives if failure were to occur. In such an instance property damage is considered irrelevant, but would be similar to Class II or III.
- Class II (significant). No loss of life expected, although a significant danger to public health may exist. There is a probable loss of or significant damage to homes, occupied structures, commercial or high-value property, major highways, primary roads, railroads, or public utilities, or other significant property losses or damage not limited to the owner of the barrier. Probable loss of or significant damage to waters identified under 11 AAC 195.010(a) as important for spawning, rearing, or migration of anadromous fish.
- Class III (low). Insignificant danger to public health. Limited impact to rural or undeveloped land, rural or secondary roads, and structures. Loss or damage of property limited to the owner of the barrier.

The planned dams at the Donlin Gold Mine Site consist of the following:

- The Tailings Storage Facility Dam (TSF) – Class I;
- Fresh Water Dam (Snow Gulch FWD) – Class I;
- The Fresh Water Diversion Dams (American FWDD and North and South FWDDs) – Class II or III; and
- Upper and Lower Contact Water Dams (CWDs) – Class II or III.

The ADNR (2005) guidelines contain design requirements for hydrology (inflow flood, precipitation, snowpack); hydraulics (flood routing, spillway, freeboard); stability under a variety of loading conditions; design earthquake levels; seepage analysis; and cold regions factors such as permafrost foundation issues, ice loading, and other cold temperature effects on construction materials and operations. Dry stack tailings are regulated by the Alaska Department of Environmental Conservation (ADEC) under their solid waste permitting program (18 AAC 60). However, the primary intent of the dam below the dry stack under Alternative 5A would be to contain operating pond water from flowing into the dry stack, and as such, would likely be regulated under the ADNR dam safety program as Hazard Class II.

Two levels of design earthquakes are required to be addressed under ADNR (2005) guidance: 1) an operating basis earthquake (OBE) representing ground motion or fault movement with a reasonable probability of occurrence over the project life, during which dams must remain functional and easily repairable; and 2) a Maximum Design Earthquake (MDE) representing the most severe earthquake that could potentially occur relative to an established acceptable risk level, during which dams must resist collapse, failure, or uncontrolled release. Risk levels for the OBE and MDE are defined in terms of earthquake return period, that is, the frequency with which a certain size earthquake is expected to occur.

For Class I dams, the return period for the OBE is specified as 150 to > 250 years, and for the MDE, the return period is specified as 2,500 years to the return period of the maximum credible

earthquake (MCE). For Class II dams, the return period for the OBE is specified as 70 to 200 years, and for the MDE, the return period is specified as 1,000 to 2,500 years. Seismic hazard analyses conducted for the Mine Site relative to these levels are described in Section 3.3.2.1.2.

The ADNR (2005) dam safety guidelines also contain requirements governing different phases of the project life, such as construction plans and construction quality assurance/quality control (QA/QC); operations, maintenance, and repairs; monitoring and inspections; emergency action planning; and closure. Emergency action plan requirements under 11 AAC 93.164(b) identify specific requirements for dam failure analysis and detailed inundation maps which estimate the extent of downstream flooding in the event of a complete dam breach. With respect to dam failure analysis, guidance is provided by ADNR (2005) for appropriate levels of engineering evaluation; quantitative dam break models; weather, breach size, and failure mode parameters; flood wave attenuation; considerations for fish habitat; and consideration of potential domino effects of dam failure on other dams located downstream. Analysis of environmental impacts from a dam breach was conducted for the purposes of the EIS. The parameters for this analysis (failure mode selection, size of the release), and inundation maps from this modeled spill scenario are provided in Section 3.24, Spill Risk.

### 3.3.1.2 PIPELINE SAFETY

The planned natural gas pipeline right-of-way (ROW) is regulated by ADNR, State Pipeline Coordinator's Office (SPCO) in accordance with AS 38.35 (Public Land, Right-of-Way Leasing Act). The U.S. Department of Transportation (USDOT) is charged with developing and enforcing minimum safety regulations for the conveyance of gases by pipeline and has issued those regulations in 49 CFR Part 192 (Transportation of Natural and Other Gas by Pipeline). The Donlin Gold pipeline will require special conditions in order to safely utilize strain-based design to account for potential ground movement from permafrost (see Section 3.2, Soils) and other geohazards. Strain-based design is used to assess parameters to allow the pipe to deform in a longitudinal direction, yet maintain serviceability and remain safe. Typical parameters assessed include pipe diameter and wall thickness, material strength, and load stress-strain under longitudinal plastic deformation (strain greater than 0.5 percent). The objective of the strain-based design is to determine the pipe elastic range to account for potential ground movement (landslide, thaw settlement, frost heave), and seismic activity (liquefaction, surface rupture). The USDOT will require conditions for the design, pipeline materials, construction, and operations and maintenance practices to ensure that in areas where strains are anticipated to approach or be above 0.5 percent, the appropriate measures are in place to mitigate these strains. These conditions for strain-based design – which will be included in the normal processes of pipeline design, construction, operations and maintenance, and any effects – are being evaluated in this EIS.

### 3.3.1.3 HORIZONTAL DIRECTIONAL DRILLING

A “frac-out,” or inadvertent release of Horizontal Directional Drilling (HDD) drilling fluids to a stream, could be caused by unforeseen geological conditions or by inappropriate drilling techniques. A frac-out would be considered a point source discharge under Section 402 of the Clean Water Act (CWA) and requires Alaska Pollutant Discharge Elimination System (APDES) permit coverage as well as a mixing zone. A State Pipeline General Permit (AKG320000) is being developed to provide this coverage as a contingency discharge. Discharge



authorizations will include evaluation of fish habitat near HDD crossings (i.e., spawning or rearing habitat for resident or anadromous fish), ambient water quality and drilling fluids used, and hydraulic data for mixing zone authorizations (ADNR 2015b). APDES coverage for frac-out, assuming coverage was obtained under the statewide oil and gas pipeline general permit (AKG320000), would require development of a drilling fluid plan and best management practices (BMPs) specific to drilling activities. Information on the likelihood of frac-out related to geotechnical conditions and the anticipated effects of drilling mud on rivers are discussed in Sections 3.3.2.3.3 and 3.3.3.2.3. Hydraulic information, ambient water quality, and fish habitat are discussed in Sections 3.5, Surface Water Hydrology; 3.7, Water Quality; and 3.13, Fish and Aquatic Resources, respectively.

### 3.3.2 AFFECTED ENVIRONMENT

#### 3.3.2.1 MINE SITE

##### 3.3.2.1.1 EARTHQUAKES

Regions of Alaska are among the most seismically active areas of the world, where earthquakes can cause major structural damage to buildings and infrastructure and injury to citizens. Such damage can occur in multiple ways. There can be damage from fault displacement, that is to say horizontal or vertical movement along a fault line. Damage can also occur due to ground shaking, which the vertical and horizontal motion imparted to structures during an earthquake. Or it can occur due to liquefaction, where saturated soil becomes unstable and no longer able to support overlying structures.

In order to evaluate seismic geohazards, faults are identified, physical properties of bedrock and surficial geology units are evaluated, and a seismic hazard analysis is conducted. For seismic geohazards, the study area can range up to several tens or hundreds of miles.

Faults fall into two broad categories based on their history of movement. The USGS Earthquake Hazards Program defines faults that show evidence of movement in the last 10,000 years, either through offsets in Holocene deposits or historical seismicity, as active faults. Faults without documented evidence of movement during the Holocene period (the last 10,000 years) are generally classified as inactive faults; however, in some cases there may be insufficient data to make a determination on whether movement has occurred during the Holocene, and faults may be described as “questionable” with regard to being active or inactive. Inactive faults may (in rare cases) still be or may become active, but a vast majority of these faults may show no activity ever again, or none within the foreseeable future.

##### Active Faults

As presented in Section 3.1, Geology, the regional geology of southwest Alaska consists of a complex mixture of lithotectonic terranes or groups of rocks, separated by tectonic sutures, that accreted onto the North American ancestral continent during the Mesozoic era (Decker et al. 1994; Bundtzen and Miller 1997; Wilson et al. 1998; Plafker and Berg 1994; Plafker et al. 1994; Silberling et al. 1994; Miller et al. 2008; Goldfarb et al. 2010). These sutures are marked by regional-scale strike-slip faults, the Denali-Farewell and Iditarod-Nixon Fork faults, that trend northeast through the Kuskokwim basin and mountains (Plafker et al. 1994; Decker et al. 1994;

Miller et al. 2008). Active tectonic processes continue to displace landforms along the Denali-Farewell fault system (Freymueller et al. 2008; Koehler et al. 2011, 2012a; Koehler 2013).

The continued displacement along the Denali-Farewell Fault is likely driven by the approximately two inches per-year oblique convergence between the oceanic Pacific/Yakutat plate and continental North America plate (Koehler et al. 2011). Two types of faulting environments occur at this tectonic plate boundary: 1) subduction of oceanic crust along the offshore Alaska-Aleutian megathrust, and 2) strike-slip and transpressional faulting on land along the Denali and other interior Alaska faults (Freymueller et al. 2008; Haeussler and Saltus 2011; Koehler and Reger 2009). A segment of the Denali-Farewell Fault located about 150 miles to the east of the Mine Site near Big River, is the closest known active surface fault to the Mine Site (Koehler 2013) (Figure 3.3-1).

The age of the Iditarod-Nixon Fork Fault at its closest point the Mine Site, about 15 miles away, is considered “questionable” (Figure 3.3-1). The most recent surface rupture on this fault, and on the Holitna segment of the Denali-Farewell fault system located 50 miles southeast of the Mine Site, is considered to be mid-Quaternary (less than 750,000 years), although some seismicity in the subsurface may be associated with these faults (Koehler 2013; Plafker et al. 1994; Wilson et al. 2013) (Figure 3.3-1). Kuskokwim Group rocks have experienced approximately 54 miles of right-lateral displacement along the Iditarod-Nixon Fork Fault, and 80 miles of right-lateral displacement along the Holitna segment of the Denali Fault, since the Early Cretaceous (Goldfarb et al. 2004).

The Donlin Creek Fault, located one mile northeast of the Mine Site, and associated small conjugate faults at the site (Section 3.1, Geology) are not recognized as active or questionable in available literature (Koehler 2013). A review of newly released high-definition topographic elevation model data (NGA 2016) also does not indicate obvious surficial evidence of active faulting in the vicinity of the Mine Site.

### **Ground Shaking and Seismic Hazard Analysis**

Primary seismic geohazards include ground shaking, landslides, liquefaction, and surface rupture. Ground shaking or ground motion is the displacement of the ground due to the passage of elastic waves arising from earthquakes or man-made explosions. Surface rupture is deformation due to loss of cohesion or loss of resistance to differential stress, and a subsequent release of stored elastic energy. Ground motion is assessed by predicting ground acceleration, velocity, and duration of a ground shaking event based on historic data. At any given location, seismic hazard is dependent on event magnitude and distance from seismic sources, event frequency, and the properties of the rocks and unconsolidated materials through which seismic waves travel. Effects of past earthquakes in an area, subsurface geologic units, distance from specific seismically active features, and long-term frequency of seismic events all factor into seismic hazard analysis.

The evaluation of potential seismic hazards typically involves an estimation of the probability of a location experiencing ground shaking over a specific period of time in the future. The ground shaking is expressed as peak acceleration over a period of return. Peak ground acceleration is typically expressed as a fraction of the gravitational constant (g). For example, a peak ground acceleration value of 0.1g in bedrock is considered the approximate threshold at which damage occurs to buildings not designed or constructed to resist earthquakes. The return period is the frequency that such earthquakes might occur. For example, a peak ground

acceleration of 0.33 g with a return period of 250 years means the peak ground acceleration that might be expected to occur over a period of 250 years is 0.33 times the force of gravity.

Statewide seismic hazard maps have been developed using multiple earthquake source zones and a range of time periods (Figure 3.3-2) (Wesson et al. 2007; Alaska Seismic Hazards Safety Commission [ASHSC] 2012). These seismic hazard maps show the distribution of earthquake shaking levels that have a certain probability of occurring statewide. The information is useful for designing buildings, bridges, highways, and utilities to withstand earthquakes. The seismic hazard maps depict specific areas associated with specific probabilities of earthquake ground motion exceeding a stated severity level in a given period of time.

The Alaska Earthquake Information Center (AEIC) maintains a database of earthquake events dating back to 1898 for the entire state of Alaska. The database characterizes the earthquake events by date, location, and earthquake strength or magnitude. The Mine Site is located in a region of Alaska that is relatively quiescent seismically compared to Cook Inlet and the Aleutians (Figure 3.3-1). A search for earthquake events with a minimum magnitude<sup>1</sup> of 4 in the area of the Mine Site revealed 23 events since 1903. Twenty-one of the events had magnitudes ranging from 4.0 to 4.9 and occurred as recently as June 8, 2005 (magnitude 4.6). Two remaining events, one in June 1903 and a second in May 1971, had magnitudes of 6.9 and 5.8, respectively. The epicenter of the 1903, 6.9-magnitude event was located south near the community of Napaimute. The epicenter of the 1971, 5.8-magnitude event occurred at a depth of 60 miles below surface and was located southeast of Sleetmute, and likely represented movement on the Denali-Farewell fault system. (AEIC 2014; Koehler 2013). In addition to those larger recorded earthquakes, a smaller magnitude 3.9 earthquake, centered near Crow Village, occurred in 2014.

Based on statewide seismic hazard maps (ASHSC 2012), peak ground acceleration in the mine region is estimated to range from 0.07g to 0.15g for a 475-year average return period (10 percent probability of being equaled or exceeded in a 50-year period), while at the Mine Site, peak acceleration is estimated at 0.09g to 0.10g over the same return period (Figure 3.3-2).

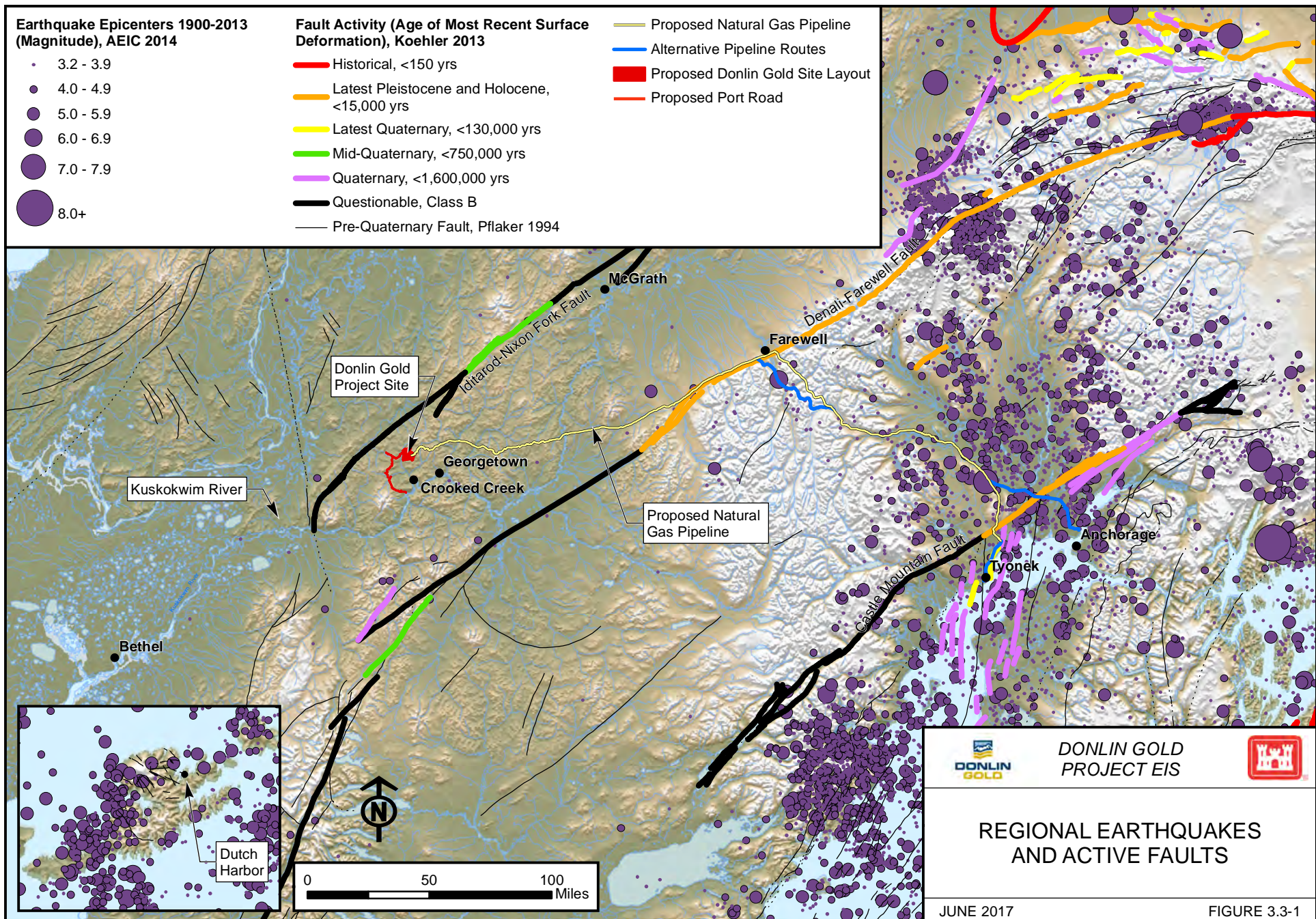
Site-specific seismic hazard evaluations have been conducted for the Mine Site to estimate a variety of ground motion parameters including peak ground acceleration. Geotechnical engineering studies by BGC Engineering (2011a, b, c) have included project-specific probabilistic and deterministic<sup>2</sup> seismic hazard assessments for the site, which are applicable to all facilities, including waste management features and buildings. Probabilistic hazard assessment of the Mine Site area was conducted to develop horizontal peak ground acceleration values. Estimated earthquake magnitudes were developed under operational- and contingency-level earthquake scenarios for the western portions of the Iditarod-Nixon Fork and Denali-Farewell faults. The estimated peak horizontal ground accelerations for return periods of 100, 475, 1,000, 2,474, 5,000, and 10,000 years are 0.05g, 0.11g, 0.16g, 0.25g, 0.34g, and 0.44g, respectively (BGC 2011a, b, c). The site-specific estimate for the 475-year return period is similar to that predicted by the ASHSC (2012) statewide maps.

---

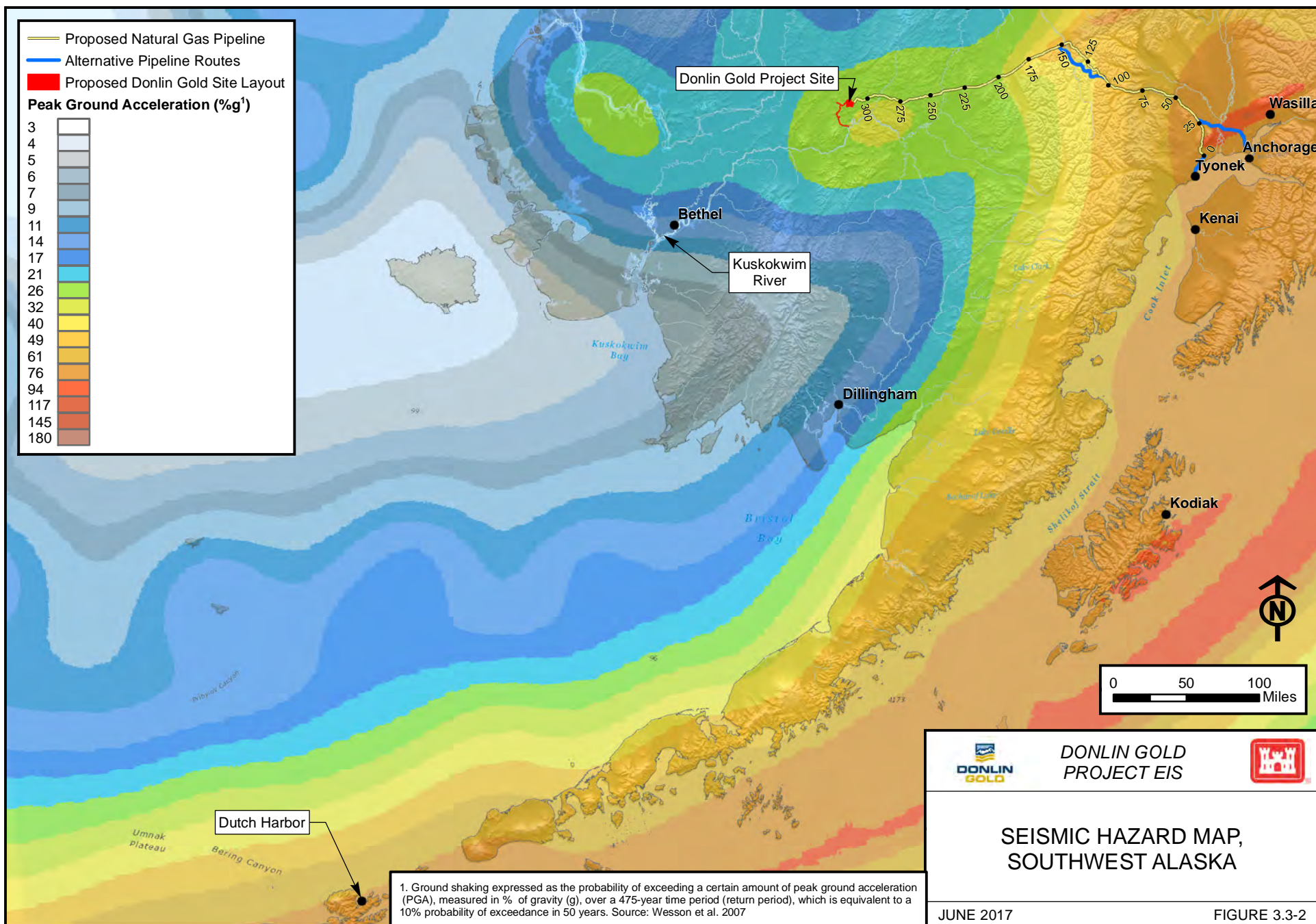
<sup>1</sup> Earthquake magnitude is a measure of the relative size of an earthquake, and is typically based on the Moment Magnitude Scale. The Moment Magnitude Scale is a logarithmic scale, meaning that each incremental whole number increase (e.g., magnitude 4 vs. magnitude 5) reflects a 10-times increase in the overall size of the earthquake. The Moment Magnitude Scale is preferred over other commonly referenced earthquake scales (e.g., Richter Scale) because it provides a more accurate characterization of a wider range of earthquakes, particularly in the extremely large range of greater than magnitude 8. (USGS 2016)

<sup>2</sup> The deterministic approach develops a scenario for each individual fault using a specified magnitude and distance. The probabilistic approach mathematically combines models for different faults and earthquake magnitudes within a region.









The deterministic hazard assessment evaluated earthquake magnitude and likely ground motion from five causative epicenters or maximum credible earthquake (MCE) scenarios. The first two scenarios looked at the subduction of the Pacific plate at the Alaska-Aleutian megathrust at 180- and 240- mile distances from the Mine Site, with depths below surface of 95 and 30 miles, respectively. The third and fourth scenarios looked at two segments of the Denali fault system at 110 miles (west end of Farewell) and 50 miles (Boss Creek-Holitna segment) from the Mine Site, with depths below surface of 30 miles. The fifth scenario looked at the closest point (15 miles) of the Iditarod-Nixon Fork Fault. The assessment concluded that the Iditarod-Nixon Fork scenario would likely be the most critical due to proximity and shallow depth of the potential epicenter; the maximum design earthquake for this scenario would have a peak horizontal ground acceleration of 0.36g from a 7.8 magnitude event. The closest fault scenario having the potential for a 9.2 MCE, the Alaska-Aleutian Megathrust Fault at 240 miles away and a depth of 30 miles, was estimated to cause peak ground acceleration of only 0.07g due to distance from the Mine Site (BGC 2011a, b, c).

As described in Section 3.3.1.1, the Alaska Dam Safety Program provides guidance on the selection of minimum return period earthquake ground motions for use in dam design based on the dam hazard classification (ADNR 2005). The design earthquake selected for the tailings dam was the MCE with a peak ground acceleration of 0.36g from a magnitude 7.8 event on the Iditarod-Nixon Fork Fault (BGC 2011a), which represents a 1 in 5,000-year return period. BGC (2011a) also recommended consideration of the 0.44g 10,000-year return period event in the design of the tailings dam. For design of the fresh water dams, a 1 in 2,475-year return period earthquake event, with a peak ground acceleration of 0.25g, was applied (BGC 2011c). These design earthquake levels meet or exceed the recommendations of the ADNR (2005), and were used to evaluate the effects of major ground shaking events on dam stability as described in Section 3.3.3.2.1. Under AS 46.17, ADNR is the controlling regulatory authority for dams in Alaska.

### Liquefaction Potential

Liquefaction is an earthquake phenomenon that reduces the strength and stiffness of a soil by ground shaking. Where the groundwater table is near surface, or the ground is otherwise saturated, the pore space between the soil particles containing water can increase, changing the physical character of the landform and weakening the natural material; in essence, the ground temporarily behaves like a liquid. Liquefaction generally affects unconsolidated, fine-grained sand and silt deposits in lowland areas. The potential for liquefaction from ground shaking at the Mine Site is less for man-made features built on ridges, where bedrock is near the surface, than in lowland areas underlain by thicker unconsolidated material (Reger et al. 2003a).

#### 3.3.2.1.2 SLOPE STABILITY

##### Slope Processes

The stability of landform slopes depends on the physical characteristics of the natural materials. Landslides are the most recognized form of slope failure. In general, the term landslide is used to describe the downslope movement of soil, rock, and organic materials (or a combination thereof) under the effect of gravity. The process of downslope movement is referred to as mass wasting. The term landslide also describes the landform that results from such movement

(Highland and Bobrowsky 2008). Downslope movement usually takes place on curved surfaces (rotational slides) or planar surfaces (translational slides). Landslides are categorized based on type of movement and the type of material involved. The type of movement describes how the material is displaced (fall, topple, slide, spread, or flow). If the material displaced consists of sand-sized or smaller particles, it is classified as an earth slide. If the displaced material consists of large rock fragments it is called a debris slide (Highland 2004). If mass wasting occurs as a flow, it can be further categorized as a debris flow, debris avalanche, earthflow, mudflow, or creep. The type of flow and rate of movement vary with size and type of material, water content, and shape. Landslides are generally caused by natural processes, such as earthquakes and erosion, and man-made processes, such as deforestation and slope excavation.

The top of a slope immediately above a slide, or failure, is referred to as the crown, and the exposed failure surface below the crown is called a scarp. The end of a landslide is referred to as the toe, and the top of the landslide is called the head. The main body of the landslide may have radial or transverse cracks and transverse ridges.

### Mine Site Slopes

A dormant landslide (slope that has failed in the past but is not currently moving) was identified near the proposed south abutment of the lower contact water dam (CWD) on the southwestern slope of American Creek Valley (BGC 2011b, c) (referred to in herein as the American Creek Landslide). The landslide was reported on a slope less than 30° with a maximum height of 310 feet. The estimated area of the landslide is approximately 60,000 square feet. The features of the landslide suggest that the failure surface is along stratigraphic bedding planes of the bedrock (translational slide), and that the slide material is less than three feet thick. Key features include lateral minor scarps, a toe that is steeper than adjacent topography, and a compound headscarp. The landslide is not considered recent as many of its features are heavily eroded (BGC 2011b, c).

The American Creek landslide failure surface lies within greywacke sandstone interbedded with siltstone, with bedding dip angles that flatten downslope; suggesting that the slide area is located within a synclinal<sup>3</sup> structure. Zones of deformation were reported beneath the interpreted failure plane, but were not considered to be controlling the failure mechanism. The study indicated that the bedding planes were deformed as well, and probably played a key role in the slope failure (BGC 2011b).

A smaller landslide was also mapped in the American Creek drainage in the northeast corner of the proposed waste rock facility footprint. This feature is a narrow 2,000-foot long debris flow that extends along an upper tributary of American Creek. The debris flow ranges from three to 10 feet thick and has been modified by more recent gully erosion (BGC 2011b).

#### 3.3.2.1.3 OTHER GEOHAZARDS

The location of the proposed mine is a long distance away from active volcanoes in Alaska. While there is no consensus among volcanologists on how to define an active volcano, scientists usually consider a volcano active if it is currently erupting or showing signs of unrest, such as unusual earthquake activity or substantial new gas emissions. Many scientists also consider a

---

<sup>3</sup> A syncline is a concave-upward fold of geologic layers.

volcano active if it has erupted in historic time (the last few thousand years). In Alaska, both active and inactive volcanoes are present in a few well-defined areas, such as western Cook Inlet and the northern Alaska Peninsula. There is no evidence of historic activity in the Project Area. The volcanoes of Cook Inlet and the northern Alaska Peninsula, located roughly 300 to 400 miles to the southeast, could potentially pose an ashfall threat to the Mine Site in the event of an eruption, depending on wind conditions. Volcanic geohazards and features are described in more detail in Section 3.3.3.2.3.

The Mine Site would likely not experience any type of avalanche or glacial hazard since the topography of the Kuskokwim Mountains consists of gentle to moderate sloping mountaintops with no recognized glaciers nearby. Local snow slides are possible in areas of steeper slopes, such as the southwest slope of American Creek and the north slope of Anaconda Creek; however, these slides would be unlikely to adversely impact facilities or operations.

### 3.3.2.2 TRANSPORTATION CORRIDOR

#### 3.3.2.2.1 EARTHQUAKES

##### Active Faults

###### *Access Roads, Ports, and Kuskokwim Corridor*

The closest known active surface faults to the proposed access roads and river corridor are the same as described for the Mine Site (Section 3.3.2.1). The north-south trending Aniak-Thompson Creek Fault exposed along the Kuskokwim River a few miles east of Aniak is considered suspicious, with minor evidence suggesting the possibility of activity (Koehler et al. 2012a, Koehler 2013). There are no recognized active surface faults along the Kuskokwim River corridor from Aniak to Bethel based on available literature (Wilson et al. 2013; Koehler 2013). A review of newly released high-definition topographic elevation model data (NGA 2016) also does not indicate obvious surficial evidence of active faulting in the vicinity of the mine Transportation Corridor.

###### *Dutch Harbor*

On Unalaska Island there are numerous northwest trending, high-angle normal faults that truncate the Unalaska Formation (Drewes et al. 1961; Koehler et al. 2012a). These faults are expressed as linear topographic landform features. The Quaternary faults on Unalaska Island are considered suspicious features, with minor evidence suggesting that the structures may be active (Koehler et al. 2012a).

##### Ground Shaking and Seismic Hazard Analyses

###### *Access Roads, Ports, and Kuskokwim Corridor*

A search for earthquakes with a minimum magnitude of 4.0 near proposed transportation facilities, access roads, ports, and along the Kuskokwim River corridor, revealed 13 occurrences since 1970. Ten of the events had magnitudes ranging from 4.1 to 4.4, and the most recent occurred in February 1994, with a magnitude of 4.1.



The largest three events occurred in June 1970, February 1973, and January 1983. All had magnitudes of 4.6. The epicenter of the 1970 event occurred at a depth of 150 miles below surface and was located west of the community of Lower Kalskag. The epicenter of the 1973 event occurred at a depth of 60 miles and was located northwest of Bethel. The epicenter of the 1983 event occurred at a depth of 20 miles and was located east of the community of Tuluksak (AEIC 2014). All three events likely represent movement on the Boss Creek, Holitna, and Togiak-Tikchik fault splays of the Denali fault system (Koehler 2013).

The statewide seismic hazard map covering the area of the proposed transportation facilities shows decrease in estimated peak ground acceleration from the Mine Site region to Bethel (Figure 3.3-2). In the Kuskokwim River corridor from Crooked Creek to Tuluksak, the peak ground acceleration is estimated at 0.10g for a 475-year average return period (ASHSC 2012). From Tuluksak to Bethel, the peak ground acceleration is estimated to range from 0.07 to 0.05g for the same return period (ASHSC 2012).

Project-specific, probabilistic and deterministic seismic hazard assessments conducted by BGC Engineering (2011a, b, c) for the Mine Site (Section 3.3.2.1.2) can be applied to the nearby proposed access roads and port sites (both Angyaruaq [Jungjuk] and BTC alternatives). Based on the results of the deterministic hazard assessment, which assumed a maximum design earthquake of 7.8 from the nearest known fault (Iditarod-Nixon Fork), the road and port would experience a peak horizontal ground acceleration of 0.36g. This event would probably be more critical than a larger magnitude event on the Alaska-Aleutian subduction zone, due to distance of potential epicenters from proposed transportation facilities (BGC 2011a, b, c).

### *Dutch Harbor*

A search for earthquake events with a minimum magnitude of 5 in the area of Dutch Harbor found 113 events since 1902. Only two of the 113 events reported magnitudes greater than 7.0, one in January 1902, and one in April 1957. The remaining 111 events had magnitudes ranging from 5.0 to 6.9, and occurred as recently as August 2012, with a magnitude of 5.1. The epicenter of the 1902 event was located south of the community of Chernofski on Unalaska Island. The epicenter of the 1957 event occurred at a depth of 2.5 miles below surface, approximately 100 miles south of Dutch Harbor (AEIC 2014).

No project-specific probabilistic and deterministic seismic hazard assessments were completed for Dutch Harbor. The estimated peak ground acceleration in the Dutch Harbor region is higher than at the Mine Site. The probabilistic peak ground acceleration is estimated to be 0.35g for a 475-year average return period (ASHSC 2012) (Figure 3.3-2).

### Liquefaction Potential

The potential for liquefaction from ground shaking is generally considered to be less for ridges in the Kuskokwim Hills than for most unconsolidated deposits in lowland areas of this region. The liquefaction potential of unfrozen fine sand and silt deposits, conditions that could occur in late summer along the banks of the Kuskokwim River, is considered to be greater. Most boreholes that encountered these types of soils along the proposed Angyaruaq (Jungjuk) and BTC roads and ports were either unsaturated (upland silt deposits) or contained permafrost, and thus have lower liquefaction potential (DMA 2007a, 2007b; RECON 2011a).

There is a potential for liquefaction from ground shaking for unconsolidated deposits and man-made structures in lowland areas of Dutch Harbor, in areas underlain by fine grained unconsolidated sediment.

#### 3.3.2.2.2 SLOPE STABILITY

##### Angyaruaq (Jungjuk) Access Road and Port

From the Mine Site to Juninggulra Mountain, the proposed Angyaruaq (Jungjuk) road would cross the lowland of Crooked Creek, then steadily ascend ridge crests and proceed over upland terrain. Along the ridge crests, the proposed route encounters moderate slopes (RECON 2011a). From Juninggulra Mountain south, the route continues along ridges with moderate to steep terrain. From Getmuna Creek south, the terrain gradually transitions from steep to a low-lying drainage valley. From the Getmuna Creek drainage, the proposed road would ascend Basalt Pass and cross broad ridge crests with moderate slopes. From Basalt Pass to Jungjuk Creek, it would continue along a north-south trending ridge with moderate slopes, then descend to the first crossing of Jungjuk Creek. From Jungjuk Creek to the proposed port site, the alignment follows the drainage valley of Jungjuk Creek, with gently sloping terrain, until reaching the Kuskokwim River (RECON 2011a). Terrain at the proposed port site on the Kuskokwim River is either benched or sloping gently to the south (DMA 2007b). Because the area is relatively flat, the potential for slope movement would be expected to be less than steeper sloped areas.

Most of the proposed Angyaruaq (Jungjuk) Road and port areas would likely not experience significant slope movement, since the route follows less extreme topography favorable for road building. The highest potential for slope movement in this area lies in the narrow stream drainages in moderate to steep terrain (Reger et al. 2003e). Two sections of the proposed road exhibit these characteristics: from Juninggulra Mountain to Getmuna Creek; and from Basalt Pass to the Jungjuk Creek drainage.

##### Birch Tree Crossing Access Road and Port

Slope conditions for the northeastern part of the potential BTC Road from the Mine Site to Juninggulra Mountain are the same as those described above for the Angyaruaq (Jungjuk) route. From Juninggulra Mountain, the BTC route traverses ridge crests with moderate to steep slopes then descends into the Iditarod River drainage lowlands. From there, the route ascends gentle to moderate slopes and then descends into the Cobalt Creek drainage lowlands. From Cobalt Creek, the route traverses gentle to low lying slopes of the Owhat River drainage until reaching Toro Creek. From Toro Creek to Ones Creek this alignment traverses open ridges with gentle to moderate slopes. From Ones Creek to its terminus, the route traverses lowlands of the Kuskokwim River to the potential BTC Port site, located on gently sloping terrain along the north bank of the river (DMA 2007a).

Most of the potential BTC Road and Port would likely not experience significant slope movement. Slope movement in this region could occur, however, in narrow stream drainages with moderate to steep terrain (Reger et al. 2003c). The one area of the BTC alignment with some of these characteristics is the portion that traverses the northwest side of Juninggulra Mountain.

### Kuskokwim Corridor and Bethel Port

In general, the proposed river transportation corridor would not be likely to experience significant slope stability geohazards, since the topography of the Kuskokwim Mountains consists of gentle to moderate sloping hills, and the lower Kuskokwim corridor is predominantly alluvial lowlands that extend to Bethel. The Bethel Port site (connected action) is located within low-lying, flat, flood-plain deposits along the western bank of the Kuskokwim River, and has a low potential for slope movement. Lateral river erosion that could undercut banks and create localized areas of slope failure along the river is described in Section 3.5. Any actions that would occur at Dutch Harbor or the Port of Bethel at the Bethel Yard Dock are not part of the proposed action, and are considered connected actions (see Section 1.2.1, Connected Actions, in Chapter 1, Project Introduction and Purpose and Need).

### Dutch Harbor

Unalaska and Amaknak Islands consist of steep-sloped mountains and low-lying, narrow drainages. On Amaknak Island, steep cliff faces are common (Lemke and Vanderpool 1995). Depending on its specific location, the proposed Dutch Harbor Port could have a potential for slope movement. Any actions that would occur at Dutch Harbor or the Port of Bethel at the Bethel Yard Dock are not part of the proposed action, and are considered connected actions (see Section 1.2.1, Connected Actions, in Chapter 1, Project Introduction and Purpose and Need).

### 3.3.2.2.3 OTHER GEOHAZARDS

#### Tsunamis

The Dutch Harbor area has a potential to experience tsunamis generated by a large-magnitude earthquake. Two earthquakes in 2011 generated tsunami warnings for Dutch Harbor. The March 11, 2011, 9.1 magnitude Fukushima earthquake in Japan produced a tsunami that reached Dutch Harbor and Adak. On June 23, 2011, a 7.3 magnitude earthquake centered near Adak, west of Dutch Harbor, generated a tsunami warning that advised residents of Dutch Harbor and Unalaska to evacuate to high ground (NOAA 2011). Tsunami hazard potential in Dutch Harbor would be high following a large magnitude earthquake generated in the Aleutian Chain or Pacific Ocean basin (Suleimani et al. 2002, Waythomas et al. 2009).

#### Volcanoes

##### *Regional Setting and Processes*

Alaska contains more than 50 volcanoes considered to be active, having erupted in the last few hundred years (Alaska Volcano Observatory [AVO] 2013a). The ongoing collision between oceanic and continental tectonic plates along the Alaska-Aleutian subduction zone generates heat that causes eruptions of active volcanoes located in the Aleutian Islands, Alaska Peninsula, and Cook Inlet. There are four types of volcanoes: cinder cones, lava domes, shield, and composite or stratovolcanoes. In Alaska, the majority are stratovolcanoes (Schaefer et al. 2011).

There are also different types of eruptions, and these play a major role in potential geohazards. These include gas emissions, lahars<sup>4</sup>, debris flows, lava flows, pyroclastic<sup>5</sup> density currents, and tephra<sup>6</sup> air fall. Volcanic eruptions are hazardous to aircraft, air quality, and infrastructure (Casadevall 1994; Guffanti and Miller 2013; Waythomas et al. 2013). Volcanic gases are the driving force of volcanic material released into the atmosphere and distributed by wind, both during and between eruptive episodes.

The majority of active volcanoes in Alaska are located along the 1,600-mile long Aleutian Island Chain, the Alaska Peninsula, and western shores of Cook Inlet (Figure 3.3-3). Since 2005, Mount Cleveland and Veniaminof volcanoes, located near Unalaska Island and the southwest extent of the Alaska Peninsula, respectively, have been the most active, both erupting in 2013 (AVO 2013a). Other recent eruptions include Pavlov volcano, west of Cold Bay, which erupted in 2007 and again in 2013 (AVO 2013a).

### *Dutch Harbor*

The Dutch Harbor fuel depot is located in the Aleutian arc, host to numerous active volcanoes – including Makushin volcano, 15 miles west of Dutch Harbor. Volcanic geohazards at Dutch Harbor are most likely to originate from three active volcanoes of the Aleutian arc. Mount Cleveland is the most active volcano near Dutch Harbor. Okmok volcano, about 73 miles west of Unalaska Island (Dutch Harbor), erupted in 2008. Makushin volcano, on Unalaska Island, last erupted in 1995.

Since 2007, Mount Cleveland has erupted every year except 2012 (AVO 2013a, Neal et al. 2011). Mount Cleveland is a stratovolcano on Chuginadak Island, approximately 158 miles southwest of Unalaska. Its two most recent eruptions were in May of 2013 and July 2011. Both of these eruptions were explosive in character and developed tephra plumes.

In July 2008, Okmok volcano, on the island of Umnak, erupted with a series of explosive eruptions that continued over a period of five weeks and produced severe volcanic ash fall, pyroclastic flows, lahars, and tephra ash clouds (Neal et al. 2011; Larsen et al. 2009; AVO 2013a). The ash cloud drifted east-northeast towards Unalaska Island, following the prevailing northeast wind direction along the Aleutians toward the Alaska Peninsula (Neal et al. 2011).

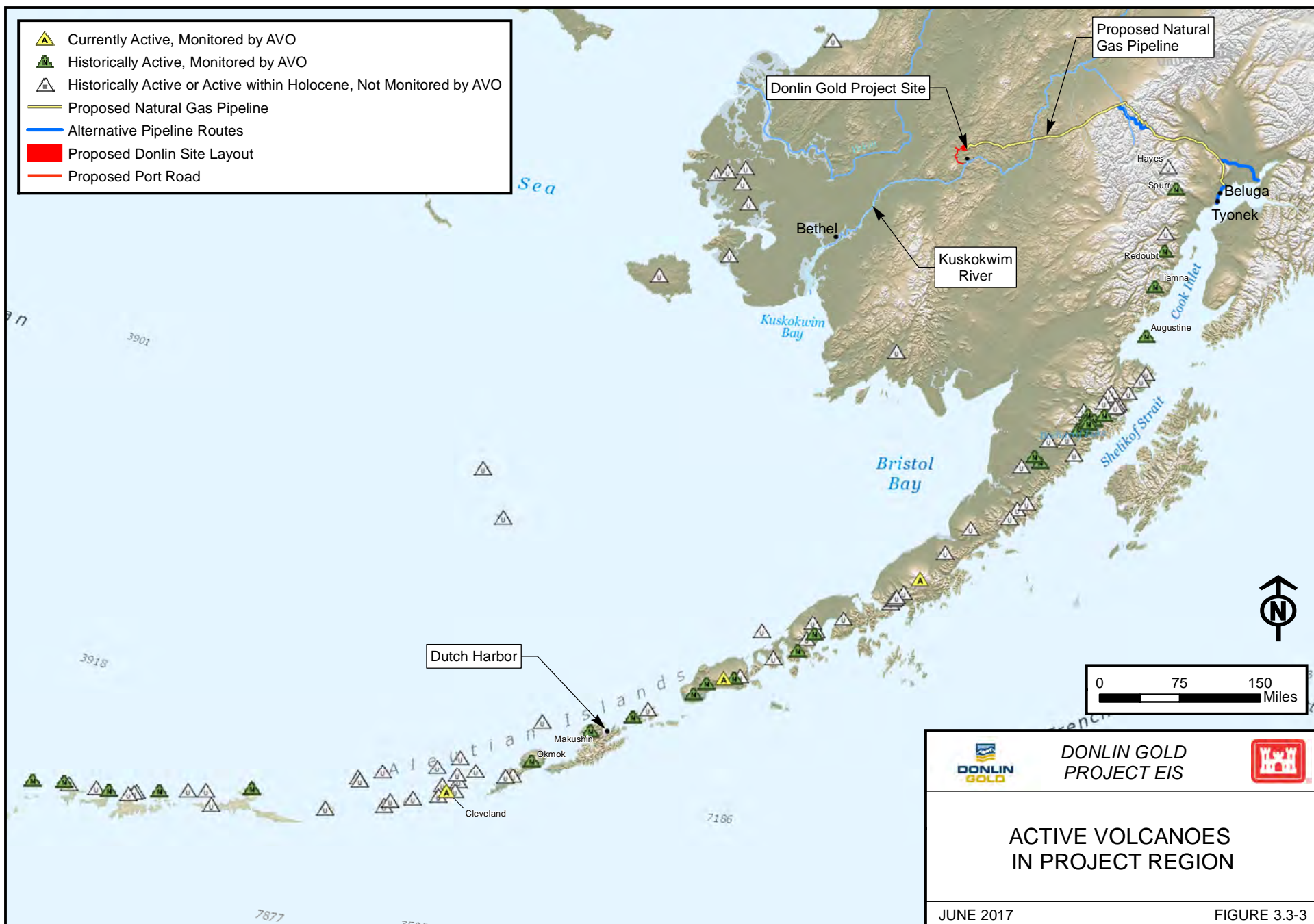
Makushin volcano has produced three documented eruptions – in 1980, 1987, and 1995 – and has been monitored for seismic activity since July 1996 to help detect signs of potential future activity (Begét et al. 2000; Neal et al. 2009; Dixon et al. 2013; AVO 2013a). At least 17 explosive eruptions have occurred at Makushin volcano since the 1700s, and the Pleistocene-age Makushin Volcanic Field contains evidence of past lava flows, mud-flows, and pyroclastic tuffs (McConnell et al. 1997; Begét et al. 2000). The last major eruption occurred on January 30, 1995, and produced an ash-laden steam plume. The plume was observed by pilots to reach an altitude of approximately 8,000 feet. It traveled northeast from the volcano until dispersing several hours later (McGimsey and Neal 1996; Begét et al. 2000; AVO 2013a). Volcanic ash clouds and fallout, lahars, and floods are potential geohazards that could reach Dutch Harbor from future eruptions of Makushin volcano (Begét et al. 2000).

---

<sup>4</sup> A lahar (also referred to as a volcanic mud-flow) is a hot or cold mixture of water-saturated rock debris flowing down slopes of a volcano under the force of gravity.

<sup>5</sup> Pyroclastic flows are high-density mixtures of hot, dry rock fragments and gases that move at high speeds and generally follow valleys.

<sup>6</sup> Tephra is a term for fragments of rock and lava that are ejected into the air and fall back to earth as volcanic ash.





### Avalanches and Glaciers

The area from the Mine Site to Bethel has low topography, and there are no reports of avalanche or glaciers along this route.

Dutch Harbor is located near the glacier-covered Makushin volcano, which contains the highest concentration of glaciers in the Aleutian Islands (Molnia 2008). Glacier-fed lakes can produce seasonal flooding that moves rapidly into established channels or develops new flood channels. The potential for experiencing avalanche or glacial geohazards in Dutch Harbor locally is likely to be minimal, because any avalanche or glacial outburst flood that originates on the slopes of Makushin volcano would probably be contained within the edifice of the mountain and would not be likely to travel far. Local snow slides are possible in areas of steeper slopes around the harbor, such as those along beach bluffs.

### 3.3.2.3 PIPELINE

#### 3.3.2.3.1 EARTHQUAKES

##### Active Faults

There are two primary regional-scale structural features along the proposed pipeline component. The Denali-Farewell fault system is the dominant morphological feature expressed along the northwestern flank of the Alaska Range, and parallels the bulk of the western half of the proposed pipeline corridor (Figure 3.3-1). In the eastern portion of the proposed corridor, the primary regional scale structural feature is the southwest extent of the Castle Mountain-Lake Clark fault system, and a sub-parallel splay off of that system, the Bruin Bay Fault (Wilson et al. 2012; Haeussler and Saltus 2011; Koehler and Reger 2009, 2011). The Denali-Farewell and Castle Mountain-Lake Clark fault systems are considered to be active along many sections of their lengths, as both faults show evidence of horizontal and vertical displacement occurring since the Holocene period (Koehler et al. 2012a; Koehler 2013).

The planned pipeline would cross the Farewell segment of the Denali Fault about one to two miles west of the South Fork Kuskokwim River (MP 149), and the Castle Mountain Fault in the Susitna Lowlands at MP 7.5 near Beluga. At fault crossings the pipeline would be constructed aboveground. The estimated length of each aboveground fault crossing is approximately 1,400 feet.

##### *Castle Mountain Fault Crossing (MP 6)*

The Castle Mountain Fault begins in the Copper River basin area, approximately 120 miles east of the east end of the proposed pipeline corridor, and extends west through the Susitna Lowlands. The Castle Mountain Fault has produced earthquakes with high magnitude since 1933, with the most recent in 1984. Approximately 15 miles of right-lateral displacement have been documented along the Lake Clark segment in the Alaska Range since the Late Eocene (Haeussler and Saltus 2005). Studies by Reger et al. (2003b) and Koehler and Reger (2011), however, suggest that the location of the planned fault crossing at about MP 6 is in an area with low potential for future displacement. Reger et al. (2003b) suggest liquefaction potential in fine-grained, unconsolidated tideland deposits from Beluga northeast to the Susitna River.

No evidence of fault gouge or other characteristics indicating movement were reported in four shallow borings drilled near the potential Castle Mountain Fault crossing. These borings encountered peat from the surface to a depth of up to 3 feet, underlain by silty sand to a depth of up to 7.5 feet. Below the silty sand, the subsurface conditions are predominantly a mixture of silt, silty sand with gravel, and gravel with sand to depth of up to 26.5 feet bgs (BGC 2013c). No paleoseismic hazard mapping trench studies across the fault trace have been performed at this location.

The slip rate for the Castle Mountain Fault at the eastern end of the proposed pipeline route is reported to be less than one inch per year (Koehler et al. 2012b; Wesson et al. 2007). The low slip rate and a recurrence time of 4,255 years suggest that the potential for surface rupture and lateral displacement at this fault crossing is low (Wesson et al. 2007).

#### *Denali-Farewell Fault Crossing (MP 149)*

The Denali Fault extends from northwestern British Columbia through Southcentral Alaska, and continues into Southwest Alaska as a system of discordant segments extending to Dillingham. The Denali Fault has been the source of two major earthquakes with magnitudes greater than 7.0; one in 1912, and one in 2002. The 2002, 7.9-magnitude Denali earthquake, was initiated by 12 feet of normal-thrust faulting on a previously unknown small splay of the Denali Fault called the Susitna Glacier Thrust Fault (Haeussler 2009; Carver et al. 2004). This event produced surface ruptures for a distance of 200 miles, with lateral displacement as large as 28 feet along the central portion of the fault, and predominant north-side-up vertical offset. Studies of the Denali Fault suggest that the location of the planned fault crossing at MP 149 is in an area with low to moderate potential for future displacement (Haeussler 2009; Koehler et al. 2011; Roeske et al. 2012). Reger et al. (2003d) suggest major liquefaction potential in fine-grained, unconsolidated deposits of the South Fork Kuskokwim River and Sheep Creek drainages.

No evidence of fault gouge or other characteristics indicating movement were reported in three shallow borings drilled at the fault crossing. The borings encountered predominantly fine-grained sandy silt and sand materials overlying bedrock. The depth to bedrock was at 10 and 20 feet in two of the borings (BGC 2013c). No paleoseismic hazard mapping trench studies across the fault trace have been performed at this location.

The slip rate estimated for the Denali Fault near Farewell is less than its slip rate in the central Alaska Range and southeastern end of the fault. The low slip rate and a recurrence time of 15,305 years near Farewell suggests that potential fault movement at the Denali Fault crossing is lower than at other places along its length (Wesson et al. 2007).

### Ground Shaking and Seismic Hazard Analysis

#### *Regional Setting*

Significant ground shaking could occur in the pipeline region from the surface faults described above, as well as the buried Alaska-Aleutian megathrust and Iditarod-Nixon Fork Fault (Section 3.3.2.1.2). The 1964, 9.2-magnitude Great Alaska Earthquake, centered along the megathrust system, shook Alaska for several minutes and generated surface ruptures, subsidence, rock falls, landslides, massive submarine landslides, and numerous tsunami waves that inundated communities within the Southcentral region (Plafker et al. 1969; Combellick 1999; Freymueller et al. 2008). The epicenter was located approximately 80 miles east-southeast of Anchorage in

the Prince William Sound region, at a depth of approximately 21 miles (Hansen et al. 1966). The earthquake caused impacts to communities across an area of approximately 50,000 square miles (Plafker et al. 1969).

In addition to causing 200 miles of surface rupture (Section 3.3.2.3.1), the 2002 Denali earthquake induced numerous debris slides and avalanches (Haeussler 2009; Koehler et al. 2011; Schultz et al. 2008). The epicenter of the earthquake occurred at a depth of approximately 2.5 miles, in the central Alaska Range near the West Fork Glacier, about 300 miles east of the proposed pipeline route (Ruppert et al. 2008). The earthquake produced shaking that was felt up to 2,100 miles from the epicenter (Haeussler 2009).

The statewide seismic hazard map covering the proposed pipeline region shows a gradational increase in estimated peak ground acceleration from the Mine Site to the Beluga/Tyonek area (Figure 3.3-2). Seismic hazard assessments of the proposed pipeline route were evaluated for peak ground horizontal acceleration values, and estimated earthquake magnitudes were developed under operational level and contingency level earthquake scenarios. A description of historic seismicity and seismic hazards analysis for the proposed pipeline corridor is divided into two parts below: the western segment from the Mine Site to Farewell, and the eastern segment from Farewell to Beluga/Tyonek.

#### *Mine Site to Farewell*

A search for earthquake events with a minimum magnitude 4 in the western portion of the proposed pipeline corridor yielded seven events since 1932. Two events, one in March 1932 and one in June 1932, reported magnitudes of 6.0. The remaining five events had magnitudes of 4.0-4.1 and occurred as recently as February 1992 (AEIC 2014). Epicenters for both 1932 events occurred at a depth of 15 miles and were located approximately 24 miles east of Farewell Lake. These two earthquakes likely represent movement on the Farewell segment of the Denali Fault system (Koehler 2013).

Based on published maps by ASHSC (2012), peak ground acceleration for a 475-year average return period is estimated to range from 0.10 to 0.15g over the western portion of the proposed pipeline corridor. CH2MHill (2011a) conducted probabilistic and deterministic seismic hazard assessments of the corridor in order to develop project-specific peak ground accelerations and maximum earthquake magnitudes under operational level earthquake and contingency level earthquake scenarios. These correspond to return periods of 150 and 500 years, respectively. The estimated peak horizontal ground accelerations for return periods of 150 and 500 years for the Western Alaska portion of the proposed pipeline (MP 316 to MP 145) are 0.06-0.10g and 0.11-0.19g, respectively. The estimated earthquake magnitude for the western portion of the Denali Fault under the operational level scenario is 5.5, and for the contingency level magnitude is 6.3. Estimated magnitudes for the Iditarod-Nixon Fork Fault, located near the west end of the pipeline, are the same as those described for the Mine Site (Section 3.3.2.1.1).

#### *Farewell to Tyonek*

A search for earthquake events with a minimum magnitude 5 from Farewell to Tyonek yielded 61 events since 1932. Two events, one in April 1933 and another in November 1943, had magnitudes of 7.1 and 7.4, respectively. Three events (in March 1932, August 1962, and June 1991) had magnitudes of 6.9, 6.4, and 6.5, respectively. The remaining 56 events had magnitudes ranging from 4.0 to 6.2 and occurred as recently as December 2012 (magnitude of 5.8). The

epicenter of the 1933, 7.1-magnitude event occurred at a depth of 15 miles below surface near the mouth of the Theodore River. The epicenter of the 1943, 7.4-magnitude event was near the community of Skwentna. The epicenter of the 1932, 6.9-magnitude event was near Shadows Glacier in the Kichatna Mountains. The epicenter of the 1962, 6.4-magnitude event occurred at a depth of 24 miles, within the Johnson Creek drainage north of Happy River. The epicenter of the 1991, 6.5-magnitude event occurred at a depth of 69 miles, near Chelatna Lake just west of the Kahiltna River (AEIC 2014). Most of these events likely represent movement on the Castle Mountain Fault (Koehler 2013).

Based on published maps by ASHSC (2012), peak ground acceleration for a 475-year average return period is estimated to range from 0.15 to 0.35g over the eastern portion of the proposed pipeline corridor (Figure 3.3-2). A project-specific, probabilistic seismic hazard assessment conducted by CH2MHill (2011a) resulted in estimated peak horizontal ground accelerations, for return periods of 150 and 500 years, of 0.15g to 0.17g and 0.22g to 0.25g respectively for the Alaska Range region (MP 145 to MP 96), and for the Cook Inlet region (MP 96 to MP 0) of 0.22g to 0.32g and 0.30g to 0.45g, respectively. The project-specific deterministic seismic hazard study estimated earthquake magnitudes for the central portion of Denali-Farewell Fault: 6.7 for the operational level scenario (150-year return period); and 7.4 for the contingency level scenario (500-year return period). The estimated earthquake magnitudes for the western and central portions of the Castle Mountain Fault were determined to be 5.3 (western) and 6.2 (central) for the operational level scenario, and 6.1 (western) and 7.0 (central) for the contingency level scenario (CH2MHill 2011a).

### Liquefaction Potential

The potential for liquefaction from ground shaking along the proposed pipeline route is generally considered to be low to moderate for colluvium on ridges, frozen or coarse-grained unconsolidated deposits in lowland areas, and peat in wetlands. These conditions generally exist in the Alaska Range and Kuskokwim Hills, in numerous wetlands areas along the pipeline (detailed in Wetlands Mapbook appendix to the EIS), and along the north front of the Alaska Range, where areas with shallow groundwater occur in wide braided streams containing coarse alluvium or are associated with areas of shallow permafrost (Figure 2.3-34, Chapter 2, Alternatives). Areas of shallow groundwater along the pipeline are shown on Figure 3.6-1 (Section 3.6, Groundwater Hydrology), and detailed soil conditions at specific MPs along the pipeline are provided in the Soil and Permafrost Data appendix to the EIS.

The liquefaction potential of unfrozen, fine sand and silt deposits in low-lying areas with a high water table would be considered moderate to high (Reger et al. 2003b, 2003c, 2003d). These types of conditions could occur in late summer in low-gradient drainages along non-permafrost sections of the proposed pipeline. Such conditions could occur locally in low gradient stream crossings of the Kuskokwim Hills or in lower Cook Inlet (CH2MHill 2011b). The 1964 earthquake caused extensive subsidence and liquefaction in the Cook Inlet region, where ground breakage occurred mostly in fine-grained, unconsolidated sediments near the shoreline, and locally in terraced moraine or channeled glacial deposits underlain by sand and silt (Foster and Karlstrom 1967).

### 3.3.2.3.2 SLOPE STABILITY

The western portion of the proposed pipeline corridor from the Mine Site to Farewell would not be likely to experience significant slope stability hazards since the topography of the Kuskokwim Mountains and upper Kuskokwim River region consists of gentle- to moderate-sloping upland hills. The highest potential for slope movement in this area would be in narrow stream drainages with moderate to steep terrain (Reger et al. 2003a, 2003d).

Likewise, the eastern portion of the proposed pipeline corridor from Tyonek and Beluga (MP 0) to the upper Happy River drainage (MP 108) consists of gentle slopes with low potential for slope instability. Any potential slope movement within the Susitna Lowlands would likely be along narrow stream drainages with moderate to steep terrain (Reger et al. 2003b).

The central portion of the proposed pipeline route, from Farewell to the upper Happy River drainage, would be likely to experience moderate to severe slope stability hazards through the steep mountainous areas of the Alaska Range, from approximately MP 108 to MP 145. Detailed terrain mapping that classifies surficial deposits based on slope stability has been completed for this part of the proposed corridor (BGC 2013a). Terrain mapping uses digital elevation models to evaluate geohazards and drainage density, and involves development of polygons to show local variations in bedrock geology, soil types, and landforms.

From Jones Creek to the Dillinger River drainage (MP 145), the proposed route traverses moderate to steep slopes with thin veneer colluvium covering bedrock and active mass wasting processes occurring on both sides of the corridor (Figure 3.3-4). From MP 111 to MP 139, 37 debris flows have been identified on the southwest side of the proposed route, and 30 debris flows have been identified on the northeast side (BGC 2010, 2013a). These areas are described in more detail below.

#### Happy River to Three-mile Creek (MP 108 to MP 113)

From MP 108 to MP 109, under the proposed alternative and the MP 84.8 to 112 North Option, the proposed pipeline route traverses bedrock mantled by glacial till, with undulating topography. On the west side of the planned pipeline course, five rapid-moving debris avalanches consisting of thin veneer of colluvium over steep exposures of bedrock have been identified, as well as one on the south side of the proposed pipeline course. From MP 109 to MP 113, the proposed route traverses bedrock mantled by glacial till across a glaciofluvial plain. At MP 112, two rock avalanches and one rapid-moving rockfall occur on the west side of the proposed pipeline course (BGC 2013a). The distance from these features increases with the North Option.

#### Three-mile Creek to Lower Jones River (MP 113 to MP 139)

From MP 113 to MP 118, nine debris flows occur over steep bedrock exposures mantled by a veneer (less than six feet thick) of colluvium, on the south side of the proposed pipeline course, and 10 on the north side. Several of the slope movements occurred rapidly, causing deep gullyng of the unconsolidated deposits above the bedrock. At MP 119, up to nine rockfall/rock avalanches occur within the colluvium covering the bedrock. From MP 119 to MP 127, 13 debris flows occur over moderate to steep bedrock exposures covered by a thin veneer of colluvium, on the west side of the proposed pipeline corridor, as well as eight on the east side. Of these slope movements, six occurred rapidly, causing deep gullyng of the unconsolidated deposits



above the bedrock. From MP 127 to MP 131, 2 debris flows occur on the west side of the proposed pipeline route, in addition to three on the east. All of these slope movements occurred rapidly, causing deep gullying of the surficial deposits. From MP 131 to MP 139, 15 debris flows occur on the west side of the proposed pipeline route, while 10 occur on the east side (Figure 3.3-4). Of these 25 debris flows, 10 occurred rapidly, causing deep gullying of the veneer of colluvium mantling the moderate to steep exposures of bedrock.

#### Lower Jones River to Kuskokwim Basin (MP 139 to MP 145)

From MP 139 to MP 145, the proposed pipeline route traverses areas of bedrock mantled by glacial till across a glaciofluvial plain, and moderate to steep exposures of bedrock covered by colluvium. As the proposed route nears the mouth of the Jones River, it traverses gentle slopes of colluvium and a glaciofluvial plain. From MP 140 to MP 142, six rock/debris avalanches moved rapidly downslope on the south side of the proposed pipeline course, and two on the north. From MP 142 to MP 145 the proposed pipeline route traverses undulating topography and an increasing amount of glacial till and organic material covering bedrock.

### 3.3.2.3.3 OTHER GEOHAZARDS

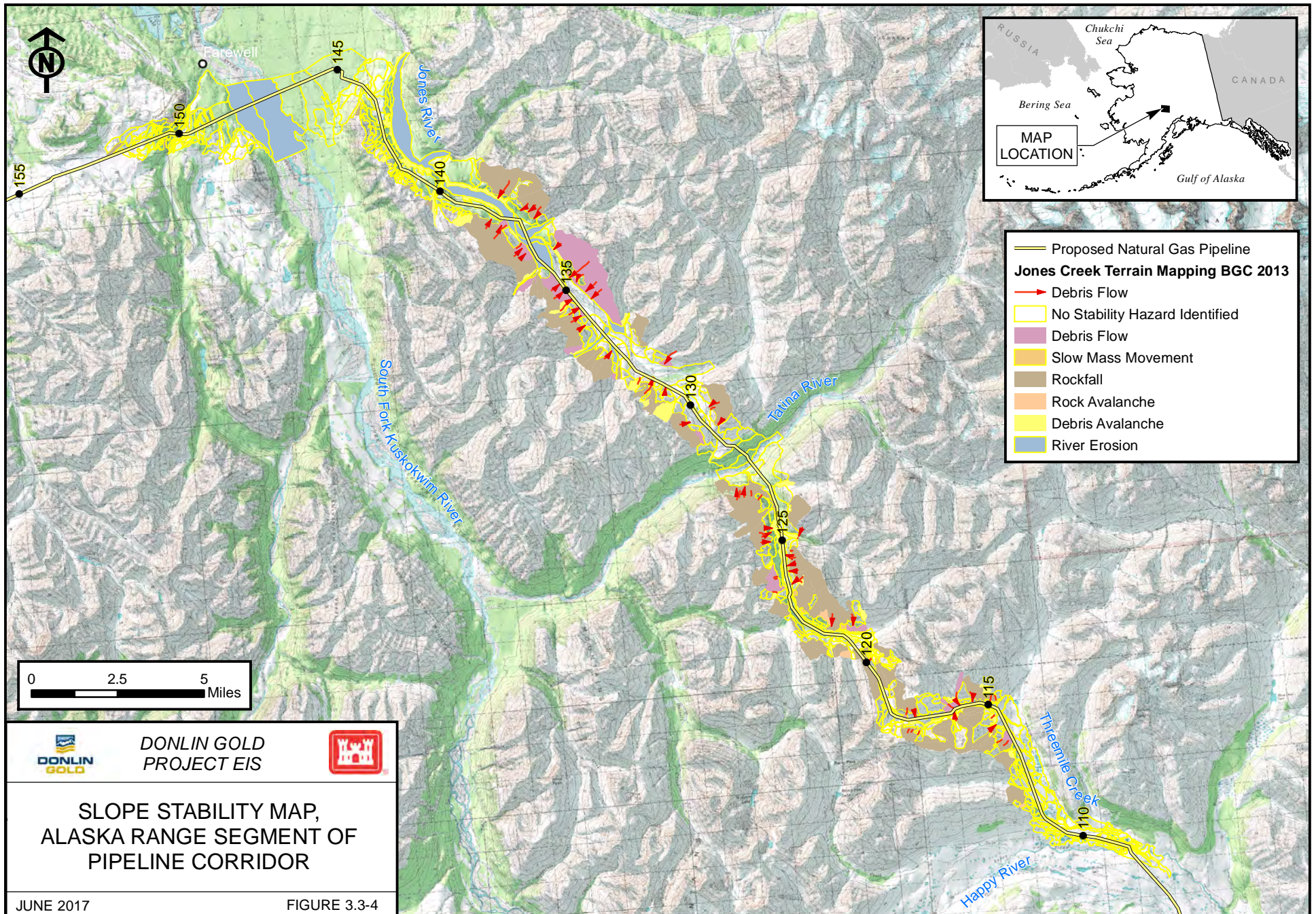
#### Geotechnical Considerations at Horizontal Directional Drilling River Crossings

The proposed pipeline corridor crosses six major rivers where the pipeline may be installed using horizontal directional drilling (HDD) methods: Skwentna River (MP 50), Happy River (MP 86), Kuskokwim River (MP 240), East Fork George River (MP 283), George River (MP 291), and the North Fork George River (MP 298). Under the North Option route, two HDD crossings of Happy River would be required. The locations of entry and exit points for each of the proposed crossings are shown on pipeline strip maps in Appendix D.

Geotechnical engineering investigations included the advancement of 15 deep exploratory boreholes at these crossings, and a geophysical survey of the Happy River drainage, to evaluate suitability of HDD methodology for pipeline installation (CH2MHill 2011a). Nearby shallow borings and test pits drilled along the alignment supplemented the deep borings to provide information on conditions at the HDD entry and exit points.

The HDD method was first introduced in the 1970s. Since then, systems have been designed that can bore pipe up to 48 inches in diameter, 200 feet below the surface, with 6,000 feet of horizontal distance. Several dozen projects have been already completed in Alaska with similar sized lines (for example, the Colville River crossing to the Alpine oil field). While HDD is typically best suited for small-diameter pipeline installation, it is considered to be a reasonable approach for this project as long as favorable subsurface conditions are present. If thick sequences of loose gravel soils containing cobbles and boulders, or highly fractured bedrock are present, the HDD procedure can become impaired, affecting drill advancement rates and increasing the difficulty of maintaining an unobstructed path for the drill string. The presence of either cobble-containing gravel soils or highly fractured bedrock may also cause loss of drilling fluid circulation, or release of drill fluids into the overlying water body, a condition referred to as “frac-out,” which has the potential to affect water quality in the river. The geotechnical conditions at individual proposed HDD crossings are described below, and the feasibility and frac-out risk associated with each are described in Section 3.3.3.2.3.







### *Skwentna River (MP 50)*

The proposed Skwentna River HDD crossing is approximately 2,500 feet in length, with moderate topography. The northwest shore has a gentle slope and the southeast shore consists of a small hill with moderate slope. The river crossing location consists of two channels separated by a gravel bar. There are bedrock outcrops on both sides of the river, which lie within 3,000 feet from the alignment on the southeast bank. Previous investigations identified a clay layer at a depth of approximately 50 feet as the target zone for HDD pipe installation.

Three exploratory borings were advanced at this location, one on each bank, and one at the gravel bar island separating the two channels. These encountered mostly sand and gravel with cobbles to depths of 57 and 59 feet, underlain by hard lean clay and dry silt to depths of 78 to 101 feet. In one boring, the silt layer was underlain by another coarse gravel zone that extended to a maximum drilled depth of 86 feet (CH2MHill 2011a).

### *Happy River (MP 86)*

One Happy River HDD crossing would be installed under the proposed alternative, and two HDD crossings of Happy River tributaries would be installed under the MP 84.8 to 112 North Option. The HDD crossings would range from approximately 3,200 to 3,700 feet long between entry and exit points, and would extend through unconsolidated glacial till overlying metamorphic bedrock. A seismic refraction geophysical survey was conducted to evaluate the thickness of the unconsolidated material above the bedrock. The geophysical survey results indicated that the glacial till is underlain by a weathered bedrock zone approximately 100 feet thick, which in turn is underlain by competent bedrock. The thickness of the glacial till was interpreted to be approximately 75 feet thick.

Three deep borings drilled at the Happy River HDD site encountered 30 to 120 feet of gravel, sand, and silt glacial-outwash deposits above bedrock across the channel. The bedrock consisted of greywacke sandstone with interbedded siltstone, which was fractured extensively at depths up to 198 feet (Flanigan 2011). Three shallow borings drilled near the proposed HDD entry/exit points encountered coarse alluvial material to maximum drilled depths of 6 and 13 feet.

Bedrock conditions are expected to be similar for the North Option Happy River tributary HDD crossings (Wilson et al. 2012). Overburden deposits are expected to be of similar type but thinner than those encountered at the Happy River HDD site.

### *Kuskokwim River (MP 240)*

The proposed Kuskokwim River HDD crossing is roughly 1-1/2 miles long, and would bore beneath two channels separated by a gravel bar island with mature vegetation. Three exploratory borings were advanced, one on each bank and one at the gravel bar island separating the two channels. These encountered sandy silt and silty sand to depths of 5 to 11 feet; then alluvial sand with gravel, silt, and cobbles to depths of 42 to 129 feet; which was underlain by weathered and fractured siltstone and mudstone bedrock to maximum drilled depths of 54 to 135 feet (CH2MHill 2011a).

### ***East Fork George River (MP 283)***

The proposed East Fork George River HDD crossing would be approximately 1.1 miles long between entry and exit points. The floodplain consists of a low-lying, meandering channel with remnant oxbow orphaned channels having little to no vegetation. Two deep exploratory borings were advanced in the river channel. These encountered silt and silty sand to depths of 2.5 to 5 feet, then alluvial sand and gravel to depths of 16 to 20 feet. Below the alluvial material, weathered sandstone and mudstone bedrock was encountered to maximum drilled depths of 28 to 65 feet.

### ***George River (MP 291)***

The proposed HDD crossing location for the main stem of the George River would be about 3,200 feet long and consists of a low-lying, meandering channel with moderate vegetation. Two exploratory borings were advanced in the river channel and two on the river banks. The boring on the east bank encountered highly weathered and fractured sandstone bedrock at a shallow depth of 4 feet beneath silty gravel. The borings in the river channel and on the west bank encountered slightly to highly weathered and fractured bedrock at depths of 11 to 24 feet, beneath alluvial sand, silt, and gravel with cobbles up to two inches. The borings were advanced to total depths ranging from 15 to 36 feet (CH2MHill 2011b).

### ***North Fork George River (MP 298)***

The proposed North Fork George River HDD crossing would be about 3,300 feet long and consists of a narrow, sloped stream channel with moderate vegetation. One exploratory boring was advanced in the river channel and two borings on the river banks. The two borings on the river banks encountered silty sand, silt, and alluvial silty gravel with cobbles to 2.5-inches to depths of 4 to 8 feet, underlain by slightly weathered and fractured sandstone and mudstone bedrock to total depths of 13 to 46 feet (CH2MHill 2011b).

### **Coastal Hazards and Tsunamis**

The eastern portion of the diesel pipeline alternative near Beluga and Tyonek has a low to moderate potential for coastal geohazards and tsunamis from seismic events. Impact from tsunamis is dependent on bathymetry, coastline configuration, and tidal interactions. Vertical sea-floor displacements following large earthquakes along the Alaska-Aleutian megathrust have produced widespread damage along the Alaskan Pacific coast and other exposed locations around the Pacific Ocean (Suleimani et al. 2002; 2005). The 1964 Great Alaska Earthquake generated numerous tsunami waves, including several that destroyed the Kodiak harbor (Plafker and Kachadoorian 1966). Tsunami wave height predictions in Upper Cook Inlet are generally considered to be low, however, due to the shallow seafloor in this area. Tsunami wave height in the Beluga/Tyonek area is estimated to be 12 to 14 feet at high tide for 100- to 500-year return period events (Crawford 1987). While no tsunami estimates are available for Dutch Harbor, estimates for the 100- to 500-year events for Kodiak Island, which is similarly exposed to the Pacific Ocean basin, range from 6 to 62 feet.

Along the northwest shoreline of Cook Inlet, stream mouths were drowned and narrow beaches backed by bluffs experienced vigorous erosion of bluff faces during the 1964 earthquake (Stanley 1968). The Susitna River delta and outlet of Beluga Lake showed evidence of slumping

from the tectonic event, and approximately three feet of subsidence was observed near Tyonek (Foster and Karlstrom 1967).

In addition to earthquake-generated tsunamis, volcanic eruptions can produce tsunamis from debris flows entering the sea that cause extensive damage at great distances from the erupting volcano (Waythomas et al. 2009). The 1883 eruption of Augustine Volcano produced a tsunami 9 to 24 feet high, and affected communities within Lower Cook Inlet (Begét et al. 2008; Kowalik and Proshutinsky 2010).

## **Volcanoes**

### ***Regional Setting***

The eastern portion of the proposed pipeline route is located within 200 miles of several active volcanoes (Figure 3.3-2). If an eruption were to occur, the likely volcanic hazard to affect the proposed corridor would be air-fall volcanic ash, the thickness of which would depend on the distance of the erupting volcano from pipeline infrastructure, prevailing wind direction, and duration of the eruption event.

There are several active volcanoes and volcanic clusters that pose potential volcanic threats to the eastern portion of the proposed pipeline. Active volcanoes of Cook Inlet within about 200 miles of the proposed pipeline corridor include Spurr, Redoubt, Iliamna, and Mount St. Augustine (Figure 3.3-3) (AVO 2013a). Southwest of Cook Inlet lies the Katmai Volcanic Cluster, a group of seven volcanoes including the Novarupta vent that erupted in 1912, and was the world's most voluminous volcanic eruption of the 20<sup>th</sup> Century (Fierstein and Hildreth 2001; Hildreth and Fierstein 2012). One notable volcanic center that may pose a potential threat to the east end of the proposed pipeline corridor is Hayes volcano in the Tordrillo Mountains. Additional details on these volcanic threats are presented below.

### ***Nearby Volcanoes***

#### **Hayes Volcano**

Hayes volcano is a stratovolcano located approximately 40 miles southwest of the proposed Skwentna River pipeline crossing and 25 miles south of the proposed Happy River crossing, approximately 84 miles from Anchorage. Hayes volcano was not discovered until 1975, and is a deeply eroded volcanic center on the flank of Mount Gerdine (Waythomas and Miller 2002). There are no documented historical eruptions of Hayes volcano; however, thick deposits of volcanic ash from a series of six major eruptions between 3,400 and 3,800 years ago have been identified in Cook Inlet. The eruptions of Hayes Volcano were the most voluminous Holocene eruptions to have occurred in the Cook Inlet region (Fierstein 2012). There is currently no fumarolic activity present. The last known eruption of Hayes Volcano occurred roughly 1,200 years ago.

Due to its location, a future eruption of Hayes volcano could generate an ash cloud that could cause disruption to aviation in the Anchorage Bowl area. Any minor episode on Hayes volcano would be likely to melt ice and snow that could generate a lahar or debris flow that would travel down Hayes Glacier and into the Skwentna River drainage.



### Mount Spurr

Mount Spurr is the closest active volcano to the east end of the proposed pipeline corridor. Located in the Tordrillo Mountains, approximately 80 miles west of Anchorage and 40 miles west of Tyonek, it last erupted in 1992 (Keith 1995). In 2004 through 2006, Mount Spurr exhibited minor volcanic activity that included increases in heat flow, seismicity and variable gas dispersion. These geothermal manifestations resulted in melting of snow and ice and subsequent development of debris flows (Coombs et al. 2006; McGimsey et al. 2008). Volcanic ash clouds and fallout, lahars, and floods are the most likely potential geohazards to the east end of the proposed pipeline corridor from future eruptions of Mount Spurr.

### Redoubt Volcano

Redoubt Volcano is an ice-covered stratovolcano approximately 50 miles south of Mount Spurr. In March 2009, following 10 months of intermittent activity including increased volcanic gas and heat flow, ground swelling, and volcanic tremors, Redoubt volcano erupted explosively and subsequently produced 19 major ash clouds that shut down the Anchorage International Airport for 20 hours (Schaefer 2012). In addition to the ash clouds, lahars flowed down the Drift River valley and inundated the Drift River Terminal oil-storage and transfer facility. The Drift River Terminal is approximately 47 miles south of Tyonek. Trace amounts of ash were reported as far north as Fairbanks, Alaska. Prior to this, Redoubt Volcano had last erupted in 1989. Since the 2009 eruption, Redoubt volcano has undergone a period of non-eruptive activity (AVO 2013a). Volcanic ash clouds and fallout are the most likely potential geohazards from future eruption of Redoubt volcano to the east end of the proposed pipeline corridor.

### Iliamna Volcano

Iliamna volcano, approximately 30 miles south of Redoubt volcano, last erupted in 1953 producing a tephra plume and steam emitting from the summit (AVO 2013a). Since 1953, Iliamna volcano continues to show non-eruptive activity, including debris avalanches in 2003 and 2005, steam emissions at its summit in 2011, and isolated seismic activity in 2012 (McGimsey et al. 2008; Dixon et al. 2013; AVO 2013a). Volcanic ash clouds and fallout are the most likely potential geohazards to the proposed pipeline infrastructure from future eruptions of Iliamna volcano.

### Augustine Volcano

Augustine Volcano is an uninhabited island in Cook Inlet, approximately 40 miles south of Iliamna volcano and 150 miles south of Beluga. It is recognized as the most historically active volcano in the Cook Inlet region (Powers et al. 2010). In 2005, precursor seismic activity began in April and continued until December, when small, localized water-rich explosions produced tephra, which were followed by four explosive eruption phases beginning on January 11, 2006 (McGimsey et al. 2008; Neal et al. 2009). The eruptions produced volcanic gases, snow-rich avalanches, pyroclastic flows, lahars, and lava flows. Following the 2006 eruption phases, the volcano began a period that produced steam plumes and isolated seismic events (McGimsey et al. 2011; AVO 2013a). Volcanic ash clouds are the most likely potential geohazards from future eruptions of Augustine volcano along the east end of the proposed pipeline corridor.

### Avalanches and Glaciers

The proposed pipeline route from Farewell to Beluga crosses the steep mountainous terrain of the Alaska Range, making the eastern portion of the corridor susceptible to avalanches that

could be generated by an earthquake or rapid change in local weather conditions, or potential outburst flooding from glaciers. Numerous avalanches and rock slides were triggered by the November 2002, 7.9-magnitude Denali Fault earthquake (Truffer et al. 2002; Schulz et al. 2008).

The potential for experiencing local avalanches along the east-central portion of the proposed pipeline will likely be moderate to high. Unlike for the Chugach National Forest east of Cook Inlet, there is no public avalanche forecast for the remote regions of the Alaska Range. Snow and ice avalanches may be triggered by ice masses breaking off from hanging glaciers (Margreth et al. 2011). The dish-shaped nature of hanging glaciers creates a very steep transition from steep or vertical terrain to relatively flat ground below (Wright and Chenoweth 2012).

Avalanche hazard along the corridor from MP 75 to MP 144 may occur where slopes are steeper than approximately 30 degrees. Avalanches occur most often in preexisting paths in mountainous terrain where unstable accumulations of snow and ice are deposited. In general, avalanches result when low shear strength within a snow slab triggers a rapid increase in stress and slab failure (Birkeland et al. 1995). Several factors determine whether an avalanche occurs, including terrain, slope angle, weather, temperature, and snowpack conditions. Variation in temperature and precipitation throughout the Alaska Range occurs both laterally and vertically (Wright and Chenoweth 2012). Large mountain avalanches are often naturally released when the snow pack becomes unstable and layers of snow begin to fail.

Extensive valley glaciers are present on the southeast side of the Alaska Range, and smaller ones are found on the northwest and west side of the Alaska Range (Wahrhaftig 1965; Molnia 2008). Glacier-fed lakes in this area may produce seasonal outburst flooding that moves rapidly into established channels and can destroy infrastructure in its path. The larger glaciers in the Alaska Range can extend up to 36 miles in length (Wright and Chenoweth 2012). Mount Gerdine hosts two glaciers on its east flank, Capps and Triumvirate glaciers, both of which drain into Beluga Lake and the Beluga River drainage, which would be crossed by the diesel pipeline alternative. The Hayes glacier northwest of Tyonek discharges into the Hayes River and Skwentna River drainages. A large eruption of Hayes volcano could also produce lahars and flooding that would flow from Triumvirate Glacier into Beluga Lake (Waythomas and Miller 2002). Similar glacial outburst flooding followed the 2006 eruption of Fourpeaked volcano on the northern Alaska Peninsula, which scoured a steep-walled canyon more than 300 feet in depth and traveled for several miles (Neal et al. 2009).

#### 3.3.2.4 CLIMATE CHANGE

Climate change is affecting resources in the EIS Analysis area and trends associated with climate change are projected to continue into the future. Section 3.26.3 discusses climate change trends and impacts to key resources in the physical environment including atmosphere, water resources, and permafrost. Current and future effects on geologic hazards, particularly geomorphological processes (landslides and avalanches) and coastal hazards (flooding and erosion), are tied to changes in water resources (discussed in Section 3.26.3.2).

### 3.3.3 ENVIRONMENTAL CONSEQUENCES

This section describes potential impacts for geohazards as a result of the project. Table 3.3-1 provides the impact methodology framework applied to assessing direct or indirect impacts for geohazards based on four factors of intensity, duration, extent or scope, and context (40 CFR 1508.27, described in Section 3.0, Approach and Methodology).

**Table 3.3-1: Impact Methodology for Geohazard Effects**

Type of Effect	Impact Component	Assessment Criteria		
<b>Changes to Physical Resource Character or Geohazard Effects on Project</b>	Magnitude or Intensity	Impacts from geohazards or changes in resource character may not be measurable or noticeable.	Noticeable changes in resource character. Design is adequate for expected range of geohazard conditions.	Acute or obvious changes in resource character. Geohazards likely to exceed design parameters.
	Duration	Geohazard impacts or resource changes would be infrequent but not longer than the span of the Construction Phase and would be expected to return to pre-activity levels at the completion of the activity.	Impacts on resource or project component would be intermittent or reduced through the life of the project (from the end of the Construction Phase through the life of the mine, and up to 100 years) and would return to pre-activity levels after actions causing impacts were to cease.	Chronic effects; resource would not be anticipated to return to previous levels even if actions that caused the impacts were to cease.
	Extent or Scope	Impacts limited geographically; discrete portions of the Project Area affected.	Affects resources beyond a local area, potentially throughout the EIS Analysis Area.	Affects resources distant from the EIS Analysis Area.
	Context	Affects usual or ordinary resources; not depleted or protected by legislation.	Affects depleted resources within the locality or region or resources protected by legislation, or resource geohazards governed by regulation.	Affects rare resources or resources protected by legislation.

In evaluating impacts to geohazards, relevant questions and factors addressed in this section include:

- What geohazards could cause impacts to project components that could affect the environment, and must be avoided or designed for? Effects are evaluated for the following categories of geohazards:
  - Earthquakes and related phenomena (e.g., liquefaction);
  - Landslides and slope instability of man-made structures; and
  - Other geohazards such as dam seepage, tsunamis, volcanoes, glaciers, and seafloor hazards.
- What is the likelihood of their occurrence under each alternative?

- What planned engineering design parameters and other safeguards are in place to reduce the risk of impacts? Are there differences in safety amongst the various alternatives?
- Are existing geohazards made worse by an alternative? Is there greater danger to human life and the environment from the project components than previously existed?

The evaluation of geohazard impacts on the project and the environment incorporates an understanding of their probability of occurrence, and of planned mitigation in the form of engineering design and maintenance that can meaningfully reduce impacts. For geohazards that are likely to occur, or are low probability but potentially high impact events, it is acknowledged that noticeable effects could occur that are reliant on a properly executed and maintained design to not reach a higher intensity of impacts. Designs that can sufficiently withstand expected geohazard impacts do not mean there would be no noticeable impacts, but that noticeable impacts could occur that would be manageable or repairable without substantial environmental impacts. Extremely low probability or worst-case scenarios, while not required to be analyzed under NEPA, are described in this analysis where pertinent to project design.

Where known based on Donlin Gold plan documents and engineering reports, planned mitigation (e.g., design and monitoring to withstand or detect geohazards) are considered part of the project description, and ratings criteria are applied with them included. This is also the case where such planned mitigation may not be specified, but is considered typical or standard engineering practice. In cases where planned mitigation is unknown or unclear, and may not be a common situation encountered, the lack of planned mitigation is taken into account in the impact analysis, and mitigation recommendations are provided in Chapter 5, Impact Avoidance, Minimization, and Mitigation, which could reduce impacts.

### 3.3.3.1 ALTERNATIVE 1 – NO ACTION

As there would be no new construction of facilities across the Project Area under the No Action Alternative, impacts to the project from geohazards, seismic events, and other geotechnical conditions would not occur as a result of this alternative. In addition, there would be no impacts to the environment as a result of natural geohazards such as earthquakes and landslides causing damage to project facilities.

### 3.3.3.2 ALTERNATIVE 2 – DONLIN GOLD'S PROPOSED ACTION

As described in the introduction to Section 3.3.2.4, potential impacts from geohazards under Alternative 2 are grouped into three categories: earthquakes (Section 3.3.3.2.1), slope stability (Section 3.3.3.2.2), and other geohazards (Section 3.3.3.2.3).

Seismic and slope stability concerns are common to all three project components and phases. Seismic events can cause ground shaking, surface rupture, liquefaction, and influence the design of major structures. Pipeline fault crossings are driven by the location of active faults and route selection. Slope stability issues include landslides, debris flows, and avalanches that can occur across built features or on natural slopes surrounding project facilities. Other geohazards considered include those associated with dam seepage, tsunamis, volcanoes, and geotechnical considerations for HDD, which depend on river morphology, pipeline route selection, and local subsurface conditions. Volcanic eruptions and tsunamis could impact certain ports and pipeline sections due to source locations and regional extent of impacts.

Based on comments on the Draft EIS from agencies and the public, one route option has been included in Alternative 2 to address concerns due to pipeline crossings of the Iditarod National Historic Trail (INHT):

- North Option: The MP 84.8 to 112 North Option would realign this segment of the natural gas pipeline crossing to the north of the INHT before the Happy River crossing and remain on the north side of the Happy River Valley before rejoining the alignment near MP-112 where it enters the Three Mile Valley. The North Alignment would be 26.5 miles long, with one crossing of the INHT and only 0.1 mile physically located in the INHT right-of-way (ROW). The average separation distance from the INHT would be 1 mile.

### 3.3.3.2.1 EARTHQUAKES

#### Mine Site

##### *Construction*

No mine facilities would be constructed on top of known active faults. As presented in Section 3.3.2, numerous northeast-trending faults and minor northwest-trending conjugate fault pairs have been identified within the Mine Site area. The regional Iditarod-Nixon Fork Fault occurs within 15 miles of the Mine Site, and the Donlin Creek Fault is within one mile of the proposed open pit. None of these faults are known to have produced surface ruptures along their traces within the last 10,000 years, are not considered active, and do not represent a threat of potential ground displacement in the event of an earthquake.

Impacts caused by ground shaking during an earthquake could cause damage to major structures at the mine and related impacts to the environment if not mitigated through seismic hazard analysis and appropriate design features. Seismic analysis and design elements that serve to reduce potential impacts from seismic hazards and reduce the intensity of impacts are described below for major mine structures.

##### **Tailings Storage Facility (TSF)**

Hazard Classification: The TSF would be regulated as a Class I (high) hazard potential dam under State of Alaska dam safety guidelines (ADNR 2005) in light of the potential for impacts to life and property, and degradation of anadromous fish habitat in the event of uncontrolled release of the stored tailings and supernatant water in an earthquake. The level of sophistication and approach to dam design are dictated by three factors: the earthquake return period, the maximum credible earthquake (MCE) magnitude, and the maximum design earthquake (MDE) magnitude:

- The return period is the frequency of earthquakes in the site area, based on historical and geologic records, which varies with earthquake size, small earthquakes being more frequent than large earthquakes.
- The MCE is the largest likely earthquake (defined by ground motion at the site) that might be generated at the site. This is determined by analyzing the historical and geologic records over the last 10,000 years. ICOLD (2001) states that the MCE should be calculated using deterministic models. The ADNR has adopted the Corps' Regulation



1110-2-1806, which specifies a deterministic method for evaluation of the MCE and the MDE.

- The MDE represents the potential ground motion from the most severe earthquake considered at the site, relative to the acceptable consequences of damage in terms of life and property. All critical elements of the dam whose failure might result in an uncontrolled release of the reservoir must be designed to resist the MDE. According to ADNR (2005), the MDE may be defined based on either deterministic or probabilistic evaluations, or both. The return period selected for the MDE should be selected in direct correlation with the magnitude of the MCE.

Seismic Hazard Analyses and Selection of MCE/MDE: BGC (2011a) analyzed the seismic hazards at the site using both deterministic and probabilistic methods. A deterministic method involves identifying the nearest active fault, and the largest earthquake that has happened on that fault during the available historical and geologic record. An assumption is made that the largest earthquake will happen at the closest point on the fault to the site. From this event, the potential ground motion at the site is calculated. The advantages of this method are that it is relatively easy to do, and provides both a realistic and conservative result. The disadvantages are that it does not take into account potentially unknown faults or events.

The probabilistic method starts by defining a source zone in which earthquakes happen, and assumes that earthquakes have an equal probability of occurring at any spot in this zone. Historic and geologic information is used to generate a list of earthquakes that have occurred in the zone to evaluate the probability of an earthquake of a given magnitude occurring in the zone in some future period of time. Hazard values are then calculated under a variety of different scenarios to generate a hazard curve, which plots the decreasing probability of larger magnitude earthquakes. Probabilistic methods can be viewed as inclusive of all deterministic events with a finite probability of occurrence.

These two methods provide complimentary results. Deterministic events can be checked with a probabilistic analysis to ensure that an event is realistic (and reasonably probable), and probabilistic analyses can be checked with deterministic events to see that rational, realistic hypotheses of concern have been included in the analyses.

As noted in the Corps' Dam Safety Assurance Program ER-1110-2-1155, September 1997, Class I dams are required to survive and remain safe during and following an MCE event. The emphasis of the analyses by BGC (2011a) was therefore to identify the ground motion parameters for design to the MCE.

The TSF dam would be designed to withstand a magnitude 7.8 earthquake and maximum ground shaking (or horizontal peak ground acceleration [pga]) of 0.36 to 0.44g<sup>7</sup> based on both deterministic and probabilistic methods of analysis. The deterministic analysis indicated that the MCE would cause ground shaking of 0.36g from a magnitude 7.8 earthquake with a 5,000-year return period, which would be generated at a shallow crustal depth occurring on the Iditarod-Nixon Fork Fault (see Section 3.3.2.1). The probabilistic approach, based on USGS data and maps (Wesson et al. 2007; Petersen et al. 2008), considered earthquakes from multiple sources, and yielded a pga of 0.44g for a 1 in 10,000-year earthquake (BGC 2011a). The probabilistic scenario considered earthquakes from deep distant subduction zone events, as

---

<sup>7</sup> Ground shaking is typically characterized by the largest initial seismic wave hitting a structure, which is measured in terms of acceleration (change in speed of the ground) and in units expressed as a fraction of gravity (g).

well as both recognized and unknown shallow crustal sources. An MCE of magnitude 7.3 was indicated for this scenario based on de-aggregated USGS data (BGC 2011l).

Thus, the recommended MDE for the tailings dam design is characterized by  $p_{gas}$  from both the deterministic and probabilistic approaches, which were used in different methods of seismic deformation analysis described below. Both earthquake scenarios were used in pseudostatic<sup>8</sup> deformation analyses, and the deterministic earthquake was used in a numerical model of seismic deformation, using the FLAC 5.00 and UBCHyst models. These approaches are consistent with relevant guidance and regulations; however, final acceptance of the results would be made by ADNR as part of dam safety permitting. The detailed assumption, resolution and design conditions applied in the seismic deformation analysis are presented in the Donlin Creek Gold Project Feasibility Study Update (BGC 2011a).

**Seismic Deformation Analyses:** The results of numerical seismic deformation analyses performed by BGC (2011a), in which a model of the dam was subjected to ground shaking based on time histories of earthquakes comparable to the MDE, indicate that maximum permanent displacement of the dam in the event of a major earthquake would be at the downstream edge of the dam crest, which is predicted to experience about four feet of vertical settlement and about 16 feet of horizontal movement in a downstream direction. The upstream edge of the crest would experience about one foot each settlement and horizontal movement (Figure 3.3-5). Results of deformation analyses conducted on abutment sections of the dam yielded similar results. Displacements are towards the downstream direction due to the presence of the tailings load at the upstream slope and the relatively steeper downstream face. The horizontal displacements are interpreted as skin failure on the downstream face, and would not compromise the integrity of the dam crest or filter zones.

Pseudostatic displacement calculations were performed based on both deterministic (5,000-year) and probabilistic (10,000-year) earthquakes using two established methods. The results predicted less deformation than the numerical analysis: in the range of 0.1- to 0.5-foot for mean horizontal displacement, and 1.1 to 1.4 feet for crest settlement for both MCE scenarios. Similar calculations for the starter dam yielded horizontal displacements of 0.07 to 0.4 feet, and crest settlement of 0.4 to 0.5 feet. The difference in predicted deformation between the deterministic and probabilistic earthquakes in this analysis was very small, on the order of a few inches; for example, the deterministic earthquake exhibited 1.1 feet of dam crest settlement, as compared to 1.4 feet for the probabilistic earthquake (BGC 2011a).

The TSF dam exhibited a pseudostatic factor of safety<sup>9</sup> (FOS) of 1.11 for the MDE  $p_{ga}$  (5,000-year event). Seed (1979) developed a pseudostatic slope stability method for dams with materials that do not undergo severe strength loss and have crest accelerations less than 0.75 g; performance is typically judged acceptable if  $FOS > 1.15$ . The result for the TSF pseudostatic test was marginally below this number based on the maximum crest acceleration obtained for the design  $p_{ga}$  from Harder (1991). However, the foundation and compacted dam materials are not expected to soften due to cyclic loading. Because of this, the FOS of 1.11 is considered acceptable to meet the Seed (1979) criteria (BGC 2011a).

---

<sup>8</sup> Pseudostatic stability analysis adds a lateral earthquake force to a static stability analysis, which evaluates stability under routine loading conditions (e.g., one involving the weight of earth and porewater above a sliding plane).

<sup>9</sup> Factor of safety is the ratio between forces resisting movement and forces driving movement. An FOS greater than 1.0 indicates stable conditions, and less than 1.0 unstable conditions. Different target FOSs may be proposed for different structures depending on risk to safety or interruption of operations; i.e., a target FOS greater than 1.0 adds a margin of safety to the minimum needed to support the design load.

The difference between the numerical and pseudostatic results is primarily attributable to different ground shaking amplification from the dam base to crest used in the two methods. The pseudostatic calculations are generally considered more realistic as they are based on an empirical relationship derived from actual field observations of earthquake damage to dams (BGC 2011a). Regardless, the extent of impacts to the dam under the more conservative (numerical modeling) approach (Figure 3.3-5) is expected to be local, limited to a small amount of settlement at the crest of the dam, which would be within the operating freeboard of about seven feet, allowing for an adequate safety margin against overtopping (BGC 2011a). A minor amount of damage could occur to the downstream edge of the dam crest which would be temporary and repairable, and the dam would continue to function.

**Dam Construction Features:** The proposed TSF would be constructed in the Anaconda Creek basin as a fully lined impoundment with a cross-valley dam located on the west side of the valley using compacted rockfill. Build-out dimensions for the TSF dam are provided in Table 3.3-2. The compacted rockfill design yields FOS under seismic loading conditions within established design criteria (i.e., exceeds recommended FOS of 1.5) (BGC 2011a). Additional discussion of static stability analysis of the dam is presented under Section 3.3.3.2.2. Design elements of the tailings dam and impoundment that serve to minimize impacts during earthquake ground shaking are described below and shown on Figure 3.3-6 and Figure 3.3-7.

- As described above, the dam would be designed to withstand ground shaking from a magnitude 7.8 earthquake on the Iditarod-Nixon Fork fault, as well as the 1-in-10,000 year ground motion event from multiple seismic sources.
- Prior to tailings dam construction, two temporary fresh water diversion dams (FWDDs), referred to as the North and South FWDDs, would be constructed upstream of the tailings impoundment in order to facilitate tailings dam and liner construction and limit the amount of surface water entering the TSF during Construction and early Operations phases. Diversion channels contoured along the slopes around the TSF area would also divert fresh water around the tailings dam site during construction (SRK 2017b). Seismic stability analyses of the FWDDs are described below under “Other Dams and Structures.”
- The TSF would be constructed as a valley dam, intentionally sited in a narrow part of the valley in order to make maximum use of natural geologic materials to provide containment. TSF dam construction would begin by clearing and stripping foundation area soils and overburden to bedrock, thereby removing ice-rich and permafrost-containing soils (described in Section 3.2, Soils). The purpose of this is to increase the strength and stability of the dam foundation by locating it directly on bedrock.
- Because the TSF dam would be constructed on bedrock, impacts from liquefaction during an earthquake are considered to be of low likelihood. The tailings themselves are likely to liquefy during ground shaking, but would be held by the dam. The effect of liquefied tailings on the upstream face of the dam was incorporated as a bounding pressure into the seismic deformation analysis of the dam described above (BGC 2011a).
- The TSF dam would be constructed of the following materials, in sequence from the downstream main body of the dam to its upstream face. The material would be obtained from rock quarried from the plant area, the open pit, or borrow sources adjacent to Crooked Creek. The material would be non-acid generating (NAG)

greywacke rock or terrace gravels sized to meet specifications as shown on Figure 3.3-7) (SRK 2014b; BGC 2011a).

- Rockfill shell (main body of dam): Rockfill consisting of gravel- to boulder-sized rock placed and compacted in 5-foot lifts;
  - Rockfill transition zone: A 20-foot thick zone consisting of gravel- to cobble-sized material compacted in 3-foot lifts to separate filter zones from the rockfill shell;
  - Filter zones: Two 10-foot thick filter zones (one coarse and one fine) placed and compacted in 1-foot lifts. The filter zone materials would be sized to prevent migration of tailings into the rockfill shell in the event that the liner is punctured. The filter thickness was selected to accommodate deformation under seismic loading (Figure 3.3-6);
  - Liner Bedding: Three feet of liner bedding consisting of terrace gravels and compacted in 1-foot lifts on top of the filter zone; and
  - Liner: A 60-mil (0.06-inch) linear low-density polyethylene (LLDPE) geomembrane liner placed on top of the liner bedding materials. LLDPE was selected over other liner materials for its cold-climate constructability, ultraviolet (UV) resistance, strength, puncture resistance, and strain tolerance. LLDPE can sustain high levels of elongation, be handled and seamed in subfreezing temperatures, and is commonly used for lining high-load mining facilities such as tailings impoundments.
- Downstream construction methods would be utilized to construct the tailings dam. The upstream face of the dam (inside the TSF) would be benched with slopes of 2H:1V (horizontal to vertical) to accommodate placement of the liner. The downstream slope would be constructed at 1.7H:1V based on results of stability modeling. The TSF dam would be constructed in stages. The first (starter) dam, constructed over a two-year period, would be 198 feet high in order to store the first year of tailings. After construction of the starter dam, the dam would be raised in the downstream direction in seven phases approximately every four years throughout the operating mine life. Thus, the TSF dam height will increase from the starter height to the final (ultimate) height of 471 feet at Year 25, and have a maximum length of approximately 6,000 feet, a crest width of 100 feet, and maximum width at the base of approximately 1,800 feet at Year 25. The rate and size of the raises are designed so that the dam has sufficient height to store the amount of tailings produced over the period, operating pond volume, flood storage, and freeboard requirement. There would be 20-foot wide benches between each dam raise.
  - Construction of the TSF impoundment (basin) would begin with excavation of the foundation soils to an average depth of 3 feet at the valley bottom and 1.5 feet in mid-slope areas in order to remove the majority of organics and some ice-rich permafrost soils (Section 3.2, Soils). These soils would be retained for later use in reclamation.
  - A geotextile-wrapped rockfill underdrain, capable of handling the baseflow through the Anaconda Creek valley, would be placed along the main creek beds and key tributaries. Three feet of bedding material would be placed on top of the undrain system and cleared impoundment foundation followed by placement of a 60-mi LLDPE geomembrane liner.

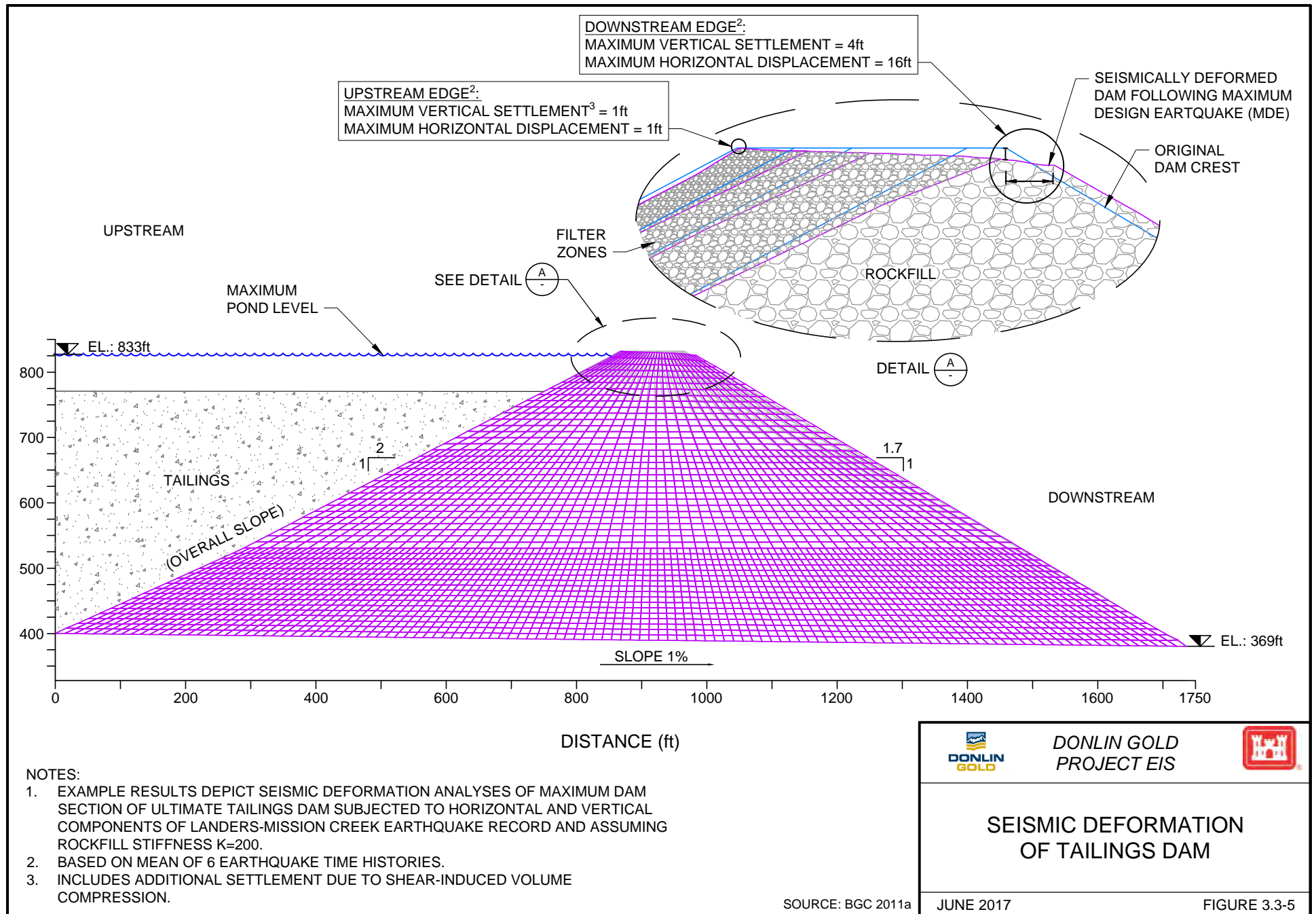
- A TSF seepage recovery system (SRS) consisting of a monitoring pond, diversion ditches, seepage recovery wells, and pump-back system would be constructed immediately downstream of the TSF dam. The SRS would capture underdrain water and incidental seepage through the TSF liner. Additional discussion of dam seepage analysis is presented under Section 3.3.3.2.3.

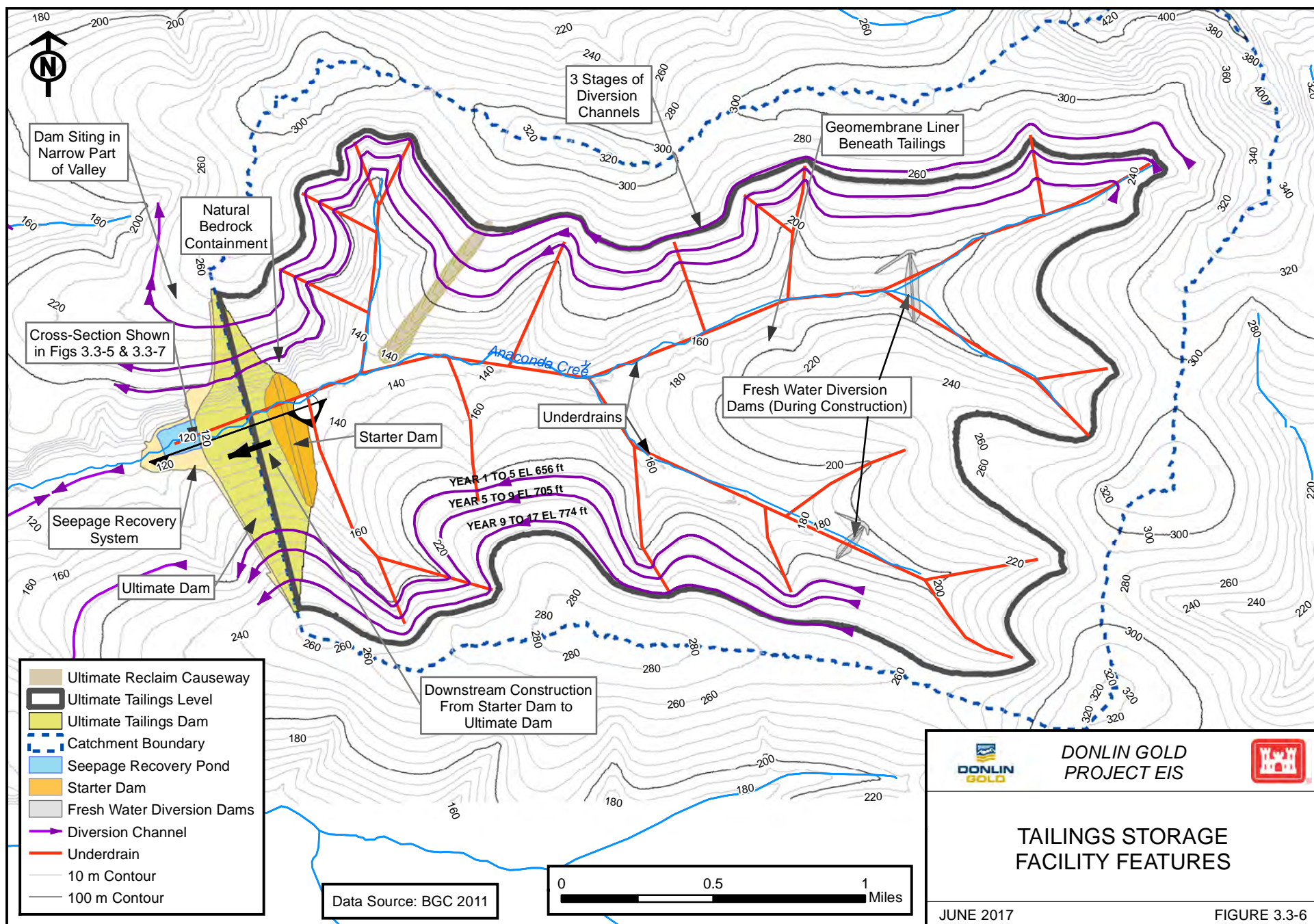
Construction Quality Assurance/Quality Control (QA/QC): A Construction QA/QC Plan would be developed by Donlin Gold to assure that the TSF dam is built in accordance with approved specifications. The plan would specify actions for approving dam materials, construction methodology, field testing, surveying, monitoring, and documentation (SRK 2014b). ADNR (2005) guidelines provide details on plan requirements, personnel responsible for QA/QC, key inspection items, and required post-construction document submittals. Required submittals would include an Emergency Action Plan (EAP) that includes a dam break analysis with potential inundation maps, and describes actions to be taken in the event of dam failure.

Examples of Dam Performance in Earthquakes: Several examples of dam performance during earthquakes may be pertinent to the proposed Donlin Gold design. The Zipingpu rockfill dam in China experienced the 8.0 magnitude Wenchuan earthquake in 2008, the epicenter of which was located about 10 miles from the dam (slightly larger earthquake, similar distance as design event for TSF dam). Several types of structural damage occurred, but no catastrophic failure or containment loss (Lekkas 2008). Damage included about one to two feet of crown subsidence, deformation of the lower face across an area of about 10,000 square feet (about 1 percent of total dam face), and landslide damage to abutments. Following the magnitude 7.9 Denali earthquake of 2002, no damage was observed at the Fort Knox tailings dam (rockfill), located near Fairbanks, Alaska about 100 miles north of the epicenter (ADNR 2007). Effects at the Torata rockfill dam at the Cuajone Mine in southern Peru following the 8.4 magnitude Peruvian Earthquake in 2001 (epicenter located about 150 miles west of mine) included minor cracking near one abutment and densification cracking in uncompacted downstream rockfill (Rodriguez-Marek et al. 2001).

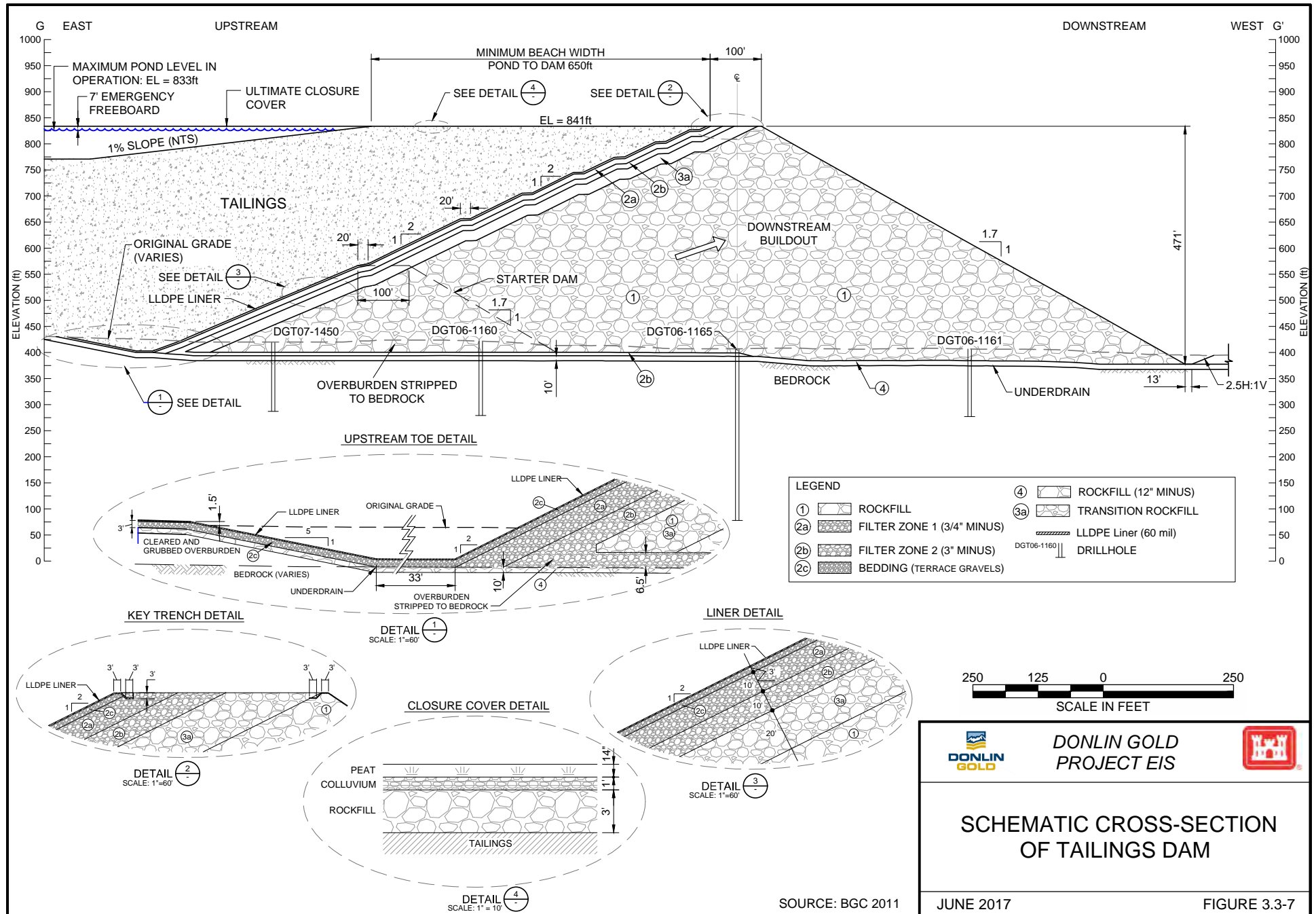
Summary of TSF Dam Seismic Stability: The TSF dam design is considered a robust tailings dam. Downstream construction is inherently the most stable construction process, and the proposed slopes and rock zones are considered safe. Seismic parameters incorporated into the design, as well as dam performance examples worldwide, indicate that the dam would be extremely unlikely to fail during the largest earthquake that is considered probable in the area, and would very likely remain functional and easily repairable. Thus, the intensity of impacts on the dam during an earthquake are expected to range from immeasurable to noticeable changes, but the design would resist the range of seismic hazards that would be expected to occur (Table 3.1-2 in Section 3.1, Geology). Valley siting, dam foundation preparation, water control structures, downstream construction method, rockfill body, filter zones, and liner materials all contribute to dam stability in the event of an earthquake.











### Waste Rock Facility (WRF)

The proposed WRF would be located in the American Creek Valley southeast of the pit and constructed in 100-foot lifts. The planned maximum dump height is approximately 1,115 feet from the toe, with the elevation of the top lift at approximately 1,700 feet (SRK 2016d). The WRF would include up to 8 percent by volume of overburden materials that could weaken the strength of the stacked and compacted waste rock. The stability of the WRF was evaluated under a major earthquake scenario using both pseudostatic and dynamic methods. The MCE used in the analysis was a 7.8 magnitude earthquake with a pga of 0.4g, which represents a return period in the range of 5,000 to 10,000 years.

The dynamic analysis indicated permanent displacements of up to 6 feet in the lower lifts, which exceeds a target allowable permanent deformation of the waste dump of 1 to 2 percent of the dump height (about 3 feet). To reduce deformation to within target levels, ice-rich loess soils along the toe of the WRF would be stripped to an average depth of 8 feet to secure the leading face of the WRF, and replaced with coarse, durable waste rock. The results of the pseudostatic analysis with this foundation preparation indicate that the stability of the proposed WRF exceed design criteria (FOS of 1.0) with estimated FOSs between 1.07 and 1.40, with the lowest values in the southwest corner near the lower Contact Water Dam (CWD) (BGC 2011b) (Figure 3.3-8). Because these analyses assume that deeper ice-rich soils beneath the depth of proposed foundation preparation do not exist and would not liquefy, the possibility exists for more marginally stable or unstable conditions in the western part of the WRF.

Diligent implementation of the proposed mitigation is crucial during the early phases of construction of the WRF to minimize potential instability resulting from a seismic event. Impacts from a major earthquake on the WRF could range from deformation that affects just the toe of the WRF above or below the Lower CWD, to sliding that affects the integrity of the Lower CWD structure or causes overtopping at high water levels. Additional discussion of the effects of permafrost on foundation soils and additional mitigation recommendations that would reduce the likelihood of effects are provided in Section 3.2, Soils.

Water Dams: Other dams at the Mine Site include the Snow Gulch Fresh Water Dam (FWD); the temporary American FWDD (which would be in use during WRF construction and expansion only); the Lower and Upper CWDs; and the temporary North and South FWDDs (which would be in use during TSF construction only). Build-out dimensions for these are provided in Table 3.3-2.

The Snow Gulch FWD is likely to be permitted as a Class I (high) hazard dam and the rest as Class II (significant) hazard dams under ADNR (2005) guidelines (Cobb 2014). As described above under TSF, Class I dams must be designed to withstand earthquakes with a return period between 2,500 years and that of the MCE. Class II dams must be designed to withstand earthquakes with a return period between 1,000 and 2,500 years. The MDE selected for all of these dams (BGC 2011a, c) was the 2,475-year return period event, which is well within ADNR guidelines for Class II and equivalent to the lower end of Class I. A 2,475-year return period earthquake of Magnitude 6.7 to 7.3 and horizontal pga of 0.25g was selected as the MDE for the dams based on USGS seismic hazard maps (Wesson et al. 2007). Target seismic design criteria were considered to be less than 3.3 feet permanent displacement to maintain freeboard, and a pseudostatic (seismic) FOSs of 1.15 for dam side slopes. Results of pseudostatic loading analysis indicated that maximum seismic deformation of the dams in the event of the MDE would be less than 3.3 feet, and the selected dam side slopes meet the target FOS (BGC 2011a, c).



**Table 3.3-2: Build-out Dimensions for Proposed Water Dams and TSF Dam**

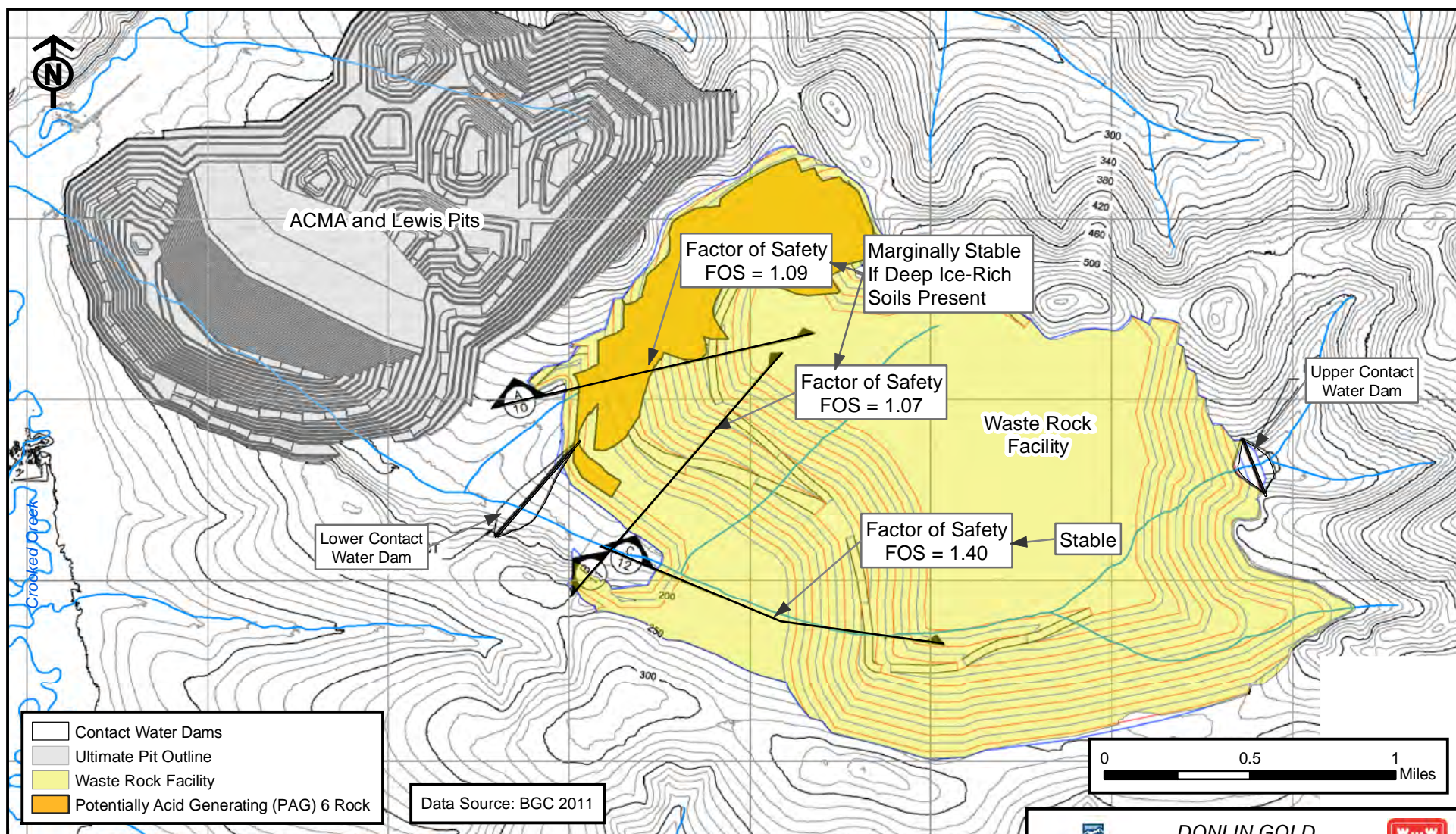
<b>Dam</b>	<b>Crest Elevation (ft)</b>	<b>Maximum Height (ft)</b>	<b>Maximum Length (ft)</b>	<b>Maximum Crest Width (ft)</b>	<b>Maximum Width at Base (ft)</b>	<b>Maximum Capacity (acre-ft)</b>
Snow Gulch FWD	756	151	1,270	33	490	3,240
American FWDD	761	95	1,170	33	380	867
Lower CWD	673	151	2,080	33	540	7,151
Upper CWD	1,114	193	1,210	33	450	3,240
North FWDD	660	74	1,170	33	280	478
South FWDD	660	66	800	33	280	211
TSF Dam (ultimate)	841	471	5,800	100	529	366,000

Sources: BGC 2011a, c; SRK 2016c, 2017b.

The dams would be constructed of compacted rockfill and lined with 60-mil LLDPE placed along the centerline of the dam in a zigzag pattern from base to crest. The zigzag liner would be protected on both sides by geotextile and gravel bedding, and would underlie much of the upstream half of the dam. Dam side slopes would be 2.1H:1V upstream and 1.7H:1V downstream. Site preparation for the dams would include stripping of overburden to bedrock. Liquefaction potential for the dams is considered to be low because of the bedrock foundation.

Overburden Stockpiles: Stability analyses of several lifts within the north and south overburden (NOB and SOB) stockpiles were conducted by BGC (2011b) for pseudostatic seismic conditions using a design criteria FOS of 1.0, and 1 in 475-year ground shaking event with a horizontal pga of 0.11. Temporary stockpiles not critical to mine operation are usually designed based on a lower level earthquake, in that they could sustain operationally repairable damage with an acceptable failure risk tolerance (BGC 2011b). The analyses assumed that the toe of lifts within the NOB stockpile would be set back a nominal distance from the upslope limit of each lower lift, and that ice-rich permafrost present at the SOB stockpile would be stripped to a depth of about 8 feet between lifts to provide good contact with more competent materials. The results yielded FOS in the range of 1.07 to 1.72, indicating that the stockpiles meet seismic design criteria under these conditions.

Other Structures: Seismic hazard and deformation analyses have not been conducted for several major structures at the mine, such as the power plant, fuel storage tanks, and process plant (SRK 2016a). However, these analyses are typically conducted during detailed design, and as such, are anticipated to be considered design features of the project. The likelihood of potential liquefaction from an earthquake event is considered low in shallow bedrock areas beneath the mill, power plant, and fuel storage areas, and moderate along roads constructed in lowland areas on unconsolidated materials, such as those crossing Omega Gulch and Crooked Creek.



Notes:

1. Factor of Safety (FOS): < 1.0 Unstable, = 1.0 Stable, > 1.0 Stable with a Safety Factor
2. Results of pseudo-static analysis based on 7.8 magnitude earthquake with a 0.4g horizontal peak ground acceleration (5,000 to 10,000 year return period).
3. Foundation preparation consists of the excavation of ice-rich loess to 8 ft and backfill with coarse waste rock.



DONLIN GOLD  
PROJECT EIS



## SEISMIC STABILITY OF ULTIMATE WASTE ROCK FACILITY

JUNE 2017

FIGURE 3.3-8



### *Operations*

TSF, WRF, and Other Dams: Impacts of ground shaking and liquefaction from a seismic event would be the same as described above under Construction on the partially and fully constructed TSF dam and WRF that are built out in raises and lifts during Operations, as well as the water dams that remain during Operations (Snow Gulch FWD and Lower and Upper CWDs). The temporary FWDDs would be removed during Operations, thereby eliminating the risk of seismic hazards to these structures.

Plans and monitoring programs for the TSF and other dams during Operations, that are considered part of the project under Alternative 2, would include the following in accordance with ADNR (2005) guidelines (SRK 2014b):

- An O&M Manual describing procedures for O&M under normal and extreme water levels, as well as for monitoring and inspections described below;
- Monitoring to confirm that dams are performing in accordance with design and to provide timely notice of any adverse changes that require attention;
- Four types of inspection: construction, routine, extraordinary, and Periodic Safety Inspections (PSIs). An earthquake occurring during Operations is an example of an event that would require an extraordinary inspection. PSIs are required at least every three years for Class I and II dams (ADNR 2005);
- An operator training program to provide personnel with proper expertise for safe operation of the dam;
- An EAP that describes actions to be taken in the event of dam failure;
- Permit reviews and updates that occur following each PSI; and
- Mitigation to conduct routine repairs (such as surface erosion or ice effects), as well as additional analyses or structural modifications that may be necessary if the dam is not performing as designed. Examples might include additional seismic modeling or modification of slope angles or lift heights.

In addition to the above, the following monitoring program is specified in the Donlin Gold Tailings Management Plan and Integrated Waste Management Monitoring Plan (SRK 2016c,e) for the TSF dam during the Operations phase:

- Daily inspection of tailings dam slopes and crest for observations of possible settlement, heaving, deflections, and lateral movement;
- Weekly and monthly monitoring of piezometers and embankment settlement monuments for deformation;
- Annual facility inspections of exposed earthwork, concrete, and structural steelwork; and
- Annual review and updates of the O&M manual and EAP, if necessary.

The WRF would be monitored quarterly during operations for physical stability (e.g., evidence of differential settlement, frost or tension cracks) (SRK 2016c, d). If design recommendations during the early phases of construction of the WRF were not implemented, the likelihood of waste materials mobilizing during a seismic event would increase during operations.

Concurrent reclamation activities would be conducted during operations at the WRF (SRK 2016a), which would add stability to this structure and help reduce seismic risk.

Occurrence of a major earthquake during Operations would require complete inspection for any impacts, as well as substantial follow-on adherence to design criteria for all major structures at the Mine Site to ensure that they continue to perform as designed. ADNR defines an earthquake worthy of additional inspections as shown in Table 3.3-3. Earthquakes of these magnitudes and distances are expected to produce a vertical motion of at least 0.1g at the Mine Site.

Based on the results of seismic analyses discussed above under Construction, minor damage may be sustained at the WRF, TSF, and other dams following a large earthquake that are within allowable design criteria for each structure. Repairs would be conducted as necessary to address displacement and settlement following such an event.

**Table 3.3-3: Earthquake Magnitude-Distance Criteria for Dam Incident Reporting**

<b>Magnitude</b>	<b>Distance to Epicenter (miles)</b>
5.0	15
5.2	17
5.4	19
5.6	21
5.8	22
6.0	24
6.2	27
6.4	29
6.6	32
6.8	34
7.0	37
7.2	40
7.4	43
7.6	47
7.8	50
8.0	53
8.2	57
8.4	62
>8.5	63

Source: ADNR 2005.

Open Pit: Stability analyses of slopes within the open pit were conducted by BGC (2007b) for pseudostatic seismic conditions under fully depressurized (dewatered) conditions based on an earthquake with a horizontal pga of 0.1 g, which is about a 1 in 500-year ground shaking event. Analyses of 9 cross-sections around the Lewis and ACMA pits yielded FOS in the range of 1.15 to 3.69, indicating stable slopes when dewatered for this level of earthquake. The analyses show the pit slopes are stable under dynamic conditions. The lowest results (least stable conditions) with FOS of 1.15 to 1.18 are expected for the south wall of the ACMA pit and northeast and south walls of the Lewis pit. Thus, the intensity of pit slope effects under earthquake loading would range from unmeasurable or unnoticeable changes, to noticeable changes with the



consideration that proposed active depressurization of the pit slopes throughout Operations is anticipated to be adequate to mitigate potential unstable conditions.

### *Closure*

The Donlin Gold Project would utilize a “design for closure” concept for reclamation and closure activities to address post-closure impacts at the Mine Site. Design criteria for major structures that would remain following closure and reclamation (e.g., TSF dam and WRF) incorporate appropriate seismic parameters that take into account the long-term condition of these facilities in post-Closure (i.e., MDE return periods of 5,000 to 10,000 years). Closure activities and seismic hazards pertinent to specific facilities are described below.

TSF: Tailings deposition at the TSF would be modified several years prior to the end of Operations to direct the saturated portion to the southeast corner of the facility. Closure of the TSF would require approximately five years to complete and would begin upon removal of water from the southeast corner through a reclaim pipeline. Capping of the TSF impoundment area would include waste rock overlain by overburden material, overlain by growth media. The downstream face of the dam would be flattened to a 2H:1V slope (SRK 2015g). The finished impoundment cover would be contoured to reduce infiltration and thus minimize the potential for mobilization of materials during an earthquake event.

The nature of the TSF material would change over time in closure. Porewater would be released (and pumped) from the tailings dam in response to loading by the closure cover, resulting in more dense but still saturated material. Laboratory tests indicate that this process would take approximately 50 years to occur after the end of mine life. Because the TSF will always contain saturated material, the potential for tailings to flow from the TSF under a dam failure event would remain after the dewatering period.

Monitoring in post-Closure would include inspections for mass stability and seepage flow monitoring on the following schedule:

- Mass Stability: Annually for the first five years, and then every five years after that until observations indicate a stable condition;
- Seepage Flow: Quarterly as part of water quality sampling until analyses indicate a stable condition (SRK 2016e); and
- Inspections after significant seismic events (ground shaking large enough to potentially affect dam stability, i.e., > 0.1g, Table 3.3-3).

Financial assurance for post-Closure monitoring and maintenance of the dam would be established in accordance with ADNR dam safety regulations (Donlin Gold 2015).

WRF: The WRF has been designed to maximize concurrent reclamation during Operations. At closure, slopes would be contoured to less than 3H:1V, and final grading would minimize ponding and infiltration, thereby decreasing the potential for liquefaction and mobilization of materials during a seismic event.

Water Dams: The water dams remaining at closure (Snow Gulch and Upper and Lower CWDs) would be removed, eliminating seismic hazard risk to these structures in post-Closure. The dams would be breached and regraded to match natural surroundings. As the Upper CWD lies within the footprint of the WRF, following dam breaching, its impoundment area would eventually be filled with waste rock and become part of the WRF.

Open Pit: During closure, waste rock would be used to backfill portions of the pit (SRK 2016e), which would act as a partial buttress to seismically induced highwall instability. Cessation of pit dewatering activities would commence following backfilling activities, and pre-construction static groundwater conditions would eventually be restored. Water table recovery in conjunction with ground shaking from an earthquake event could impact closure activities or the stability of pit walls in post-Closure.

Pseudostatic stability analyses (earthquake loading) of pit slopes in the post-Closure period were performed by BGC (2014j) under both partially full and full lake scenarios for four highwall cross-sections. The analyses used 50 percent of the 1 in 475-year return period earthquake (pga of 0.05g), and target FOS of 1.0 to 1.2, which are typically considered adequate for short-term loading of closed pit walls in an earthquake. The results indicated unstable conditions (FOS of 1.0) for the south-southwest ACMA pit wall along the current trace of American Creek with a partially full lake, and marginally stable conditions (FOS of 1.08) with a full lake, likely due to highly fractured and faulted bedrock in this area (Figure 3.3-9). The northwest wall, south corner of the ACMA wall, and southeast Lewis pit wall (adjacent to WRF) were all stable under these conditions. The northwest wall adjacent to Crooked Creek had the highest FOS (most stability) of the four cross-sections, 2.35 to 3.04 for the partially full and full lake scenarios, respectively.

Effects of failure of the south-southwest pit wall into the pit lake could include disruption of lake stratification (Section 3.7, Water Quality) and damage to outfall piping structures leading to the lake. Changes in water chemistry due to lake stratification disruption are expected to be manageable through monitoring and alterations in water treatment if necessary. Any drainage from piping damage would continue to flow towards the lake until repairs could be made, and would not discharge to Crooked Creek. Wall collapse in this area would not impact the WTP or fuel storage areas located along the ridge southeast of American Creek. Thus, effects resulting from a potential failure of this pit wall would be manageable within a short timeframe.

There is a low probability, however, that a larger earthquake with a longer return period could result in wall failures not covered by the BGC (2014j) analysis. The probability could be considered somewhat equivalent to the return period used in the analysis (roughly 1 in 250). Effects of highwall failure into the pit lake could range from manageable impacts such as disruption of lake stratification described above, to displacement of the WRF or capturing of Crooked Creek. Recommendations are provided in Chapter 5, Impact Avoidance, Minimization, and Mitigation, for possible mitigation to address and lower this risk.

### *Summary of Mine Site Impacts*

In terms of intensity, direct effects in the event of a major earthquake at the mine site would vary considerably. Changes in the resource character may not be measurable (e.g., liquefaction or ground shaking from ~~form~~ small earthquakes). Noticeable changes in resource character could occur, but the design is expected to be adequate for the range of geohazards conditions. Resource characteristics at two specific locations (WRF lower lifts if deep ice-rich soils are present, and pit walls in post-Closure) could cause acute or obvious effects on the stability of these structures, which could be reduced with additional mitigation. The duration of impacts would also vary depending on the likelihood or type of effects on dam stability. In cases of minor damage that is easily repairable, geohazard impacts would be infrequent but not longer than the life of the mine for structures that would be removed at Closure (e.g., water dams) and

resource conditions would be expected to return to pre-activity levels at the completion of the activity; but impacts could occur throughout post-Closure for structures that would remain in place (e.g., TSF, WRF). In cases of potential pit wall failure, resources would not be anticipated to return to previous levels even if actions that caused the impacts were to cease. The extent or scope would mostly remain within the footprint of the Mine Site. In terms of context, seismic hazards are considered a usual or ordinary geologic phenomenon in Alaska, which are governed by regulation for certain structures (e.g., dams).

## Transportation Corridor

### *Construction and Operations*

Disturbance from ground shaking during earthquake events could affect several Transportation Corridor facilities during Construction and Operations, including the proposed mine access road, airstrip, permanent camp, and port sites. Ground shaking can cause damage to bridges, slope instability of cut-and-fill road sections, differential settlement of graded fill areas, and damage to built structures such as docks and storage tanks. Liquefaction and related lateral spreading and cracking could potentially occur in low-lying areas such as along Jungjuk Creek or the Bethel Port, although gravel fill would likely mitigate these effects. Any actions that would occur at Dutch Harbor or the Port of Bethel at the Bethel Yard Dock are not part of the proposed action, and are considered connected actions (see Section 1.2.1, Connected Actions, in Chapter 1, Project Introduction and Purpose and Need).

The constructed gravel road would have appropriate load ratings, limited elevated fill sections, and design configurations to withstand moderate to substantial seismic events. Additional engineering associated with the six bridges along the road would take place during the final engineering phase (e.g., seismic design). The ADOT&PF utilizes seismic bridge design protocols based on AASHTO guide specifications (FHWA 2011a; UAF-INE 2014). These analyses are typically conducted during the detailed design stage, and are anticipated to be incorporated into final design of the mine access road bridges.

As shown on Figure 3.3-2, ground shaking predictions for the Dutch Harbor area in the event of a large earthquake are higher than at the Mine Site and Kuskokwim River corridor. Damage to docks and fuel storage tanks during large earthquakes could occur through earth settlement, liquefaction, lateral spreading, tank bulging, buckling, and sloshing damage to tank roofs (e.g., Yokel and Mathey 1992; Eberling and Morrison 1993). Potential spill effects from a ruptured fuel tank are discussed in Section 3.24, Spill Risk. The duration of seismic hazard effects on these structures would depend on the length of time needed for repairs, if any. Mitigation against liquefaction and settlement effects on tanks can be partly handled through siting in upland locations, which is the case at the Angyaruaq (Jungjuk) Port. Seismic design of storage tanks, which is typically based on national engineering standards and guidance (e.g., API 2013), has not been conducted to date, but is anticipated to be incorporated into final design.

As presented in Section 3.3.2.2.1, there are no recognized surface faults along the mine access road or at the Angyaruaq (Jungjuk) and Bethel ports. Quaternary surface faults with potential evidence of activity identified in the Dutch Harbor area indicate a low probability of surface rupture at this expanded fuel storage facility, and could threaten the integrity of storage tanks and other infrastructure. This risk of surface fault rupture, and an estimate of vertical or lateral

displacement associated with it, are typically evaluated in final design, and as such, are anticipated to be considered design features of the project.

While not necessarily governed by specific regulations, seismic design of roads, ports, and storage tanks are typically addressed through national standards and guides that are adopted by federal and state government agencies.

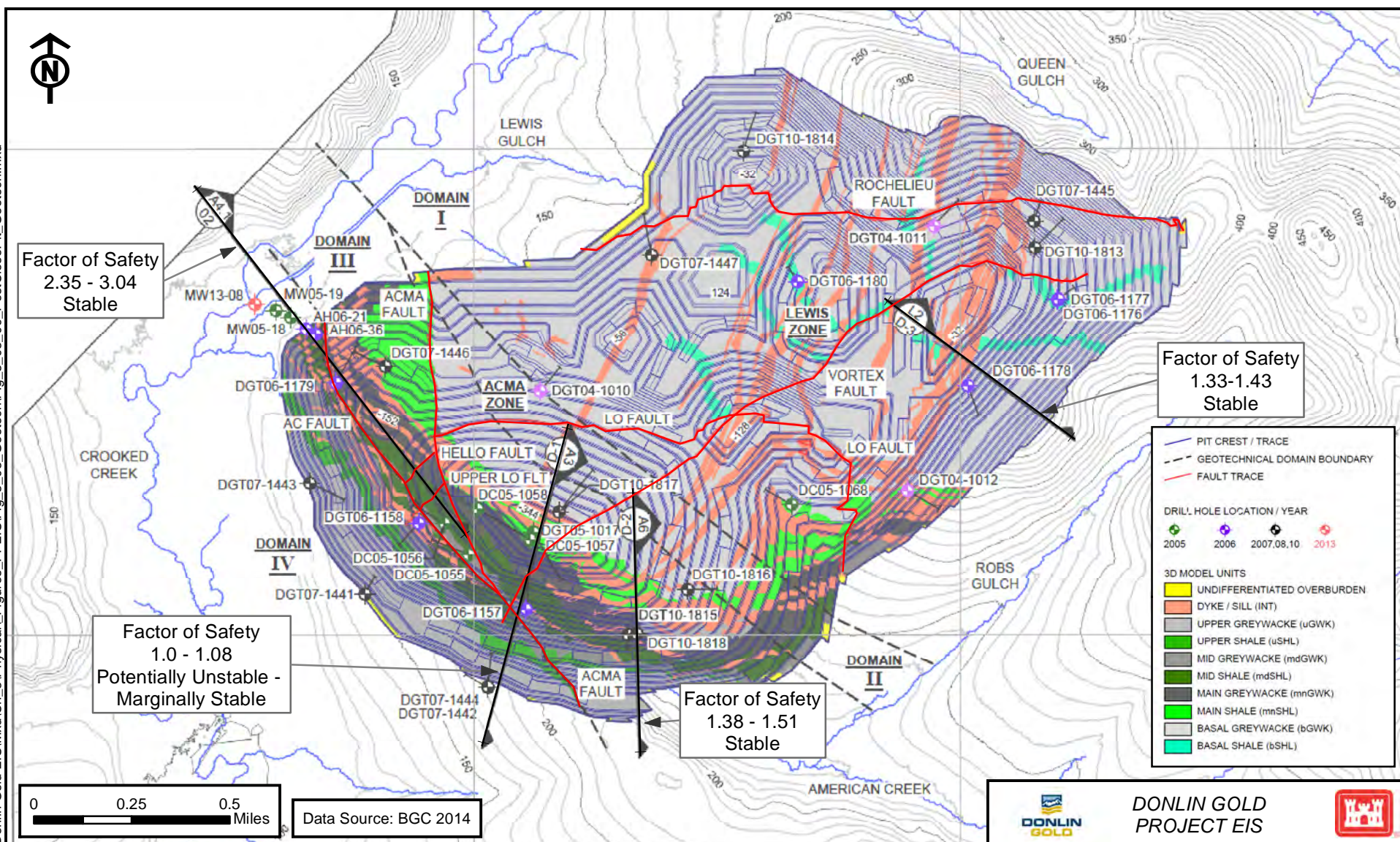
### *Closure*

Impacts during Closure and post-Closure would be the same as described above for those facilities that remain in service to support water treatment at the mine (access road, airstrip, and permanent camp) or for facilities that continue as third-party operations (Bethel and Dutch Harbor ports). The Angyaruaq (Jungjuk) Port site would be decommissioned and reclaimed during Closure, eliminating seismic hazard risk to the dock and storage tanks at this site in post-Closure. Any actions that would occur at Dutch Harbor or the Port of Bethel at the Bethel Yard Dock are not part of the proposed action, and are considered connected actions (see Section 1.2.1, Connected Actions, in Chapter 1, Project Introduction and Purpose and Need).

### *Summary of Transportation Corridor Impacts*

In terms of intensity, direct impacts from an earthquake on Transportation Corridor facilities would vary depending on the type of structure. In most cases, impacts from geohazards or changes in resource character may not be measurable or noticeable. Noticeable effects could occur, but the design is expected to be adequate for the range of geohazard conditions. For structures where site-specific seismic design has not been conducted yet (e.g., bridges, storage tanks, sheet pile docks), it is reasonable to assume that this would occur during final engineering, which would mitigate the intensity of impacts. The duration of seismic hazard effects on these structures would be mostly intermittent and could occur throughout the life of the mine. The duration of impacts would depend on the length of time needed for repairs, if any. Resource conditions would be expected to return to pre-activity levels at the completion of the activity for structures that would be removed at Closure, and continue throughout post-Closure for components that would remain (e.g., mine access road). The extent or scope of effects would remain within the immediate vicinity of the various facility footprints. In terms of context, seismic hazards are considered a usual or ordinary geologic phenomenon in Alaska, that are typically addressed through national standards and guides adopted by federal and state government agencies for certain structures (e.g., bridges, tanks, docks).





**Notes:**

1. Factors of Safety (FOS) under full and partially full pit lake conditions:  $\leq 1.0$  Unstable,  $> 1.0$  Stable
2. Results of pseudo-static analysis based on 50% of 475 year return period earthquake at 0.05g horizontal peak ground acceleration.



**DONLIN GOLD  
PROJECT EIS**



**SEISMIC STABILITY OF PIT WALL  
IN POST-CLOSURE PERIOD**

JUNE 2017

FIGURE 3.3-9

## Pipeline

### *Construction and Operations*

Effects of a major earthquake on the natural gas pipeline during Construction and Operations phases could include damage to both on- and to off-ROW facilities. Earthquake effects on pipelines are typically associated with ground displacement (such as surface fault rupture, earthquake-induced landslides, and liquefaction), as ground shaking alone causes pipelines to move closely with the ground around them causing little damage (Lanzano et al. 2013). Two seismic design levels have been established for the pipeline corridor: one with a high likelihood of occurrence, called the operating-level earthquake (OLE), during which ground movement would not affect the operation of the pipeline; and one for a large unlikely event, called the contingency-level event (CLE), during which some damage and disruption of gas supply could occur, but would be reparable within a short time frame (CH2MHill 2011b).

Potential surface rupture from a major earthquake event at the Castle Mountain and Denali fault crossings could cause pipeline failure if not designed adequately for permanent lateral ground displacement. The above ground design at these locations (CH2MHill 2011a, b; SRK 2013b) follow accepted industry guidelines (Honegger and Nyman 2004), which have proven effective for large surface fault ruptures, such as the 18-foot lateral displacement experienced by the TAPS during the Magnitude 7.9 Denali Earthquake of 2002 (Hall et al. 2003).

The pipeline would be designed to accommodate displacement on the Castle Mountain fault of five feet of horizontal offset and two feet of vertical offset for the recommended CLE of magnitude 6.8. The pipeline crosses the Denali fault at two locations: MP 148.5 and MP 180.7 (Figure 2.3-33, Chapter 2, Alternatives). Design displacement for the recommended CLE of magnitude 7.1 on the Denali Fault would be 5 feet horizontal (14 feet including one standard deviation) and 2 feet vertical (north-side-up) for both crossings. The length of special design required across the zone of possible distortion on the Castle Mountain fault is estimated to be about 300 feet wide, and on the Denali Fault about 1,000 feet wide. Preliminary design of the crossings would include transitions to the above-ground sections, which would approach the faults at sub-perpendicular angles, and "Z"-segment configurations at each end of the potential movement zones to accommodate lateral stress on the pipe in the event of surface fault rupture (CH2MHill 2011b; SRK 2013b).

As described in Section 3.3.2.3.1, Pipeline, Liquefaction Potential, a major earthquake could cause liquefaction in unfrozen lowland areas with fine sandy soils. Porewater pressure increases during liquefaction can cause buried pipelines to become buoyant, which if not properly accounted for in design, could lead to pipe floatation and possible damage. The loss of soil shear strength can lead to permanent ground movements through downslope lateral spreading, flow failure, and settlement. Liquefaction potential during the CLE is considered moderate to high in the Cook Inlet region of the pipeline, and could also occur in some low-gradient drainages in the Kuskokwim Hills and along the north flank of the Alaska Range. Liquefaction is not expected to occur through the Alaska Range due to deeper groundwater and dense coarse soils (CH2MHill 2011b; BGC 2013b). Estimated lateral spreading effects range from less than 0.2 feet for pipeline sections north of the Alaska Range, to more than two feet in lower Cook Inlet (CH2MHill 2011b). Control measures for buoyancy (e.g., select compacted backfill, increased cover depth, increased pipe wall thickness, swamp weights) would be considered in certain



high liquefaction areas so that design deflection and stress on the pipe are not exceeded (CH2MHill 2011b).

Earthquakes can trigger landslides where a combination of steep slopes and loading from ground shaking exceed soil strength. The effects of earthquake-induced landslides on the pipeline would be similar to those described under Section 3.3.3.2.2 (Slope Stability, Pipeline).

Pipeline design is governed by USDOT PHMSA regulations. As described in Section 3.2.3.2.2 (Soils, Permafrost), the proposed pipeline would likely require a Special Permit for strain-based design (SBD). Special Permit conditions that are considered part of the proposed project (Appendix E) contain provisions for identifying geohazards (including seismic effects), design specifications, and inspections to ensure that geotechnical limits for SBD are not exceeded.

A major earthquake could also cause impacts to off-ROW facilities, pipeline appurtenant structures (e.g., valve stations), and temporary facilities such as access roads, airstrips, construction camps, pipe storage yards, barge landing sites, and material sites. Typical effects could include damage to temporary bridges, slope instability of cut and fill ROW and road sections, differential settlement of graded fill areas, lateral spreading and surface cracking in low-lying liquefaction-prone areas, cracking of concrete foundation pads, and damage to built structures such as the compressor station. Constructed roads would have appropriate load ratings, limited elevated fill sections, and design configurations to withstand moderate to substantial seismic events. Material sites and stream crossings could experience temporary sloughing or slope failure during a major earthquake. Most pipeline facilities, roads, and airstrips would be constructed on engineered fill placed either on bedrock or unconsolidated deposits, thereby reducing the potential for liquefaction and lateral spreading.

While compressor stations have typically performed well in major earthquakes worldwide, damage can occur from tipping and sliding of equipment and lack of anchorage (Yokel and Mathey 1992). The compressor station would be built in accordance with national codes and standards (SRK 2013b) such as those of the ASCE (ASCE 2006), which contain provisions for seismic design of structures, minimizing the effects of seismic hazards on this structure.

Complete inspection of all pipeline features would typically occur after a major earthquake. Sizeable surface rupture and ground shaking would likely warrant additional geotechnical engineering data collection to identify seismic damage, modifications of design criteria that may be necessary, and substantial follow-on adherence to any redesign criteria resulting from the seismic behavior of structures and earthworks.

Because the pipeline design is expected to withstand the effects of a major earthquake without rupture and/or be reparable within a short timeframe, the intensity of impacts would range from unmeasurable or unnoticeable changes, to noticeable changes with the consideration that design is adequate for the expected range of geohazard conditions. Damage to appurtenant facilities through settlement, sloughing, and ground cracking are expected to be easily reparable within a temporary timeframe (days to months).

### *Closure*

Closure and reclamation activities would involve decommissioning of most access roads, airstrips, material sites, camps, and storage yards immediately following Construction, eliminating seismic risk to these facilities during the Operations phase. Likewise, seismic effects on the pipeline would be reduced or eliminated following in-place decommissioning of the

buried pipeline and removal of above ground facilities such as the fault crossings and other appurtenant structures during the closure phase. Reclamation of surface soils following facility removals would reduce sloughing effects during a major earthquake.

### *Summary of Pipeline Impacts*

In terms of intensity, direct effects in the event of a major earthquake along the pipeline corridor would vary. In most cases, impacts from geohazards or changes in resource character (e.g., liquefaction effects) may not be measurable or noticeable. Noticeable changes in resource character could occur, for example at pipeline fault crossings, but the design is expected to be adequate to withstand earthquake lateral displacement. While ground displacement at fault crossings in the event of surface fault rupture would not be anticipated to return to previous levels even if actions that caused the impacts were to cease, the majority of effects on the pipeline would be infrequent (e.g., damage reparable within days to months). The extent or scope of effects would mostly remain within the footprint of the pipeline ROW and associated facilities. In terms of context, seismic hazards are considered a usual or ordinary geologic phenomenon in Alaska, which are governed by regulation for pipelines.

#### 3.3.3.2.2 SLOPE STABILITY

##### Mine Site

As presented in Section 3.3.2.1.2, landslides are generally caused by natural processes like geomorphic landform development, heavy precipitation, and earthquakes, as well as man-made causes like construction of unstable earthworks or slope excavation. Avalanches and snow slides are another form of slope instability. The effects of these types of geohazards are described below for specific Mine Site facilities.

##### *Construction and Operations*

Open Pit: Potential slope instability in an open pit is a well-known concern, although such events typically cause little impact outside of pit operations, and can often be forecasted in advance through geotechnical monitoring. For example, two large rock avalanche landslides that occurred at the Bingham Canyon Mine in Utah in 2013, which is hosted in fractured intrusive and sedimentary rock similar to the Mine Site, were predicted and monitored weeks in advance so that employees and affected structures could be evacuated and relocated. Alaska's Fort Knox Mine, which has benched pit walls similar to the proposed pit, has experienced several small to medium rock slides extending across benches.

Faults and fractured bedrock at the Donlin Gold open pit are predicted to cause varying degrees of slope instability in the open pit as construction and pit development progresses. Numerous small and intermediate scale faults, large scale geological fold structures that define major changes in bedding orientation, and continuous and discontinuous fractures parallel to stratigraphic bedding planes are interpreted within and proximal to the pit area. Rock with lower quality shear strength associated with the southeast-trending AC and ACMA faults and northeast-trending intermediate faults, could have an impact on achieving design criteria for pit slopes (BGC 2011k, 2014d).

Geotechnical domains and sectors defined by these features have been established at the pit for the purpose of pit slope design (e.g., BGC 2011k). The results of slope stability analyses and



hydrogeologic modeling indicate that moderate to aggressive slope depressurization through dewatering wells and horizontal drains would be required to maintain slope integrity in the upper south-southwest slopes of the ACMA pit and main footwall of the Lewis pit (BGC 2011h, 2014c). Static stability analyses indicate that the highwall design satisfies the minimum design FOS criteria of 1.2 for multiple wall configurations under proposed dewatering conditions (BGC 2011k). The proposed open pit would be blasted and excavated with overall slope angles ranging from 23 to 42 degrees. Faults and fractures control the proposed slope design, including slope orientation, slope angles, bench height, and safety berm widths, which would be different for different parts of the pit depending on geotechnical sector. The safety berms between benches would capture rockfall and sloughing.

Pit maturation during Operations would gradually increase the extent of exposed pit walls that could be impacted by potential failure planes along faults and fractures. Experience would be gained throughout Operations as to the performance and deformation behavior of the slopes, and the design may be adjusted accordingly. In addition, ground control instrumentation is commonly employed to monitor slope movements. In terms of intensity, the potential slope instability effects in the pit (in the absence of earthquake loading, see Section 3.3.3.2.1) are expected to result in noticeable changes in resource character; while some sloughing of slopes is expected, the overall design is expected to be mostly adequate to mitigate this geohazard.

TSF Dam: Concerns regarding tailings dam stability are common. It is estimated that there are over 3,500 tailings dams around the world (Davies 2002). The three leading causes for tailings dam failures are overtopping, slope stability incidents, and earthquakes (ICOLD 2001). Other long-term failure mechanisms for tailings dams include cumulative damage (e.g. internal dam erosion and multiple earthquake events), geologic hazards (landslides, etc.), static load-induced liquefaction (the loss of strength in saturated material due to the buildup of porewater pressures unrelated to dynamic forces—most typically earthquakes), and changing weather patterns (ICOLD 2001).

Tailings dam incidents can be largely attributed to engineering design or construction failures. Independent reviews and regulatory oversight can also play a role. The 2014 Mt. Polley tailings dam breach in British Columbia is a good example. The cause of the breach was found to be failure to identify the extent and properties of an underlying clay layer, which led to misinterpreting static stability conditions. As noted in Section 3.3.3.2.1, the Donlin TSF dam would be placed on bedrock, which is not prone to the same type of failure. Certain types of dam safety inspections in British Columbia occur less frequently than in Alaska (5 years versus 3 years for high hazard dams). The Donlin TSF would also undergo owner-required independent technical reviews (AECOM 2015b; IEEEIRP 2015).

Static stability analysis was performed for the TSF dam to evaluate slope conditions and determine overall FOS for design purposes. ADNR (2005) provides guidelines on stability analysis methods and verification calculations required for dams of different classes, and indicate that target FOS should be proposed by the applicant during scoping. The abutments and dam side slopes, constructed of compacted rockfill, yielded FOS of 1.5 to 1.7, which meet or exceed the proposed minimum FOS of 1.5 for this structure (BGC 2011a). As the dam would be constructed on bedrock after removing all permafrost suspect materials, the stability analysis considered failure planes within weathered bedrock beneath the base of the structure for both the starter dam and the abutment, and main sections of the ultimate dam. In terms of intensity, impacts at the TSF dam caused by downslope movement from natural processes and man-made

causes during construction may or may not be noticeable, and the design is expected to be adequate to mitigate this geohazard.

**TSF Impoundment:** Colluvial soils and permafrost on side slopes beneath the tailings impoundment (upstream of the dam) could become unstable during construction leading to slippage and liner damage. As noted in Section 3.2.2.1.2 (Soils, Permafrost), sporadic permafrost may occur along lower slopes and becomes more infrequent on the upper side slopes at the TSF site. Preparation of the impoundment slopes prior to liner placement is expected to largely mitigate this concern. This would include clearing and grubbing, placement of coarse bedding material, and installation of underdrains to promote drainage out of unstable soils and reduce the potential for soil creep.

**Lower CWD:** A dormant landslide was identified near the planned Lower CWD on the southwestern slope of the American Creek Valley (BGC 2011c). The landslide was mapped on a slope less than 30 degrees with a maximum height of 310 feet. The estimated area of the landslide is approximately 60,000 square feet (ft<sup>2</sup>) and is located near the proposed south abutment of the dam. The landslide is classified as a translational slide with a potential failure surface along bedrock stratigraphic bedding planes.

During construction of the Lower CWD, specialized stabilization efforts are planned to minimize the potential for remobilization of the American Creek landslide and achieve recommended design FOS of 1.3 to 1.5 (BGC 2011c). Excavation near the south abutment would be monitored and conducted in a top-down method to minimize the potential for further slope instability. Temporary buttressing of the slope during construction would prevent potential remobilization of the landslide. A planned stabilization berm would be constructed at the toe of the landslide, which would increase the stability of the landslide. The berm would be constructed on a foundation free of unsuitable organics and fine-grained and potentially ice-rich soils, using free draining, competent, and durable sedimentary rockfill (BGC 2011c). The berm would be constructed concurrently with the dam, with dam rockfill placed as soon as foundation clearing and grubbing is implemented. Additional downslope movement during construction could require additional mitigation measures described in Chapter 5, Impact Avoidance, Minimization, and Mitigation.

In terms of intensity, potential slope stability effects at the Lower CWD would mostly result in noticeable changes in resource character; construction methods of the stabilization berm are expected to be adequate to mitigate this geohazard. There is a low probability of potentially acute or obvious changes in the resource character from landslide movement during the Construction Phase that would require additional mitigation to reduce the intensity.

**WRF:** The proposed WRF in American Creek Valley southeast of the pit would be constructed in increments of 100-foot lifts with a setback distance of 155 feet from the crest of the previous lift. The designed overall dump slope for the WRF is 3H:1V. The foundation preparation would lock in the toe of the WRF to competent soils to reduce the potential for increased instability. The WRF would include up to eight percent by volume of overburden materials that could weaken the strength of the stacked and compacted waste rock. The stability of the WRF was evaluated under static loading conditions for two foundation scenarios: 1) waste rock placed directly on in-situ soils; and 2) foundation preparation be excavated to eight feet and replacement with waste rock. The results showed that the stability of the proposed WRF meets or exceeds the recommended design criteria of 1.5 (BGC 2011b; Corps 2003) under static conditions, with FOS ranging from 1.82 to 2.13 for both scenarios, indicating stable slopes in

either case. (Implementation of foundation preparation that is crucial to maintain stability under earthquake loading conditions is described in Section 3.3.3.2.1.) Under pseudo-static loading, a factor of safety of 1.0 is generally considered acceptable using the Hynes-Griffin and Franklin (1984) approach where the horizontal seismic coefficient ( $k_h$ ) is equal to half the peak horizontal ground acceleration for the design event considered. Thus, the design of the WRF is expected to be adequate to mitigate potential slope stability effects on this structure under routine conditions (in the absence of earthquake loading). Monitoring for physical stability of the structure during Operations would be as described in Section 3.3.3.2.1.

Underdrains would be placed beneath the WRF in American Creek and its tributary drainages to promote internal drainage and stability. The size of the Lower CWD is such that it can store about 405 acre-feet of water without inundating any of the waste rock upgradient of it. As water storage volumes increase beyond that point, potential siltation of underdrain voids over time could pose geotechnical stability issues with the WRF pile. Water management strategies have been developed that limit the storage volume in the Lower CWD to mitigate this concern, as well as geochemical and water quality concerns with wetting and drying of the waste rock. Water would be pumped to the Upper CWD and used in the process plant when the Lower CWD storage volume exceeds 284 acre-feet; and in extremely wet years, water could also be pumped to the TSF, open pit, or sent to the water treatment plant and discharged (SRK 2017b).

**Other Facilities:** The mill, power plant, and fuel storage area would be constructed on engineered fill placed on a shallow bedrock ridge. As such, the likelihood of impacts caused by downslope movement from natural geomorphic processes and man-made causes would be low at these facilities, although fill slopes on the sides of the ridge could experience some sloughing. Roads constructed on side slopes at the Mine Site could also experience minor sloughing of cut and fill slopes. These potential effects are expected to be largely controlled through BMPs and ESC measures described in Section 3.2.3.2.3 (Soils, Erosion).

While avalanches are unlikely at the Mine Site due to the gentle mountaintops of the Kuskokwim Mountains, the intensity of local snow slides could vary along the steep southwest slope of American Creek or the north slope of Anaconda Creek, such as temporary burial of facilities and roads. The ore stockpile and lower contact water pond would be located below the steep American Creek slope, and the SRS and potential TSF dam construction activities would be located below the steep Anaconda Creek slope.

### *Closure*

**Open Pit Walls:** During closure and reclamation activities, waste rock would be used to backfill portions of the pit (SRK 2016e). Slope repressurization from cessation of pit dewatering could increase the likelihood of rockfall or landslides that could cause low to medium intensity effects such as slowing natural revegetation of pit walls or disruption to pit lake stratification in the post-Closure period (Section 3.7, Water Quality). Sufficiently large post-Closure wall failure could also cause high intensity effects such as collapse of structures or roads near the pit rim. Post-Closure slope instability would be addressed in part by rockfall catchment benches.

Potential instability of the northwest portion of the pit wall could be accentuated by groundwater infiltration from Crooked Creek, or lateral erosion from Crooked Creek if flood levels reach or exceed the extent of ancient floodplain deposits (Pleistocene terraces). Slope stability analysis of the northwest pit wall closest to Crooked Creek was conducted under static conditions assuming two saturated groundwater scenarios (partially full and full lake). The

results indicate FOS of 2.65 to 3.55 which are well above design criteria (FOS of 1.0 to 1.2) (BGC 2014j), indicating that large-scale slope failure in this part of the pit is unlikely in post-Closure.

The results of static slope stability analysis in post-Closure for other pit walls (cross-sections shown on Figure 3.3-9) indicate that FOS mostly meet or exceed design criteria with results in the range of 1.16 to 1.67 for the south and east walls of the pit. The lowest of these results represents the south-southwest wall of the pit under partially full lake scenario (about 10 years post-Closure), and is likely due to lower quality rock in this area.

**Pit Crest:** The ultimate pit crest in the northwest portion of the pit near the mouth of American Creek is separated from the edge of the Crooked Creek floodplain by a narrow geomorphic barrier of surficial overburden material composed of terrace gravel, colluvium, and alluvial fan deposits (Figure 3.3-10), the thickness of which is approximately 15 feet in this area (BGC 2014j). The distance between the pit crest and the edge of the Crooked Creek floodplain ranges from 50 feet to more than 200 feet in this area.

The integrity of the narrow barrier could be reduced through several processes. Lateral erosion from Crooked Creek channel migration could occur over a long period of time. Settlement of the pit crest due to permafrost thaw, vertical strain, or loss of integrity of the interior pit slope could decrease the stability of the barrier, thereby increasing the potential for Crooked Creek reaching and possibly overtopping the pit during a flood event. There is a moderate likelihood that up to 10 feet of settlement of the pit crest would occur by completion of mining at the northwest wall of ACMA pit. The elevation of the barrier at flood level sections A and B shown on Figure 3.3-10, at 367 to 376 feet, is approximately five to 10 feet above the estimated 100- to 500-year flood levels at these locations, depending on the specific location of measurement along the pit crest. Flood levels would be lower at the mouth of American Creek as the floodplain widens in this area. Because overtopping due to flooding would require greater than a 500-year flood event, the likelihood of disturbance of the geomorphic barrier is expected to be low (BGC 2014j), but would increase with increased settlement of the pit crest. In terms of intensity, potential instability of the internal pit walls and pit crest during the post-Closure period would mostly result in noticeable changes in resource character; the design is adequate for the expected range of geohazard conditions. There is a low probability of acute or obvious changes in the resource character in the event of northwest pit crest settlement and overtopping by Crooked Creek. The duration of effects could range from infrequent (e.g., pit lake restratification within weeks) to chronic effects that would not be anticipated to return to pre-activity levels even if actions that caused the impacts were to cease (e.g., pit wall failure). Recommendations to reduce the likelihood of these effects are provided in Chapter 5, Impact Avoidance, Minimization, and Mitigation.

**WRF, TSF, and Other Dams:** The potential for impacts from instability of these man-made structures or landslides caused by natural processes would be less during closure than described above under Construction and Operations. Closure activities at the TSF, WRF, and water dams are described in Section 3.3.3.2.1, Earthquakes.

Tailings deposition at the TSF would be modified several years prior to the end of Operations to direct the saturated portion to the southeast corner of the impoundment. The finished surface would be contoured and covered to promote surface runoff, reduce the potential for infiltration, and minimize impacts from downslope movement of waste materials. Potential seepage-related internal effects on stability of the TSF dam (Section 3.3.3.2.3) would be reduced through consolidation of tailings and pumping water captured in a rockfill capillary layer between the



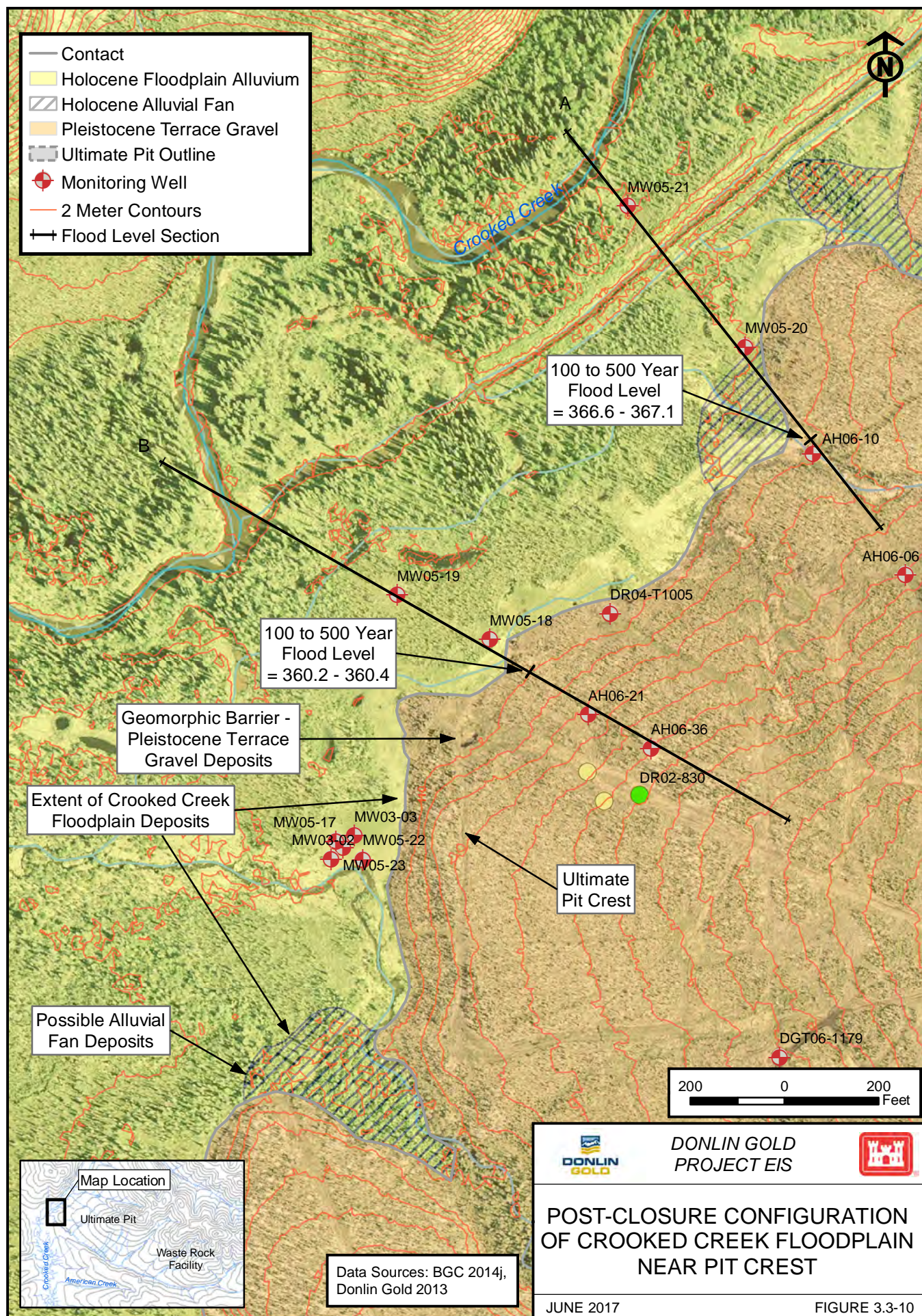
tailings and the cover material over a period of about 50 years. The downstream slope of the dam would be flattened to a 2H:1V slope, further reducing the likelihood of instability of this structure in post-Closure (SRK 2015g). Post-Closure monitoring for mass stability and seepage flow would be as described in Sections 3.3.3.2.1 (Earthquakes, Closure) and 3.3.3.2.3 (Other Geohazards, Closure).

The WRF would be reclaimed in stages currently with Operations. Placement of an engineered cover on the WRF would reduce infiltration and the likelihood of instability of this facility in post-Closure. The Lower and Upper CWDs and Snow Gulch FWD would be breached and regraded to lower slope angles to match natural grade (BGC 2011c), eliminating the potential of slope instability effects at these facilities in post-Closure.

### *Summary of Mine Site Impacts*

In terms of intensity, slope stability effects at the mine site would vary depending on the form of slope instability. Avalanches and snow slides would result in changes that may not be measurable or noticeable. Impacts may result in noticeable changes in the resource character (e.g., static stability of pit walls in Operations). There is a low probability of acute or obvious effects at two locations (activation of American Creek Landslide near Lower CWD during construction, and northwest pit crest settlement and overtopping by Crooked Creek), that could potentially be reduced in intensity with additional mitigation. The duration of effects would range from infrequent (e.g., landslide impacts to Lower CWD construction) to chronic, where resource conditions would not be anticipated to return to pre-activity levels even if actions that caused the impacts were to cease (e.g., pit wall failure). The extent or scope of effects would mostly remain within the footprint of the mine site. In terms of context, slope stability hazards are a usual or ordinary occurrence in Alaska, which are governed by regulation for certain structures (e.g., dams).







## Transportation Corridor

### *Construction, Operations, and Closure*

Effects of downslope movement during Construction, Operations, and Closure at Transportation Corridor facilities could occur along the mine access road and material sites, new airstrip, permanent camp, and Angyaruaq (Jungjuk) and Dutch Harbor ports. The potential for unstable slopes in these areas is described in Section 3.3.2.2.2. Moderate to steep terrain with narrow stream drainages occur along roughly one-third of the mine access road and on either side of the airstrip. Minor sloughing of cut and fill slopes along the road and airstrip would be mostly controlled through BMPs and ESC measures recommended in Section 3.2, Soils. Larger areas of slope instability could lead to safety issues for truck traffic during all phases of the project. Impacts from local snow slides along the road would be negligible as the road would only be operated seasonally in the summer. Localized rockfall or debris slides could occur at bedrock and gravel material sites along the road, and at the permanent camp, which would be constructed in one of the material site locations. Slope issues at the Angyaruaq (Jungjuk) and Bethel ports would be expected to be minimal due to relatively gentle slopes or flat terrain. Unstable slopes and snow slides are possible in areas of steep coastal cliffs at potential fuel storage sites around Dutch Harbor (Figure 3.3-4), and avalanches could originate from the slopes of Makushin volcano, northwest of potential port sites.

In terms of intensity, the effects of slope stability hazards would vary at most transportation facilities and may result in noticeable changes in the resource character. Site-specific slope design may be warranted for certain sections of the mine access road, material sites, and the Dutch Harbor fuel storage expansion to mitigate slope stability hazards. Stability of road side slopes, and blasted or ripped slopes at material sites, are expected to be of adequate design due to state road engineering standards or typical benching of cut slopes at material sites (ADOT&PF 2004; Shannon & Wilson 2012).

### *Summary of Transportation Corridor Impacts*

In terms of intensity, slope stability effects at Transportation Corridor facilities would vary depending on the form of slope instability. Rockfall or debris slides at material sites could result in noticeable changes in the resource character. Standard engineering practices would mitigate potential effects in certain areas (e.g., road side slopes) to reduce the intensity of impacts. The duration of effects would range from infrequent (e.g., slope mitigation during construction) to chronic effects that would not be anticipated to return to pre-activity levels even if actions that caused the impacts were to cease (e.g., material site rockfall). The extent or scope of effects would mostly remain within the immediate vicinity of the various facility footprints. In terms of context, slope stability hazards are a usual or ordinary occurrence in Alaska, but are typically addressed through state or national standards and guides that are adopted by government agencies for certain structures (e.g., roads, material sites).

## Pipeline

### *Construction*

Slope stability effects during construction activities along the pipeline could occur, depending on local terrain. In particular, the central portion of the pipeline route through the steep mountainous areas of the Alaska Range could experience moderate to severe downslope

movements. While numerous pipelines located in mountainous terrain are operated safely and without incident around the world (for example, the Trans-Alaska Pipeline which crosses both the Brooks and Alaska ranges), several have experienced notable slope stability problems. Examples include pipeline ruptures, abandonment, and rerouting due to naturally occurring landslides in the Andes and Caucasus mountains (Lalazashvili and Ingorokva 2006; Porter et al. 2006); and activation of landslides and mudflows by poor construction practices on Sakhalin Island resulting in work stoppage (Afanasiev 2006).

Direct impacts during proposed pipeline construction activities in the Alaska Range could occur from debris flows, rockfall, rock or debris avalanches, snow avalanches, mudflows, existing landslides activated by cut slopes, loss of soil cover by exposure, and channel runoff and erosion. These geohazards could affect the integrity of the pipeline, the ROW, stream crossing passages, temporary access roads, airstrips, and constructed foundation pads for structures. Localized rockfall or debris slides could also occur at bedrock and gravel material sites along the pipeline. The largest impacts would likely occur along the pipeline corridor from the upper Happy River drainage to Farewell, from approximately MP 108 to MP 145. From MP 111 to MP 139, 37 debris flows have been mapped on the southwest side of the pipeline route and 30 debris flows on the northeast side (Figure 3.3-4, Section 3.3.2.3.2). However, most slope instability impacts would be limited to the eight mile section in the Threemile Creek/Jones River area from MP 111 to MP 119, some of which has been identified for specialized construction techniques to mitigate these geohazards, such as HDD or deep trenching of bedrock (Fueg 2014).

The likelihood of impacts from landslides and avalanches is considered low to moderate for the rest of the pipeline north and south of the Alaska Range. Terrain from Farewell to the Mine Site consists of gentle to moderate sloping uplands of the upper Kuskokwim River region and Kuskokwim Mountains. The highest potential for ground movement in this area may occur along temporary access roads that would traverse many narrow stream drainages with moderate to steep terrain. Slope stability issues along the pipeline south of the Alaska Range would likely be limited to minor sloughing of cut and fill slopes, which would be mitigated by planned BMPs and ESC measures described in Section 3.2, Soils.

In terms of intensity, effects of slope stability hazards would vary for the whole pipeline corridor, and may result in noticeable changes in the resource character. As described in Section 3.3.2.1, Earthquakes, the proposed pipeline would likely require a PHMSA Special Permit for strain-based design (SBD), which would have conditions considered part of the proposed project for identifying geohazards (including slope stability issues), design specifications, and inspections to ensure that geotechnical limits for SBD are not exceeded (Appendix E). Stability of temporary road slopes, and of blasted or ripped slopes at material sites, are expected to be adequate for design due to state or national road engineering standards or typical benching of cut slopes at material sites (USDOT 2011; Shannon & Wilson 2012). Thus, most effects would likely be minimized through planned mitigation.

### *Operations*

Impacts from slope stability hazards during Operations would be similar to the Construction phase. Slope stability effects such as debris slides, avalanches, and slow creep of active landslides could continue to occur in steep areas of the Alaska Range. The pipeline would be monitored for such ground movements during Operations. Serious, continuing slope stability



issues could warrant additional geotechnical engineering data collection during Operations to modify or repair specific sections of the pipeline. Impacts in steep sections of the Alaska Range could potentially last for the life of the project in the case of reactivated landslides or creep that requires monitoring, inspection, and repair over years.

### *Closure*

Reclamation of the ROW, temporary access roads, and other off-ROW facilities immediately following Construction, through restoration and stabilization of natural drainage courses, and grading and contouring to provide adequate drainage and restore cut slopes to natural terrain conditions, would reduce the potential for slope failures and sloughing in most areas along the pipeline.

Impacts from slope stability hazards on the pipeline would be reduced or eliminated during closure, due to in-place decommissioning of the pipeline that would not involve additional grading of the ROW.

### *Summary of Pipeline Impacts*

In terms of intensity, slope stability effects along the pipeline corridor would vary depending on local terrain. In most cases, sloughing in low to moderate relief areas may not be measurable or noticeable. Noticeable changes in the resource character could occur but the design would be adequate to mitigate debris flows in the Alaska Range, assuming that effects at specific high hazard locations in the Alaska Range are reduced through additional geotechnical investigation and/or special design prior to construction. The duration of potential effects could range from infrequent ROW or road cut damage repairable within days to month, to active debris flows that require pipeline monitoring and repairs over the life of the project. The extent or scope of effects would be limited mostly within the immediate vicinity of the pipeline ROW and associated facilities. Slope stability hazards are considered a usual or ordinary phenomenon in Alaska, which are governed by regulation for pipelines.

#### **3.3.3.2.3 OTHER GEOHAZARDS**

##### **Mine Site – Dam Seepage and Ice hazards**

Other geohazards that were considered for the Mine Site include seepage through the dams, which could be a source of piping<sup>10</sup> and internal erosion, and lead to slope erosion, excessive water loss, or dam failure; and ice damage to the TSF impoundment liner, which could cause liner leakage and increased contaminated groundwater seepage to the underdrains and SRS.

### *Construction and Operations*

#### **Dam Seepage**

Seepage and piping are typically controlled through the use of filters, removal of cohesionless (gravel) soils during foundation preparation, reduction in seepage flow by installing impermeable barriers, compaction within and beneath the dam, and lengthening the flowpaths of water within and around the dam (Thomas et al. 2014). A description of the TSF dam design

---

<sup>10</sup> Piping is the internal erosion of a dam foundation or fill caused by seepage. Piping typically starts at the toe of a dam and works upstream, potentially forming channels or pipes under the dam over years.

and construction materials is provided in Section 3.3.3.2.1 (Earthquakes) and shown on Figure 3.3-7. Features at the TSF and water dams at the Mine Site that would reduce the potential for piping and seepage-related problems include the following:

- Multiple filter zones at gradually smaller gradations to prevent movement of soil particles through low permeable flow paths;
- Fully lined impoundment and dam face (or in the case of the water dams, along the zigzag centerline of the dam with upstream runout) to reduce seepage;
- Liner material composed of single-sided textured (TSF) or double-sided textured (water dams) 60-mil LLDPE geomembrane selected for cold-climate constructability, with UV resistance, adequate tensile strength and puncture resistance, and strain tolerance;
- Dam foundation preparation involving the removal of all overburden to bedrock;
- Compaction of liner bedding, filter zones, and rockfill in 1- to 3-foot lifts;
- Routing of Anaconda Creek baseflow through underdrains beneath the TSF impoundment;
- If necessary, use of evaporators in TSF pond during late spring and summer to reduce water volume; and
- Daily inspections of dam slopes, and daily readings of seepage pumps, seepage well water levels, and SRS pond levels during Operations.

Seepage analyses were conducted for the TSF starter and ultimate dams (BGC 2011a, 2014b), as well as for the water dams at the Mine Site (BGC 2011c), in order to model flow through and beneath the dam, provide an estimate of the total seepage that would report to the SRS or other pumping facilities at the water dams, and provide inputs to static stability analyses (Section 3.3.3.2.2) and site-wide water balance and groundwater modeling (Sections 3.5, Surface Water Hydrology, and 3.6, Groundwater Hydrology). The seepage analyses conducted for the Mine Site dams utilized Seep/W modeling software to predict seepage rates. Inputs, assumptions, boundary conditions, and modeling process are described in detail in the Project Feasibility Study Update prepared by BGC Engineering in 2011 (BGC 2011a). The individual analyses proposed mine facilities are described below.

**TSF Dam and FWDDs:** Seepage analysis conducted for the TSF dam and North FWDD utilized hydraulic conductivity values for bedrock material within the footprint of the dams, and site-specific dam dimensions for the steady-state seepage analysis. (A separate analysis was not performed on the South FWDD, as it is identical in design to the North FWDD.) Both the TSF dam and FWDDs would be constructed on bedrock following removal of surficial overburden material. Zones of higher hydraulic conductivity were observed at lower elevations along Anaconda Creek beneath the southern abutment of the planned TSF. The seepage analysis utilized the zone of higher hydraulic conductivity to model groundwater flow to the underdrain. Within the Mine Site area, specifically in the pit region, bedrock hydraulic conductivity decreases with depth (BGC 2011a). Hydraulic conductivity values for tailings, dam materials, overburden, combined bedding and underdrain material, combined bedding and colluvium overburden, and the LLDPE membrane liner were also used as input parameters in the seepage analysis.

Seepage analyses were conducted for both the starter and ultimate TSF dams using maximum pond and tailings elevations. Total seepage catchment is planned at the SRS facility downstream of the TSF dam that would capture Anaconda Creek baseflow discharge to the underdrain, seepage from upstream FWDDs, seepage through the liner from the tailings impoundment, and some groundwater flow through bedrock around and beneath the dam (BGC 2011a). Baseflow to the underdrain is expected to decline over time as a result of lining the impoundment. Seepage flow from the FWDDs would be replaced by baseflow through the underdrains during the Operations phase after completion of the TSF starter dam and removal of the FWDDs. Total flow through the TSF system, including both tailings/liner seepage and baseflow, is estimated to range from about 500 to 1,000 gpm, depending on season, operational year, and water management at other Mine Site facilities. Estimated seepage through the TSF dam represents about 0.3 percent of these totals for the starter dam, and about 4 percent of total flow for the ultimate main dam (BGC 2015f, SRK 2017b).

Some flexibility for controlling seepage flow has been built into overall water management at the Mine Site. Under average precipitation conditions, the Mine Site is predicted to operate with a water surplus. Excess water would be stored in the TSF pond, which would gradually rise over the life of the mine. Treatment and release of some pit dewatering water, CWD water, SRS water, and excess precipitation water in the TSF provides an important mechanism to reduce pond storage volumes at the TSF. Without this discharge, pond volumes would be more than twice that predicted with dewatering discharge (BGC 2015f, SRK 2017b). While the results of seepage analysis assume maximum operating pond conditions, actual water levels would likely be managed at lower levels by discharging of pit dewatering water during Operations and using evaporators in the TSF pond, which would reduce the probability that seepage hazards would occur.

Based on multiple features in dam design (the use of an LLDPE liner, and a series of filter-zones), construction, and operations that would minimize seepage and piping, and the results of seepage analyses predicting very little seepage through the dam, the likelihood of internal erosion and related effects at the TSF dam would be reduced (design adequate to mitigate hazard).

Water Dams: Seepage analyses conducted for the water dams at the Mine Site (Snow Gulch FWD, American Creek FWDD, and Lower and Upper CWDs) utilized similar methodologies and hydraulic conductivity values as the TSF analyses. Based on the geometric mean of bedrock hydraulic conductivity data collected at these sites, seepage rates ranged from about 100 gpm for the American FWDD to 400 gpm for the Lower CWD (BGC 2011c). However, seepage estimates using upperbound hydraulic conductivity values for bedrock could result in seepage rates as high as 10 times these totals.

Potential consequences of high seepage rates include difficulty in managing pumping, internal erosion, and having to conduct seepage mitigation after construction. BGC (2011c) provides recommendations for additional testing of bedrock hydraulic conductivities in final design to mitigate the potential effects of high seepage rates. In the event of excess water buildup in the Lower CWD in an extremely wet year, a contingency plan would be in place to pump excess water to either the TSF or open pit, depending on the construction status of these facilities, or to treat the water in the water treatment plant (WTP) and release it (SRK 2017b). Because multiple features in construction and operations of the water dams would minimize seepage and piping,

and additional bedrock testing is anticipated in final design, the design is expected to be adequate for the expected range of geohazard effects on dam stability.

### Ice Loading

The TSF liner would be subjected to vertical and lateral stresses from ice on top of the TSF pond as a result of wind movement or water levels rising and falling, which could cause liner damage and increased seepage flow if not mitigated. Efforts would be made to minimize ice touching the liner through tailings beach development and monitoring. However, based on the 50<sup>th</sup> percentile site-wide water balance model, ice could potentially contact the liner over significant lengths ranging from 8,080 to 34,610 feet (about 1-1/2 to 6-1/2 miles) about half of the time (SRK 2016c). Donlin Gold has proposed additional contingencies to prevent damage to the liner. These include operating the TSF such that the tailings beach above water has a slope of 0.5 percent, and the beach below water has a slope of 1.0 percent, in order to keep the TSF pond and floating ice within the inner core; and an update to the tailings deposition plan in final design (SRK 2016c).

### Closure

Seepage through the TSF system would decline during closure as surface flow is reversed to Crevice Creek, infiltration of the impoundment is reduced following installation of the cover material, and tailings consolidate reducing porosity and squeezing out porewater. Total seepage reporting to the SRS in post-Closure after about 50 years is estimated to be about 400 gpm, of which 4 percent is estimated to come from seepage from the tailings (SRK 2017b). Closure and post-Closure monitoring would include:

- Seepage flow monitoring quarterly until results indicate a stable condition;
- Mass stability inspections annually for the first 5 years, and then every 5 years thereafter until results indicate a stable condition (SRK 2016e); and
- Inspections after significant seismic events (ground shaking large enough to potentially affect dam stability, i.e., > 0.1g, Table 3.3-3).

The water dams at the Mine Site would be removed during closure, eliminating potential effects of seepage on stability of these structures (Snow Gulch FWD, American Creek FWDD, and Lower and Upper CWDs).

### Summary of Mine Site Impacts

The design of the TSF and water dams is expected to be adequate for the expected range of seepage and ice effects on dam stability and liner leakage, assuming additional bedrock testing at water dams would occur in final design, and that contingencies for protecting the TSF liner from ice damage are effective during Operations. The duration of effects would vary based on the type and location of activities. Water dams removed at Closure would limit the duration of effects from these structures to the life of the mine. TSF dam seepage would involve chronic effects persisting beyond the life of the mine, and while the effects on dam stability would reduce following closure, they would not return to pre-mining conditions. The extent or scope of effects would be mostly within the footprint of the mine site. Seepage hazards are considered a usual or ordinary phenomenon in natural materials and water-retaining structures, which are governed by dam safety regulations (ADNR 2005).



### Transportation Corridor – Tsunamis, Volcanoes

Other geohazards considered for Transportation Corridor facilities include the effects of tsunamis and volcanoes on the Dutch Harbor fuel storage facility. The likelihood that these geohazards would impact other Transportation Corridor facilities along the Kuskokwim River corridor is low, due to distance from the marine coastal shoreline, shallow water depths near the mouth of the Kuskokwim River, lack of coastal exposure to the Pacific Ocean basin, and distance from the active volcanoes located in the Aleutian Islands.

#### *Construction and Operations*

As described in Section 3.3.2, Affected Environment, the Dutch Harbor area has the potential to experience both tsunamis generated by a large earthquake, and effects from nearby active volcanoes. Tsunamis could reach Dutch Harbor in the event of a large magnitude earthquake in the Aleutian chain or Pacific Ocean basin. Two earthquakes in 2011 generated tsunami warnings for Dutch Harbor; one was centered near Adak and the other in Japan. While tsunami estimates are not available for Dutch Harbor, predictions for the south coast of Kodiak, which is similarly exposed to the Pacific Ocean basin, range from about 10 to 60 feet for a 100- to 500-year return period event (Crawford 1987). Typical effects from tsunamis could include coastal flooding and risks to worker safety and structures, such as docks and storage tanks. Tanks can be damaged through initial wave crushing or buoyancy failure, which can cause tipping and sliding (Brooker 2011). Effects at the fuel storage site would vary depending on final site selection, elevation, and coastal configuration. (The effects of a potential fuel tank release are discussed in Section 3.24, Spill Risk.)

Three active volcanoes (Makushin, Okmok, and Mount Cleveland) are located 15, 73, and 158 miles respectively from Dutch Harbor. Makushin last erupted in 1995, Okmok in 2008, and Mount Cleveland in 2013. Impacts from an eruption of Mount Cleveland or Okmok could include a volcanic ash cloud and fallout that could be transported by prevailing northeast winds along the Aleutian chain towards Dutch Harbor (Figure 3.3-3). An eruption of Makushin volcano that produces volcanic ash clouds and fallout, as well as lahars and flooding, could directly impact the Dutch Harbor fuel storage facility, depending on site location (Figure 3.2-4, Soils). Volcanic ash particles are particularly abrasive, corrosive, and pervasive. Typical effects from ashfall include damage to equipment and engine components and electronics.

In terms of intensity, impacts from tsunamis and volcanoes on the Dutch Harbor fuel storage facility would vary depending on the location and elevation of the storage tanks and assuming planned mitigation occurs. These impacts may be measurable or noticeable. Effects could result in infrequent disruption of shipping and fuel supply to the mine. These effects are considered indirect due to third party operation at Dutch Harbor. Typical mitigation might include a vulnerability analysis of equipment and facilities, incorporation of flooding into tank design (e.g., tie-downs), and emergency action planning with tsunami escape routes.

#### *Closure*

Potential impacts to the Dutch Harbor site in closure would be the same as under Construction and Operations, as this third-party facility would likely continue to operate independently of the proposed action.

### *Summary of Transportation Corridor Impacts*

In terms of intensity, tsunami and volcano effects on Transportation Corridor facilities would vary depending on the location and type of geohazard. Impacts may not be noticeable at Bethel dock. Tsunami flooding or ashfall causing disruption of Dutch Harbor fuel storage operations would result in noticeable impacts on facilities or changes in resource character. These impacts assume that typical hazard planning occurs and tank design incorporates tsunami risk. The duration of effects on these structures would be mostly infrequent (e.g., several days of ashfall interruption over a period of months). However, effects could occur from Construction through the life of the project. The extent or scope of these geohazards on transportation infrastructure would be limited to the immediate vicinity of the Dutch Harbor facility. While tsunamis and volcanoes are relatively uncommon events, the context of such events are considered usual or ordinary, because it is reasonable to expect that an event could occur sometime over the life of the project. Any actions that would occur at Dutch Harbor or the Port of Bethel at the Bethel Yard Dock are not part of the proposed action, and are considered connected actions (see Section 1.2.1, Connected Actions, in Chapter 1, Project Introduction and Purpose and Need).

### Pipeline – HDD Frac-out, Tsunamis, Volcanoes

Other geohazards considered for the natural gas pipeline include the risk of HDD frac-out during drilling at certain river crossings, and tsunamis and volcanoes for the eastern end of the pipeline.

#### *Construction*

HDD Frac-out. The pipeline corridor crosses six major rivers where the pipeline will be installed using HDD methods: Skwentna River (MP 50), Happy River (MP 86), Kuskokwim River (MP 240), East Fork George River (MP 283), George River (MP 291), and North Fork George River (MP 298). Under Alternative 2 – North Option, two HDD crossings of Happy River tributaries would be installed, and the main Happy River HDD would be avoided. HDD installation carries the risk of “frac-out,” in which drilling fluids are lost into fractures or voids, caused in part by hydraulic pressure during drilling, and surface into the river above, potentially affecting water quality. The HDD method is well-suited for small diameter pipeline installation as long as favorable subsurface conditions exist. Geotechnical engineering investigations at the river crossings were conducted to evaluate the suitability of subsurface conditions. In general, the presence of either cobble-sized gravel soils or highly fractured bedrock could cause loss of drilling fluid circulation or release of drill fluids into the overlying water body.

The risk of frac-out has been partly mitigated through site selection of river crossings suitable for HDD. Based on geotechnical drilling results to date, some river crossings (e.g., South Fork Kuskokwim River) were considered unsuitable for HDD due to unfavorable subsurface conditions, and would be crossed by other methods.

Based on existing geotechnical conditions described in Section 3.3.3.2.1, the feasibility and risk of HDD at each of the six rivers proposed for HDD are described below:

- Skwentna: The clay/silt layer identified at this location is considered feasible for HDD, and suggests a low to moderate potential for loss of drilling fluid.
- Happy: Subsurface conditions are considered generally feasible for HDD, though the extensive bedrock fracturing is less than optimal (CH2MHill 2011b), and suggest a

moderate potential for loss of drilling fluid. Subsurface bedrock conditions underlying the North Option route are expected to be similar to those beneath the proposed route.

- Kuskokwim: The fractured bedrock below alluvium at this site is considered feasible for HDD, and suggests a moderate potential for loss of drilling fluid.
- East Fork George: The sandstone/mudstone bedrock below alluvium at this site is considered feasible for HDD, and suggests a low to moderate potential for loss of drilling fluid.
- George: The fractured bedrock below alluvium at this site is considered feasible for HDD, and suggests a moderate potential for loss of drilling fluid.
- North Fork George: The slightly fractured bedrock below alluvium at this site is considered feasible for HDD, and suggests a low to moderate potential for loss of drilling fluid.

Thus, of the six rivers proposed for HDD drilling, half are considered to have low to moderate risk of frac-out, and half considered to have moderate risk. Those with lower risk would be drilled through either thick clays (at Skwentna crossing) or slightly to moderately weathered and fractured bedrock (East and North George crossings). Those with higher risk would be drilled through more extensively fractured bedrock (Happy, main Kuskokwim, and George crossings) (CH2MHill 2011b).

A major frac-out at any one of these locations could cause temporary high intensity effects on water quality, such as introduction of suspended solids or other constituents (depending on drilling fluid) at levels temporarily above ADEC water quality standards (Section 3.7, Water Quality). Such an event would be readily detected due to the loss of pressure and drilling mud. In this event, water quality effects would likely return to baseline conditions in a relatively short timeframe. Effects could range from local to regional in extent, with water quality impacts potentially extending downstream beyond the immediate vicinity of the crossing within a short timeframe depending on the volume of mud loss, river conditions, and the length of time needed to identify and control drilling pressures.

Besides advance site selection, the risk of frac-out during HDD drilling is typically mitigated through site-specific contingency planning. A design feature to complete plans for each HDD crossing during final design is provided in Chapter 5, Impact Avoidance, Minimization, and Mitigation (Table 5.2-1), that would likely reduce the intensity of these effects (i.e., construction methods adequate to control this geohazard).

The response to HDD failure depends on the cause of the failure. Frac-out can be caused by geology (underlying materials), geometry (the angle of the crossing), and location (size of the crossing). The most common response to HDD frac-out is to modify the drilling methods. Changing the angle or depth of the HDD can prevent issues, as well as the consistency of drilling fluids and drill speed.

Tsunamis. As described in Section 3.3.2, Affected Environment, the Beluga area at the southeast end of the pipeline has a low risk of tsunamis from an earthquake, with predicted maximum wave heights at on the order of 12 to 14 feet at high tide for a 100- to 500-year event. Impacts from a tsunami are dependent on the location of the earthquake source and bathymetry, coastline configuration, and tidal interactions. In the event of a tsunami, effects could include flooding and coastal erosion from waters that reach the elevation of the barge landing only, as

the proposed storage yards and camp at Beluga are at elevations of roughly 100 feet. Thus, the likelihood and intensity of these effects on the eastern end of the pipeline and off-ROW facilities is expected to be low.

**Volcanoes.** Several active volcanoes are located within 50 to 200 miles of the eastern portion of the pipeline (Figure 3.3-3). As described in Section 3.3.2.3.3, Mount Spurr, the closest active volcano to the east end of the corridor last erupted in 1992. Augustine, Iliamna, and Redoubt volcanoes last erupted in 2006, 1953, and 2009, respectively. Volcanic ash clouds and fallout from these volcanoes could affect the eastern portion of the pipeline corridor. In addition, a minor volcanic episode on the Hayes volcano could cause ice and snowmelt that could generate a lahar, debris flow, or flooding down the Hayes Glacier and into the Skwentna River drainage that crosses the pipeline at MP 50. The Hayes River comes closest to the pipeline ROW near MP 70 near its confluence with the Skwentna River, but does not cross it. The likelihood of these geohazards occurring over the life of the project is considered low for lahar-generated flooding and moderate for ashfall events.

The effects from ashfall or lahars on the pipeline and appurtenant facilities could include damage to equipment and electronics at the compressor station, transmission line, valves, and metering stations; interruption of power supply; or flooding and erosion at the Skwentna river crossing. Effects of ashfall could be of highest intensity at the compressor station, where ashfall could plug filters, ruin motors, trip transmission lines, or cause a power outage. Disruption of compressor station operations could temporarily shut down gas flow through the pipeline, although diesel fuel reserves at the mine would likely prevent interruption of power supply at the mine. Interruption of air travel in the Anchorage Bowl could temporarily disrupt labor schedules and delivery of supplies to the compressor station. The likelihood of lahar-related effects on the pipeline at the Skwentna HDD river crossing is low, because the pipeline would be well below scour depth.

The likelihood and intensity of volcanic effects on the pipeline and appurtenant structures is expected to vary, and may result in noticeable changes in the resource character. These impacts assume that potential ashfall effects on the compressor station and air travel would be mitigated through advance planning, such as a vulnerability analysis of equipment, flexible labor scheduling, and backup fuel supply at the mine. The duration of effects would typically last only days, but may involve intermittent episodes over months. The extent or scope of effects would be felt beyond a local area, in that they could affect both Pipeline and Transportation Corridor facility operations.

### *Operations*

There would be no effects from HDD frac-out during Operations, as this hazard would only occur during drilling and pipe installation. Likewise, there would be little to no impact from a tsunami during Operations, as this hazard would only affect the barge landing area of the project that is primarily used during construction. There would be little to no impacts from volcanic hazards to the pipeline during Operations, since most of the pipeline would be buried. However, ashfall could cause damage and temporary shutdown of the compressor station.



### *Closure*

Impacts from volcanic geohazards on the pipeline and associated facilities during closure would be reduced or eliminated, due to removal of above ground facilities and in-place decommissioning of the pipeline.

### *Summary of Pipeline Impacts*

In terms of intensity, the effects of HDD frac-out, tsunamis, and volcanoes on the pipeline and related facilities would vary depending on the type and location of activities. A tsunami at Beluga barge landing would result in impacts or changes in the resource character that may not be measurable or noticeable. However, ashfall causing a disruption at a compressor station would be a noticeable effect. There is a low probability of acute or obvious effects on river water quality in the event of HDD frac-out that could potentially be reduced in intensity with additional mitigation planning. The duration of effects would be mostly limited to several days (e.g., control of HDD frac-out). The extent or scope of effects would range from those limited to discrete portions of the Project Area (e.g., tsunamis) to those beyond a local area (e.g., ashfall affecting pipeline operations). These geohazards are considered usual or ordinary in context for the following reasons: their likelihood of occurrence ranges from common to uncommon; water is a common resource, but reductions in water quality are governed by regulation.

#### **3.3.3.2.4 CLIMATE CHANGE**

Predicted overall increases in temperatures and precipitation, and changes in the patterns of their distribution have the potential to influence the projected effects of the Donlin Gold Project on some geologic hazards, including geomorphological processes (landslides and avalanches) and coastal hazards (flooding and erosion). These effects are tied to changes in water resources as discussed in Section 3.26.4.2.2, Climate Change.

#### **3.3.3.2.5 Summary of Alternative 2 Impacts**

Applying the methodology defined in Table 3.3-1 to the information and data presented in this section, Alternative 2 has potential direct and indirect impacts from geohazards. Table 3.3-4 provides a summary of impacts by the four assessment factors.

Direct effects from earthquakes, slope instability, or dam seepage hazards at the Mine Site would mostly range from effects that may or may not be noticeable (e.g., liquefaction) to noticeable changes in project structures or resource character (e.g., design of TSF dam is adequate to mitigate earthquake or seepage hazards). There is a low probability of acute or obvious effects at several specific locations and situations listed in Table 3.3-4 that could potentially be reduced in intensity with additional mitigation (see Chapter 5, Impact Avoidance, Minimization, and Mitigation). Effects would range from infrequent (e.g., landslide impacts to Lower CWD construction) to impacts lasting beyond the life of the mine (e.g., pit wall failure). The extent or scope of effects would mostly be within the footprint of the Mine Site. The context of geohazards are considered a usual or ordinary geologic phenomenon in Alaska, which are governed by regulation for certain structures (e.g., dams).

Direct impacts on transportation facilities would mostly range from effects that may or may not be noticeable (e.g., ground shaking from ~~form~~ small earthquakes) to noticeable changes in the resource character (e.g., ashfall causing noticeable disruption of Dutch Harbor fuel storage

operations), assuming that additional engineering analysis considered standard practice would take place during final design (e.g., seismic design of bridges and tanks). The duration of seismic hazard effects on these structures would range from infrequent (minor repairs) to impacts lasting through the life of the mine (material site rock slides). The extent or scope of effects would mostly be within the immediate vicinity of the various facility footprints. The context of geohazards are considered a usual or ordinary geologic phenomenon in Alaska, that are typically addressed through national standards and guides adopted by federal and state government agencies for certain structures (e.g., bridges, tanks, docks).

Direct effects of geohazards along the pipeline corridor would mostly range from unmeasurable or unnoticeable (e.g., tsunami at Beluga barge landing) to noticeable changes in the resource character (e.g., fault crossing design adequate to withstand earthquake lateral displacement). There is a low probability of acute or obvious changes in the resource character in the event of HDD frac-out that could be reduced in intensity with additional mitigation (see Chapter 5, Impact Avoidance, Minimization, and Mitigation). The duration of effects could range from infrequent (e.g., damage reparable within days to months) to impacts lasting from the end of the Construction Phase and through the life of the mine (e.g., active debris flows in Alaska Range that require pipeline monitoring and repairs over the life of the project). The extent or scope of effects would range from within the immediate vicinity of the pipeline ROW and associated facilities to ashfall affecting both transportation facilities and pipeline operations. The context of geohazards are considered a usual or ordinary geologic phenomenon in Alaska, which are governed by regulation for pipelines and water quality.

#### 3.3.3.2.6 MITIGATION AND MONITORING FOR ALTERNATIVE 2

Effects determinations take into account impact reducing design features (Table 5.2-1 in Chapter 5, Impact Avoidance, Minimization, and Mitigation) proposed by Donlin Gold and also the Standard Permit Conditions and BMPs (Section 5.3) that would be implemented. Several examples of these are presented below, and others are discussed above in Section 3.3.3.2.1 and Section 3.3.3.2.3.

Design features important for reducing impacts from geohazards and seismic conditions include:

- Areas of disturbed bedrock and surficial deposits along the pipeline ROW, roads, and material sites would be contoured to match existing landforms as feasible, ripped to mitigate compaction effects, covered with growth media as needed and revegetated, and would support the overall drainage of the site, the long-term geotechnical stability, and post-mining land use;
- The TSF and water dams will be designed using rockfill, bedrock foundations, multiple filter zones, liners, and downstream construction methods to address seismic hazards, static stability, and seepage concerns. This aligns with specific Mount Polley Independent Review Panel recommendations for Best Applicable Practices (BAP) for tailings retention dam design. Final design would be reviewed by ADNR Dam Safety and subject to change as needed to protect life and property;
- Based on the proposed design, the WRF stability meets or exceeds industry design criteria under both static and pseudo-static (earthquake) loading conditions;

- The overall and area-specific pit wall slopes were designed to accommodate varying faults, fractures, and rock quality to ensure stability;
- Seismic stability analyses of the southern pit wall in the post-Closure period would include analysis with high level seismic event possibilities (due to recovered groundwater levels), and would include discussion with permitting agencies in final design as to acceptable level of risk for the post-Closure pit. Experience gained during Operations as to performance and deformation of the pit walls would be taken into account if there is a need to modify location of the waste rock backfill accordingly (as a buttressing effect) to increase the post-Closure stability of the pit (BGC 2014j);
- Further investigation and revised seismic stability analysis of the WRF design criteria and plans for excavation at the WRF toe would continue to assess if deeper liquefiable materials exist and would require additional excavation during site preparation;
- Monitor slope stability with sufficient lead time to preclude the potential for a breach to occur at the narrow geomorphic barrier between the Crooked Creek floodplain and the northwest pit crest;
- Monitor the American Creek Landslide area during construction of the Lower CWD (see Sections 3.3.2.1.2 and 3.3.3.2.2), utilizing instrumentation such as an inclinometer and piezometer (BGC 2011c), for indications of downslope movement and the need for additional mitigation measures beyond the planned stabilization berm;
- The above-ground fault crossing of the pipeline was designed to resist surface fault rupture hazards, and would be designed to withstand the stress that could occur during a seismic event;
- A special permit granted by PHMSA would allow the use of strain based design in areas where geotechnical hazards may be present to maintain equivalent levels of safety. The strain based design may use heavier wall pipe in these areas, rather than just using the wall thickness required for pressure containment, so that equivalent levels of safety are maintained; and
- The pipeline route has been selected, and will continue to be refined in detailed design, to avoid slope stability hazards as much as feasible and practical.

Standard permit conditions and BMPs important for reducing impacts from geohazards include:

- Preparation and implementation of a Stabilization, Rehabilitation, and Reclamation Plan;
- Compliance with ADNR Dam Safety requirements through certificates of approval to construct and operate dams to include preparation of Emergency Action Plans and completion of a Failure Modes Effects Analysis (FMEA); and
- Development of Blasting Plans.

Additional measures are being considered by the Corps and Cooperating agencies and are further assessed in Chapter 5, Impact Avoidance, Minimization, and Mitigation (Section 5.5 and Section 5.7). Examples of additional measures being considered that are applicable to this resource include:

- Make the Emergency Action Plan for the tailings dam available to the public to review. Require a communication and alert system to be in place that is sufficient to warn people in Crooked Creek and boaters on the Kuskokwim near Crooked Creek of the potential need to move out of the area;
- Include detailed contingencies to mitigate the risk of ice damage and liner leakage in the TSF in an updated tailings deposition plan during final design; and
- Implement pertinent Best Applicable Practice (BAP) recommendations from the Mount Polley review panel for the tailings storage facility dam, design and tailings management, including participation in formalized tailings management program with audit functions, declaration of Quantitative Performance Objectives (QPOs) for tailings facility design and management, and use of independent tailings review boards.



**Table 3.3-4: Summary of Impacts<sup>1</sup> of Alternative 2 for Geohazards by Project Component**

Assessment Criteria					
Project Component	Impact Type/Facility	Magnitude or Intensity	Duration	Extent or Scope	Context
Mine Site	Earthquakes				
	TSF Dam, Water Dams, Stockpiles, Plants, Tanks, Pit-Operations	Intensity of impacts would vary. Ground shaking or changes in resource character may not be measurable or noticeable. Noticeable changes in resource character may occur, but design is adequate for expected range of geohazard conditions.	Duration of impacts would vary. Easily repairable damage would be infrequent and would return to pre-event levels at the completion of repairs. Pit wall failure would result in chronic effects and the resource would not be anticipated to return to previous levels even if actions that caused the impacts were to cease.	Impacts would be limited geographically to discrete portions of the Project Area.	Context of impacts would vary; dams are governed by regulation.
	WRF	Low probability <sup>1</sup> of acute or obvious changes in the resource character (lower lifts deform, if deep ice-rich soils present).			
	Pit-Closure	Low probability of acute or obvious changes in the resource character (wall failure in post-Closure).			
	Slope Stability				
	TSF Dam, WRF, FWDs, Stockpiles, Plants, Tanks, Pit-Operations	Intensity of impacts would range from unmeasurable (snow slides) to noticeable changes in the resource character. Design is adequate for expected range of geohazard conditions.	Duration of impacts would range from infrequent throughout the life of the mine to chronic effects that result in the resource not anticipated to return to previous levels even if actions that caused the impacts were to cease (pit wall failure).	Same as above.	Same as above.
	Lower CWD	Low probability of acute or obvious changes in the resource character (landslide activation during construction).			
	Pit-Closure	Low probability of acute or obvious changes in the resource character (pit crest settlement and overtopping).			

**Table 3.3-4: Summary of Impacts<sup>1</sup> of Alternative 2 for Geohazards by Project Component**

Assessment Criteria					
Project Component	Impact Type/Facility	Magnitude or Intensity	Duration	Extent or Scope	Context
	<b>Other Geohazards (Dam Seepage and Ice Hazards)</b>				
	TSF Dam, Water Dams	Noticeable changes in the resource character. Design expected to be adequate to mitigate seepage and ice hazards with additional bedrock testing in final design and contingencies for protecting the TSF liner.	Duration of impacts would vary. Water dams would be removed at closure and the resource would return to pre-activity levels after actions causing impacts were to cease. TSF dam seepage would result in chronic effects with the resource not anticipated to return to previous levels even if actions that caused the impacts were to cease.	Same as above.	Same as above.
<b>Transportation Corridor</b>	<b>Earthquakes</b>				
	Roads, Bridges, Docks, Tanks	Intensity of impacts would range from unmeasurable (minor ground shaking) to noticeable changes in the resource character. Design is adequate for expected range of geohazard conditions.	Duration would range from infrequent to impacts lasting through the life of the mine, depending on repair time.	Same as above.	Same as above.
	<b>Slope Stability</b>				
	Roads, Bridges, Docks, Tanks	Intensity of impacts would range from unmeasurable (debris slides at material sites) to noticeable changes in the resource character. Design is adequate for expected range of geohazard conditions.	Duration would range from infrequent (construction slope mitigation) to impacts lasting through the life of the mine (material site Rockfall).	Same as above.	Same as above.
	<b>Other Geohazards (Tsunamis, Volcanoes)</b>				

**Table 3.3-4: Summary of Impacts<sup>1</sup> of Alternative 2 for Geohazards by Project Component**

Assessment Criteria					
Project Component	Impact Type/Facility	Magnitude or Intensity	Duration	Extent or Scope	Context
	Roads, Bridges, Docks, Tanks	Intensity of impacts would range from unmeasurable (tsunami effects at Bethel dock) to noticeable changes in the resource character (planned mitigation minimizes ashfall or tsunami disrupting Dutch Harbor fuel operations).	Duration would range from infrequent (ashfall disruption) to impacts lasting through the life of the mine (tank farm repair).	Same as above.	Typical geologic phenomena.
Pipeline	<b>Earthquakes</b>				
	Pipeline, Associated Facilities	Intensity of impacts would range from unmeasurable or unnoticeable (ground shaking) to noticeable changes in the resource character (fault crossing design withstands lateral displacement).	Impacts would be infrequent and expected to return to pre-event levels at the completion of repair activities. Damage would likely be reparable within days to months.	Same as above.	Context of impacts would vary; pipeline design is governed by regulation.
	<b>Slope Stability</b>				
	Pipeline, Associated Facilities	Intensity of impacts would range from unmeasurable or unnoticeable (minor sloughing) to noticeable changes in the resource character (site-specific design in high landslide hazard locations adequate for conditions).	<b>Temporary</b> Duration would range from infrequent impacts (ROW damage reparable within days to months) to impacts lasting from the end of the Construction Phase and through the life of the mine (active debris flow repairs over life of project).	Same as above.	Same as above.
<b>Other Geohazards (HDD Frac-Out, Tsunamis, Volcanoes)</b>					

**Table 3.3-4: Summary of Impacts<sup>1</sup> of Alternative 2 for Geohazards by Project Component**

Assessment Criteria					
Project Component	Impact Type/Facility	Magnitude or Intensity	Duration	Extent or Scope	Context
	Pipeline, ROW, Roads, Airstrips, Pads	Intensity of impacts would range from unmeasurable or unnoticeable (tsunami at Beluga barge landing) to noticeable changes in the resource character (planned mitigation minimizes ashfall disruption at compressor station).	Impacts would be infrequent but not longer than the span of the Construction Phase and effects from damage would be expected to return to pre-activity levels at the completion of the activity (several days of ashfall interruption or frac-out control).	Extent or scope would range from tsunamis limited to discrete portions of the Project Area to ashfall affecting pipeline and mine operations.	Same as above.
	HDD River Crossings	Low probability of acute or obvious effects (frac-out impacts to river water quality). Resource would be anticipated to return to previous levels if actions that caused the impacts were to cease.			

**Notes:**

1 "Low probability" = unlikely but plausible over project life, not a worst-case scenario (see 3.3.2.4, Introduction).

2 The expected impacts accounts for impact-reducing design features proposed by Donlin Gold and Standard Permit Conditions and BMPs that would be required. It does not account for additional mitigation measures being considered.



### 3.3.3.3 ALTERNATIVE 3A – REDUCED DIESEL BARGING: LNG-POWERED HAUL TRUCKS

#### 3.3.3.3.1 EARTHQUAKES

##### Mine Site

Effects from earthquakes at the Mine Site under Alternative 3A would mostly be the same as those described for Alternative 2, with the exception that the LNG plant would need to be designed to resist seismic hazards such as ground shaking. It is reasonable to assume that such design would be conducted during the final engineering phase of the project. Because the LNG plant would be a small addition to the overall number of major structures at the mine site which seismic hazards could have an effect on, the intensity, duration, extent, and context of effects would be the same as described for Alternative 2.

##### Transportation Corridor

Effects from earthquakes under Alternative 3A for Transportation Corridor facilities would be similar to those described for Alternative 2, except for a reduction in port usage that may reduce the need for consideration of seismic hazards in design. Because the Dutch Harbor and Bethel ports would not require as much expansion (if any) or fuel storage under Alternative 3A, there may not be a need for design of storage tanks and dock structures to resist seismic hazards under this alternative. While the total number of transportation structures would be reduced under this alternative, the range of seismic effects on remaining facilities (e.g., roads, bridges, some tanks and docks at Angyaruaq [Jungjuk] and Bethel) would be the same as Alternative 2, and there would be no difference in the intensity, duration, extent, and context of impacts as compared to Alternative 2. Any actions that would occur at Dutch Harbor or the Port of Bethel at the Bethel Yard Dock are not part of the proposed action, and are considered connected actions (see Section 1.2.1, Connected Actions, in Chapter 1, Project Introduction and Purpose and Need).

##### Pipeline

Effects from seismic hazards under Alternative 3A would be the same as those described for Alternative 2 for the pipeline. There would be no difference in earthquake effects or seismic design requirements for addressing these geohazards under Alternative 3A.

#### 3.3.3.3.2 SLOPE STABILITY

##### Mine Site

The LNG plant under Alternative 3A would be sited in the plant area of the Mine Site, and likely constructed on engineered fill placed on the leveled shallow bedrock ridge in this area. As such, the likelihood of impacts caused by downslope movement from natural geomorphic processes and man-made causes would be low, although fill slopes on the sides of the ridge could experience some sloughing. These potential effects would be the same as those described for the plant area facilities under Alternative 2, and are expected to be largely controlled through standard BMPs and ESC measures.

Because the LNG plant would be a small addition to the overall number of major structures at the Mine Site which slope stability hazards could have an effect on, the intensity, duration, extent, and context of impacts would be the same as Alternative 2.

### Transportation Corridor

Effects from slope stability hazards under Alternative 3A for Transportation Corridor facilities would be mostly the same as those described for Alternative 2, except for minor changes associated with a reduction in port usage. Because the Dutch Harbor port would not require as much expansion (if any) or fuel storage under Alternative 3A, there may not be a concern regarding steep coastal cliffs at potential port sites around Dutch Harbor. While the slope stability hazards would be slightly less under this alternative, the range of effects on remaining facilities (e.g., roads, material sites) would be the same as Alternative 2, the overall intensity, duration, extent, and context of impacts would be the same as Alternative 2.

### Pipeline

Effects from landslides and avalanches under Alternative 3A would be the same as those described for Alternative 2 for the pipeline. There would be no difference in slope stability effects and related design requirements for addressing these geohazards under Alternative 3A.

#### **3.3.3.3.3 OTHER GEOHAZARDS**

Effects from dam seepage, tsunamis, volcanoes, and HDD river crossings under Alternative 3A would be the same as those described for Alternative 2 for the three project components. There would be no difference in direct or indirect effects or related design and mitigation for addressing these geohazards as compared to Alternative 2.

#### **3.3.3.3.4 SUMMARY OF ALTERNATIVE 3A IMPACTS**

Effects from earthquakes, slope instability, and other geohazards at the Mine Site under Alternative 3A would be the same as Alternative 2, with the exception that the LNG plant would need to be designed to resist seismic hazards, such as ground shaking, to minimize potential effects. The overall intensity, duration, extent, and context of impacts would be the same as Alternative 2.

Effects from geohazards on Transportation Corridor facilities under Alternative 3A would be the similar to those described for Alternative 2. While there would be a reduction in transportation facilities (port fuel storage) requiring consideration of seismic effects in design and slightly less slope stability concern, the changes would be relatively small, and the intensity, duration, extent, and context of impacts would be the same as Alternative 2.

Effects from geohazards along the pipeline under Alternative 3A would be the same as those described for Alternative 2. Impacts associated with climate change would also be the same as those discussed for Alternative 2.

The design features, standard permit conditions, and BMPs most important for reducing impacts from geohazards are described under Alternative 2. Examples of additional measures being considered that are applicable to this resource are listed under Alternative 2.

#### 3.3.3.4 ALTERNATIVE 3B – REDUCED DIESEL BARGING: DIESEL PIPELINE

Two options to Alternative 3B have been added based on Draft EIS comments from agencies and the public:

- **Port MacKenzie Option:** The Port MacKenzie Option would utilize the existing Port MacKenzie facility to receive and unload diesel tankers instead of the Tyonek facility considered under Alternative 3B. A pumping station and tank farm of similar size to the Tyonek conceptual design would be provided at Port MacKenzie. A pipeline would extend northwest from Port MacKenzie, route around the Susitna Flats State Game Refuge, cross the Little Susitna and Susitna rivers, and connect with the Alternative 3B alignment at approximately MP 28. In this option, there would be no improvements to the existing Tyonek dock; a pumping station and tank farm would not be constructed near Tyonek; and the pipeline from the Tyonek tank farm considered under Alternative 3B to MP 28 would not be constructed.
- **Collocated Natural Gas and Diesel Pipeline Option:** The Collocated Natural Gas and Diesel Pipeline Option (Collocated Pipeline Option) would add the 14-inch-diameter natural gas pipeline proposed under Alternative 2 to Alternative 3B. Under this option, the power plant would operate primarily on natural gas instead of diesel as proposed under Alternative 3B. The diesel pipeline would deliver the diesel that would be supplied using river barges under Alternative 2 and because it would not be supplying the power plant, could be reduced to an 8-inch-diameter pipeline. The two pipelines would be constructed in a single trench that would be slightly wider than proposed under either Alternative 2 or Alternative 3B and the work space would be five feet wider. The permanent pipeline ROW would be approximately two feet wider. This option could be configured with either the Tyonek or Port MacKenzie dock options.

##### 3.3.3.4.1 EARTHQUAKES

###### Mine Site

Effects from seismic hazards under Alternative 3B would be the same as those described for Alternative 2 for the Mine Site. There would be no difference in potential effects or related design requirements for addressing these hazards as compared to Alternative 2.

###### Transportation Corridor

Effects from seismic hazards under Alternative 3B for Transportation Corridor facilities would be similar to those described for Alternative 2, except for the expansion of the Tyonek dock (Alternative 3B or Alternative 3B Collocated Natural Gas and Diesel Pipeline Option), which would require consideration of seismic hazards in design. Ground shaking in the Tyonek area would be similar to that described for the eastern portion of the pipeline under Alternative 2 (Section 3.3.2.3.2). The risk of liquefaction on the seafloor at Tyonek is considered moderate to high for these deposits (Section 3.3.3.2.1), depending on the thickness of recent fine-grained marine sediments and possible presence of coarse material in this area (described below under Section 3.3.3.4.3, Other Geohazards). Because the tanker berth and pile support system would be designed to accommodate seismic risk (Michael Baker Jr. Inc. 2013b), and would likely consider both ground shaking and liquefaction risk, the level of intensity of effects to this structure would be medium (design expected to be adequate for geohazard), assuming that

additional geotechnical data is collected in final design for this structure to support seismic analysis.

While the total number of port sites that could experience earthquake effects under Alternative 3B and options would increase under this alternative, the increase would be relatively small compared to range of these effects on all transportation facilities, including roads, bridges, and other port sites. Thus, the intensity, duration, extent, and context of impacts would be the same as Alternative 2.

### **Diesel Pipeline**

Effects from seismic hazards under Alternative 3B for the pipeline would be mostly the same as those described for Alternative 2, except for 19 to 20 miles of additional pipeline, depending on selected option, that could experience ground shaking and liquefaction effects in an earthquake, and the addition of a Tyonek or Port MacKenzie tank farm and pumping facility, which would require consideration of seismic hazards in design. Moderate to high liquefaction potential and estimates of lateral spreading for the lower Cook Inlet area, as well as typical control measures for these geohazards described under Alternative 2 (Section 3.3.3.2.1, Pipeline) are likely to be similar for the Beluga-to-Tyonek pipeline segment under Alternative 3B. Thus, the effects from liquefaction and related geohazards on Alternative 3B are expected to be the same as Alternative 2.

Effects of ground shaking and liquefaction on the Tyonek or Port MacKenzie tank farm (depending on selected option) would be similar to those described under Alternative 2 for these structures (Section 3.3.3.2.1, Transportation Facilities), assuming that seismic design would be evaluated in the final engineering phase of the project. Mitigation against liquefaction and settlement effects on tanks can be partly handled through siting in upland locations, which is the case for the tank farm (Chapter 2, Figure 2.3-39).

Surface fault rupture would be the same under Alternative 3B as Alternative 2, because the fault crossings are both located north of Beluga. Unlike Alternative 2, however, ground shaking and surface fault rupture under Alternative 3B have the potential to produce a diesel fuel spill either from the pipeline or tank farm. These effects are described in Section 3.24, Spill Risk.

The intensity of impacts from an earthquake on the pipeline and associated structures under Alternative 3B are expected to range from liquefaction effects that may or may not be noticeable to noticeable changes in the resource character (e.g., fault crossing design adequate to withstand earthquake lateral displacement). It is reasonable to assume that seismic effects at a Tyonek or Port MacKenzie tank farm would be reduced in intensity with seismic design in the final engineering phase prior to construction. The duration, extent, and context of impacts on the pipeline under Alternative 3B would be the same as Alternative 2.

### **3.3.3.4.2 SLOPE STABILITY**

#### **Mine Site**

Effects from landslides and static stability of man-made structures at the Mine Site under Alternative 3B would be the same as those described for Alternative 2. There would be no difference in potential effects or related design requirements for addressing these geohazards as compared to Alternative 2.



### Transportation Corridor

Coastal bluff instability is known to occur along the shoreline in the area of the Tyonek dock and barge landing, due to undercutting of bluffs composed of unconsolidated glacial deposits from coastal waves during very high tides and storm events, and along Tyonek Creek during periods of high flow. While the Tyonek dock and tank farm would be located sufficiently east of the Tyonek Creek to avoid its slopes, the coastal slope beneath the pipeline from the shoreline to the tank farm rises rapidly from sea level to 200 feet within ¼ mile. Part of this slope appears to be previously disturbed and laid back (Chapter 2, Figure 2.3-39). The tank farm would be sited sufficiently inland to avoid this slope. The effects of slope instability on this part of the pipeline would be similar to those described under Alternative 2, (Section 3.3.3.2.2, Pipeline), and represent a small additional concern compared to overall slope stability concerns for the pipeline in the Alaska Range. Planned BMPs and ESC measures described in Section 3.2, Soils, would likely be effective in addressing the coastal bluff issues in the Tyonek area.

### Diesel Pipeline

Slope stability concerns for the remaining sections of the pipeline between Beluga and Tyonek or MP 28 and Port MacKenzie that are different under this alternative compared to Alternative 2, would be limited to the steep slopes of open-cut river crossings at the Chuitna River and Three-mile Creek, and cut slopes of material sites, as the terrain is relatively flat in this area. Three additional airstrips required under Alternative 3B for oil spill response (OSR) purposes would probably be sited to avoid steep slopes to the extent possible, but could experience minor sloughing of cut and fill slopes. Like the Tyonek coastal bluff, planned BMPs and ESC measures would likely be effective in addressing riverbank slope stability issues and minor sloughing of cuts and fills.

#### **3.3.3.4.3 OTHER GEOHAZARDS**

### Mine Site – Dam Seepage

Effects from seepage hazards at constructed dams at the Mine Site under Alternative 3B would be the same as those described for Alternative 2. There would be no difference in potential effects or related design requirements for addressing these geohazards as compared to Alternative 2.

### Transportation Corridor – Tsunamis, Volcanoes, and Seafloor Geohazards

Effects from tsunamis and volcanoes under Alternative 3B for Transportation Corridor facilities would be similar to those described for Alternative 2, except for the addition of the Tyonek dock, which should consider potential impacts from tsunamis in its design. Tsunami potential at Tyonek would be the same as that described for Beluga (Section 3.3.2.3.3, Coastal Hazards and Tsunamis). The predicted wave heights are small compared to tidal range in this area, and are likely to cause less damage to the dock or barge landing site than sea ice (Section 3.5, Surface Water Hydrology). The intensity of impacts from tsunamis on this structure would vary from unmeasurable or unnoticeable effects to noticeable effects within design limits for other geohazards.

Boulders and cobbles have been noted on the seafloor in the Tyonek area and may be present in the subsurface (e.g., Donlin Gold 2013d). Boulders would need to be removed from the seafloor

prior to dock expansion as they could affect pile driving activities, potentially affecting construction schedules and dock stability, and could represent a hazard to ships and mooring activities during Operations. It is reasonable to assume that additional geotechnical investigation would take place during final design prior to construction, and that this would reduce the risk of these geohazards.

### **Diesel Pipeline – HDD Frac-out, Tsunamis, Volcanoes**

Depending on selected option, Alternative 3B includes an HDD crossing of the Beluga River upstream of its discharge point into Cook Inlet north of Tyonek (Alternative 3B and Alternative 3B Collocated Natural Gas and Diesel Pipeline Option) or the Susitna River (Alternative 3B Port MacKenzie Option). The suitability of subsurface conditions for HDD at the proposed crossing points are currently unknown as geotechnical drilling has not been conducted at these sites. It is possible that deep coarse-grained glacial deposits are present that would represent a high potential for frac-out and preclude the use of HDD methods at these locations, or that competent clay layers or Tertiary-age formations are present at suitable depths that would represent a low to moderate potential for loss of drilling fluid during HDD pipeline installation activities. Frac-out is of less concern at the proposed Susitna River crossing point due to the reported presence of bedrock in close proximity to the river edges. Additional geotechnical drilling at the proposed crossing locations would likely be conducted during final design to reduce this potential hazard, and if suitable, to include potential frac-out risk in a contingency plan along with other HDD sites.

Effects from tsunamis and volcanoes under Alternative 3B for the pipeline would be mostly the same as those described for Alternative 2, except for the additional airstrips and air travel needed for OSR, and effects on the Tyonek or Port MacKenzie tank farm and pumping station, which should consider ashfall in planning. The effects of ashfall on the pumping station would be similar to effects and mitigation described for the compressor station (Section 3.3.3.2.3, Other Geohazards, Pipeline). Ashfall effects on air travel would be similar to those described under Alternative 2, except that disruption of air travel under Alternative 3B and options could also affect OSR maintenance activities at remote airstrips and spill response in the event that a diesel spill occurs at the same time. The intensity of volcanic effects on the pipeline and appurtenant structures under Alternative 3B and options is expected to range from mostly unmeasurable or unnoticeable effects, to noticeable effects, assuming that most effects can be mitigated through advance planning.

#### **3.3.3.4.4 SUMMARY OF ALTERNATIVE 3B IMPACTS**

Effects from earthquakes, slope instability, and other geohazards at the Mine Site under Alternative 3B would be the same as those described for Alternative 2. Impacts associated with climate change would also be the same as those discussed for Alternative 2.

As discussed under Alternative 2, effects from earthquakes, slope instability, and other geohazards on transportation facilities under Alternative 3B would mostly range from unmeasurable or unnoticeable changes, to noticeable changes with the consideration that design is adequate for the expected range of geohazard conditions. There would be an increase in the number of port sites requiring consideration in final design for seismic risk and coastal bluff stability, but assuming additional design would take place in the final engineering phase, the increase would be relatively small compared to the range of effects on all transportation

facilities (roads, bridges, other ports), such that the range of the intensity, duration, extent, and context would be the same as Alternative 2.

For the pipeline, effects from earthquakes, slope instability, and other geohazards under Alternative 3B and options would mostly range from unmeasurable or unnoticeable changes, to noticeable changes with the consideration that design is adequate for the expected range of geohazard conditions. There would be an increase in seismic risk due to the increase in tank farm facilities and pipeline length; an increase in slope stability concerns at 2 additional open-cut steep-sided river crossings, 3 additional airstrips (minor sloughing), and 5 additional material sites (debris slides); and an increase in ashfall concerns at the Tyonek pumping station. Assuming additional design and planning would take place in the final engineering phase to mitigate these hazards, the increase would be relatively small compared to the range of effects on all pipeline facilities, such that the range of impacts ratings for these facilities would be the same as Alternative 2. There could be additional low-probability/acute or obvious effects on river water quality in the event of HDD frac-out at the Beluga River or Susitna River crossing, depending on selected option, which can be mitigated through advance planning (Chapter 5, Impact Avoidance, Minimization, and Mitigation).

The design features, standard permit conditions, and BMPs most important for reducing impacts from geohazards are described under Alternative 2. Examples of additional measures being considered that are applicable to this resource are listed under Alternative 2.

### 3.3.3.5 ALTERNATIVE 4 – BIRCH TREE CROSSING (BTC) PORT

#### 3.3.3.5.1 EARTHQUAKES

##### Mine Site

Effects from seismic hazards under Alternative 4 would be the same as those described for Alternative 2 for the Mine Site component. There would be no difference in potential effects or related design requirements for addressing these geohazards as compared to Alternative 2.

##### Transportation Corridor

Effects from seismic hazards for Transportation Corridor facilities under Alternative 4 would be mostly the same as those described for Alternative 2, except for the longer BTC Road, which would require consideration of seismic hazards in bridge design. As there would be 8 bridges along the BTC Road compared to 6 for the mine access road under Alternative 2, this represents a slight increase in seismic hazards effects under Alternative 4. The types of seismic effects along the road and bridges, ADOT&PF bridge guidelines, and planned mitigation (seismic design) would be the same as described for Alternative 2.

##### Pipeline

Effects from seismic hazards under Alternative 4 would be the same as those described for Alternative 2 for the Pipeline. There would be no difference in potential effects or related design requirements for addressing these geohazards as compared to Alternative 2.

### 3.3.3.5.2 SLOPE STABILITY

#### Mine Site

Effects from landslides and avalanches under Alternative 4 would be the same as those described for Alternative 2 for the Mine Site component. There would be no difference in potential effects or related design requirements for addressing these geohazards as compared to Alternative 2.

#### Transportation Corridor

Slope conditions for the northeastern part of the BTC Road from the Mine Site to Juninggulra Mountain are the same as those described for Alternative 2. From Juninggulra Mountain to the BTC Port site on the Kuskokwim River, the route traverses moderate to steep and gentle to moderate slopes across hilltop ridges and narrow stream drainages (Section 3.3.2.2.2). The length of road traversing moderate to steep slopes which would require sidehill cut and fill construction comprises roughly half of the total BTC Road length (RECON 2007a) or about three times the length of moderate to steep slopes along the mine access road under Alternative 2. Potential slope stability effects at material sites would also be increased under Alternative 4, as there would be roughly three times as many material sites for the BTC Road than the Alternative 2 mine access road.

The types of disturbance caused by downslope movement during Construction, Operations, and Closure along the BTC Road under Alternative 4 would be similar to those described for the mine access road under Alternative 2 (Section 3.3.3.2.2), and could include sloughing, landslides, and rockfall along the road and at material sites. The intensity, duration, extent, and context of effects would be the same as described for Alternative 2.

#### Pipeline

Effects from landslides and avalanches under Alternative 4 would be the same as those described for Alternative 2 for the Pipeline. There would be no difference in potential effects or related design requirements for addressing these geohazards as compared to Alternative 2.

### 3.3.3.5.3 OTHER GEOHAZARDS

Effects from dam seepage, tsunamis, volcanoes, and HDD river crossings under Alternative 4 would be the same as those described for Alternative 2 for the project components. These geohazards would have little to no effect on the BTC Road and Port that would change under Alternative 4, and there would be no difference in potential direct and indirect effects or related design requirements for addressing these geohazards in other areas of the project.

### 3.3.3.5.4 SUMMARY OF ALTERNATIVE 4 IMPACTS

Effects from earthquakes, slope instability, and other geohazards at the Mine Site under Alternative 4 would be the same as those described for Alternative 2. Impacts associated with climate change would also be the same as those discussed for Alternative 2.

Because of the longer BTC road, there would be a 33 percent increase in number of bridges requiring seismic design (8 under Alternative 4 vs. 6 under Alternative 2), about three times the road length with moderate to steep side slopes, and three times the number of material sites



with rock slide potential. Though there would be an increase in seismic and slope stability issues for the road, assuming additional design would take place in the final engineering phase, the intensity, duration, extent, and context of effects would be the same as described for Alternative 2.

Effects from earthquakes, slope instability, and other geohazards on the pipeline under Alternative 4 would be the same as those described for Alternative 2.

The design features, standard permit conditions, and BMPs most important for reducing impacts from geohazards are described under Alternative 2. Examples of additional measures being considered that are applicable to this resource are listed under Alternative 2.

### 3.3.3.6 ALTERNATIVE 5A – DRY STACK TAILINGS

This alternative includes two options:

- **Unlined Option:** The tailings storage facility (TSF) would not be lined with a linear low-density polyethylene (LLDPE) liner. The area would be cleared and grubbed and an underdrain system placed in the major tributaries under the TSF and operating pond to intercept groundwater base flows and infiltration through the dry stack tailings (DST) and convey it to a Seepage Recovery System (SRS). Water collecting in the SRS pond would be pumped to the operating pond, lower contact water dam (CWD), or directly to the processing plant for use in process.
- **Lined Option:** The DST would be underlain by a pumped overdrain layer throughout the footprint, with an impermeable LLDPE liner below. The rock underdrain and foundation preparation would be completed in the same manner as the Unlined Option.

#### 3.3.3.6.1 EARTHQUAKES

##### Mine Site

##### *Construction and Operations*

Differences between Alternative 5A (unlined or lined options) and Alternative 2 that are pertinent to earthquake geohazards and seismic dam safety elements in Construction and Operations phases include the following:

- A dry stack tailings pile and permanent upper dam would be located in the Anaconda Creek drainage above the main dam, which would be used in Alternative 5A for an operating pond and removed at closure.
- The main operating pond dam construction under Alternative 5A would be similar to the TSF dam in Alternative 2. The main dam of Alternative 5A would be constructed to contain a fully lined operating pond, holding water from the dewatering and filtration of the tailings, clay-rich off-specification tailings, and excess water from site water balance management.
- The disturbed footprint of the whole TSF facility under the two alternatives would be very similar (2,500 acres under Alternative 5A vs. 2,450 acres under Alternative 2).

- The dry stack would hold a smaller tailings volume of 239,200 acre-feet than the TSF impoundment under Alternative 2 (334,300 acre-feet), primarily because they would contain less moisture.
- Alternative 5A would place the tailings further upstream from the main dam behind one large upper dam that initially spans across both the north and south forks of Anaconda Creek and the ridge that separates the 2 tributaries, rather than 2 FWDDs initially in Alternative 2 that are removed and the area merged into one TSF facility over the life of the mine.
- Under the unlined option, the dry stack upstream from the operating pond and upper dam would not include a liner below the impoundment footprint. Under the lined option, the dry stack would be fully lined with LLDPE beneath the tailings and a pumped rock overdrain layer over the liner. In both unlined and lined options, drainage through the stack would not be impeded, thereby improving stability of the stack (BGC 2011a, 2013g, 2014a, 2015d).
- The main Operating Pond dam would be constructed to an elevation of 751 feet, which is lower than the TSF dam for Alternative 2, and includes an emergency spillway on the left abutment. The total height of the main Operating Pond dam from downstream toe to crest would be 367 feet, which is 97 feet shorter (roughly 20 percent shorter) than the TSF dam under Alternative 2.
- The crest elevation of the upper Operating Pond dam would be the same as the main Operating Pond dam (751 feet). The total height of the upper dam would be 218 and 206 feet for the north and south halves, respectively (BGC 2014a).
- Similar to the proposed action, seepage through the dry stack and main dam under the unlined option would be captured and contained by underdrains and a SRS located downstream from the main dam. Under the lined option, pumped dry stack seepage from the overdrain would also report to the SRS (BGC 2015d).

No seismic stability analysis has been conducted specifically for Alternative 5A. However, the design and construction of the two dams would be essentially the same as that of the main TSF dam under Alternative 2, including rockfill construction on bedrock dam filter gradation designed for grain size compatibility between adjacent materials (BGC 2013g, 2014a). Thus, the seismic stability analyses conducted for Alternative 2 (Section 3.3.3.2.1) would also apply to Alternative 5A, and the intensity of seismic effects on the dams is expected to be the same as Alternative 2.

Ground shaking and liquefaction effects in the part of the dry stack that rises above the upper dam could potentially cause slumping or mass wasting over the dam in a large earthquake. Liquefaction potential of the dry stack is related to moisture content and porewater in the dry stack. Alternative 5A would require dewatering, filtering, and thickening of the tailings; as compared to the slurry mixture of the proposed action conveyed by large hoses with spigots into the fully lined TSF. Processed filter cake will have a reduced moisture content of 19.7 percent by mass to accommodate planned deposition and compaction practices (BGC 2013g, 2014a). The dewatered tailings would undergo an 8-stage filtration process and thickening utilizing reagents then captured in a surge tank at the filter plant before being transferred to haul trucks. During winter months, the haul truck beds would be heated to keep the dewatered tailings from freezing. The dry stack would be built from truckloads of dewatered tailings

placed within the impoundment then spread and compacted by heavy equipment. The results of the seepage analyses conducted by BGC (2015d) show that, irrespective of the liner system, the tailings remain at a moisture content below saturation under both unlined and lined options, provided that surface drainage controls prevent ponding of surface water during Operations.

The front 1,000 feet (west edge) of the dry stack would be graded towards the operating pond with a relatively low slope angle of 5H:1V. No seismic stability analysis of the stack has been conducted for the part that rises above the upper dam. In the event of a large earthquake during winter months, stack instability could occur from alternating layers of frozen and thawed tailings or entrained snow. Inadequate compaction would impact the strength and seismic stability of the stack and increase the potential for liquefaction during an earthquake. Some mounding of the water table is to be expected beneath such a large landform, but it would be minimized through the use of diversion channels, underdrains (in the case of the unlined option), and the pumped overdrain (in the lined option). As such, the water table is unlikely to reach the top of the upper dam.

Because of reduced moisture content from tailings dewatering, compaction, low sloping stack face, and drainage features to handle seepage, the likelihood of liquefaction potential in the dry stack above the dam is expected to be low under both options, although there is a low probability of greater effects such as slumping into the operating pond causing discharge of untreated pond water through the spillway. It is reasonable to assume that seismic stability analysis of the dry stack would take place during the final engineering phase of the project to further evaluate this potential effect, and the dry stack design adjusted if needed.

### *Closure*

Closure of the dry stack tailings pile would be simplified and require less time than the proposed action. Dry stack tailings would have a moisture content of 20 percent, and behave like a solid, resistant to flow under gravity. Therefore, Alternative 5A would form a permanent seismically stable landform shortly after closure.

The operating pond water and liner would be removed once all off-spec tailings are pumped to the open pit, and the main dam and downstream face of the upper tailings dam are regraded to 3H:1V slopes, eliminating seismic risk associated with this structure.

The dry stack landform would be situated higher in the valley and extend to a greater final elevation of 950 feet than the remaining landform under Alternative 2 (830 feet), which would cover the entire TSF footprint. Reclamation of the dry stack under both unlined and lined options would include a cover system that incorporates an impermeable LLDPE liner to reduce dry stack infiltration. Reclamation of the upper dam would include placement of overburden and slope flattening that would improve the seismic stability of the entire structure in post-Closure.

### Transportation Corridor and Pipeline

Earthquake and seismic dam safety for the Transportation Corridor and Pipeline components under Alternative 5A would be the same as described under Alternative 2. There would be no difference in potential effects or related design requirements for addressing seismic hazards for these project components.

### 3.3.3.6.2 SLOPE STABILITY

#### Mine Site

The static stability of the main dam containing the operating pond under Alternative 5A, and of the upper dam holding the dry stack pile, would be similar to analyses described under Alternative 2 (Section 3.3.3.2), as the design and construction of these dams are the same as the main TSF dam under Alternative 2.

Similar to seismic stability described above (Section 3.3.3.6.1), static stability of the portion of the dry stack that extends above the upper dam depends on moisture content in the tailings, elevation of the water table, and slope of the downstream stack face. The dry stack would be built from truckloads of dewatered tailings placed within the impoundment then spread and compacted by heavy equipment. The front 1,000 feet (west edge) would be graded toward the operating pond with a relatively low slope angle of 5H:1V. Although some mounding of the water table is expected, it is predicted to remain below the height of the upper dam under both unlined and lined options. The pumped overdrain, however, would reduce buildup of saturation at the base of the tailings pile, which would improve overall stability under the lined option.

No static stability analysis has been conducted of the proposed Alternative 5A stack behind the sloped west edge, although it is reasonable to assume this would occur in final design. Stack instability could occur during the winter months from alternating layers of frozen and thawed tailings. Inadequate compaction would impact the strength and stability of the stack. The effect of the frequent application of resin for dust control on pile stability is unknown.

Excavation of some overburden prior to tailings placement (unlined option) or liner installation (lined option) would be required to prevent excessive slope deformation (BGC 2013g, 2014a). Modified underdrains and permanent tailings loading pressure would likely result in comparable permafrost degradation and overburden compaction below the impoundment areas of Alternatives 2 and 5A during Operations and throughout post-Closure (in perpetuity). For this reason, the potential for long-term disturbances from landslides in native materials under Alternative 5A (both options) are expected to be similar to those of the proposed action.

Static stability concerns for the Alternative 5A main dam in post-Closure would be reduced or eliminated due to removal of the main operating pond dam during closure and reclamation. The operating pond and liner would be removed, the main dam breached, and the main dam and downstream face of the upper dam regraded to 3H:1V slopes.

The dry stack landform would be situated higher in the valley and extend to a higher final elevation of 950 feet than the remaining landform under Alternative 2 (830 feet), which would cover the entire TSF footprint. Reclamation of the dry stack would include an LLDPE and soil cover system to reduce infiltration, and reclamation of the upper dam would include placement of overburden and slope flattening, improving static stability of these structures in post-Closure.

The intensity of slope or static stability effects at the TSF under both options of Alternative 5A is expected to be mostly the same as Alternative 2. However, because of the unprecedented size of the dry stack, more complex tailings management and placement, lack of site-specific static stability analysis for the dry stack above the upper dam, and some unknowns such as the effects of polymer on stability, there could be increased difficulty in controlling stability under this



alternative compared to Alternative 2. This may result in intermittent acute or obvious changes in the resource character with geohazards likely to exceed design parameters. It is reasonable to assume that the additional analysis to reduce the intensity of effects would take place during the final engineering phase of the project, and the dry stack design adjusted if needed.

### Transportation Corridor and Pipeline

Landslides and avalanches for the Transportation Corridor and Pipeline components under Alternative 5A would be the same as described under Alternative 2. There would be no difference in potential effects or related design requirements for addressing slope stability hazards for these project components.

### 3.3.3.6.3 OTHER GEOHAZARDS

#### Mine Site

The types of seepage-related geohazards that could affect the internal stability of main and upper dams under Alternative 5A are generally similar to those described under Alternative 2 (Section 3.3.3.2.3). Seepage through and beneath the main and upper dams under Alternative 5A would be similar to the main TSF dam in Alternative 2, as the design and construction of the dams and operating pond impoundment under both alternatives would be the same. Both the dams and operating pond impoundment would be fully lined. Ice hazard effects on the Operating Pond liner would be more likely to occur under Alternative 5A due the lack of a protective tailings beach.

The differences between Alternative 5A unlined and lined options that govern the path that tailings seepage takes at the base of the dry stack would have less of an effect on dam stability than the dam design. Seepage through the unlined dry stack under the unlined option would be captured by underdrains and report to the SRS located downstream from the main dam. Seepage through the dry stack under the lined option would be pumped out of the overdrain layer above the LLDPE liner and report to the SRS. Design measures for minimizing seepage and piping through the dry stack and upper dam under both options include multiple filter zones and LLDPE liner in the dam, dewatering of tailings to within three percent of optimum moisture content to facilitate compaction to a minimum of 90 percent maximum dry density in one foot lifts, construction of diversion channels around the perimeter of the dry stack, and grading and sloping of dry stack surfaces to the south to minimize surface infiltration.

Seepage flow and water balance have been estimated by BGC (2015d, j, k) for Alternative 5A assuming precipitation conditions similar to the TSF, infiltration into the dry stack based on tailings grain size data presented in Paterson & Cooke (2014), and foundation conditions specific to the two options. Under the unlined option, the underdrain beneath the dry stack and operating pond would be designed to convey seepage and baseflow with a factor of safety of 10, and would be wrapped in geotextile to filter out tailings fines from clogging the underdrain. Underdrains would still be used beneath the dry stack liner under the lined option, but would not receive seepage from the dry stack. Seepage flow through the dry stack into either the underdrain (unlined option) or rock overdrain (lined option) would range from 49 to 95 gpm over the life of the mine (BGC 2015d, j). Seepage through the dry stack would be on the lower end of this range under the lined option in the early Operations period, as the LLDPE liner would prevent groundwater wicking into the base of the dry stack, which increases seepage

flow out of the dry stack in the unlined option (BGC 2015d). After about 3 years of operation, seepage through the tailings is expected to stabilize at about the same rate for both options (about 80 gpm).

Average flow through the whole TSF system (including dam seepage, tailings seepage, and baseflows) that reports to the SRS over the life of the mine under both options of Alternative 5A is similar to that of the TSF under Alternative 2, averaging about 700 gpm, of which about 1 percent represents seepage through the main dam and liner, and the remainder is tailings seepage and inflows to the underdrain system (BGC 2015j).

At closure, seepage through the dry stack would decrease under both options over time due to placement of the impermeable cover. This decline is expected to take roughly 150 to 200 years until it reaches the same rate as the TSF under Alternative 2 (18 gpm). The primary difference between the two Alternative 5A options in closure is the pathway that the seepage flow takes: under the unlined option, where seepage reports to the underdrain and could infiltrate groundwater before reaching the SRS; while under the lined option, seepage would be pumped from the overdrain directly to the pit lake. The effects of seepage flow on groundwater flow and groundwater quality under the two options are discussed in Sections 3.6, Groundwater Hydrology, and 3.7, Water Quality, respectively.

Total underdrain water reporting to the SRS in post-Closure under Alternative 5A (both options) is expected to be roughly half that of Alternative 2 (BGC 2015f, 2015j) due to positioning of the dry stack and post-Closure SRS higher in the watershed, which captures less runoff and baseflow.

Based on seepage controls such as dam filter zones, underdrains, and surface water diversion, the intensity of potential seepage impacts under Alternative 5A would mostly result in noticeable changes in the resource character with the consideration that design is adequate for the expected range of geohazard conditions. The differences between the lined and unlined options that govern the path that tailings seepage takes would have less of an effect on dam stability than the design of the dams themselves, which would be the same as Alternative 2.

### Transportation Corridor and Pipeline

Effects from other geohazards (tsunamis, volcanoes, HDD frac-out) for the Transportation Corridor and Pipeline components under Alternative 5A would be the same as described under Alternative 2. There would be no difference in potential effects or related design requirements for addressing these geohazards under Alternative 5A.

#### **3.3.3.6.4 SUMMARY OF ALTERNATIVE 5A IMPACTS**

There could be increased ground shaking, liquefaction, and slope instability effects in the part of the dry stack that rises above the upper dam as compared to the TSF under Alternative 2. There would be reduced seismic and slope stability concerns in closure, as the dry stack would become a stable landform, while the Alternative 2 TSF would not become a stable landform due to continued saturation of tailings. While seepage flow through the dry stack under both Alternative 5A options would be higher than Alternative 2 for about 150 to 200 years after closure, seepage effects on dam stability would be similar to Alternative 2 as the design of the dams and operating pond impoundment are the same as that of the Alternative 2 TSF. There

could be increased ice damage effects on the operating pond liner under Alternative 5A due the lack of a protective tailings beach.

The intensity of impacts would range from unmeasurable or unnoticeable changes (e.g., minor liquefaction in stack that is contained by the upper dam) to noticeable changes with the consideration that the design of operating pond and upper dams adequate to withstand earthquakes and seepage hazards. These impacts take into account that additional analyses would take place during final design to mitigate seismic and slope stability concerns. Low probability, but acute effects described under Alternative 2 for the WRF, pit walls, and Lower CWD (Table 3.3-4) would be the same under Alternative 5A. The duration of effects would range from intermittent instability of the dry stack to dry stack seepage flow that may not return to previous levels even if actions that caused the impacts were to cease.

Effects from earthquakes, slope instability, and other geohazards on Transportation Corridor and the Pipeline under Alternative 5A would be the same as those described for Alternative 2. Impacts associated with climate change would also be the same as those discussed for Alternative 2.

The design features, standard permit conditions, and BMPs most important for reducing impacts from geohazards are described under Alternative 2. Examples of additional measures being considered that are applicable to this resource are listed under Alternative 2.

### **3.3.3.7 ALTERNATIVE 6A – MODIFIED NATURAL GAS PIPELINE ALIGNMENT: DALZELL GORGE ROUTE**

#### **3.3.3.7.1 EARTHQUAKES**

Effects from seismic hazards under Alternative 6A would be similar to those described for Alternative 2 for each of the project components. There would be no difference in potential effects for the Mine Site and Transportation Corridor facilities under Alternative 6A. While there could be an increase in earthquake-triggered landslides on the pipeline under Alternative 6A, these impacts are included in the effects ratings under Slope Stability (Section 3.3.3.7.2). Thus, the intensity, duration, extent, and context of effects from earthquakes on the project under Alternative 6A would be the same as Alternative 2.

#### **3.3.3.7.2 SLOPE STABILITY**

##### **Mine Site and Transportation Corridor**

Landslide geohazards and static stability of man-made structures under Alternative 6A would be the same as those described for Alternative 2 for two of the project components. There would be no difference in potential effects or related design requirements for addressing these geohazards at Mine Site and Transportation Corridor facilities under Alternative 6A.

##### **Pipeline**

Alternative 6A involves a different pipeline route through the mountainous terrain of the Alaska Range that has a greater geotechnical hazard from unstable slopes than along the Alaska Range section of Alternative 2 (SRK 2013b). From Rainy Pass at MP 113 to MP 148, the route traverses steep-sided glacial valleys with unvegetated slopes that would be susceptible to

landslides, rock fall, and creep. A total of 10 high-risk landslide areas were identified over the 15-mile section of this route between MP 118 and MP 133 oriented both perpendicular and parallel to the ROW (CH2MHill 2011d), as compared to high-risk landslides over an 8-mile section of Alternative 2 in the Three-mile Creek/Jones River area. In particular, active landslides occur on both sides of the pipeline corridor at Dalzell Gorge near MP 127, and numerous unstable and active landslides have been identified in Dalzell Creek and Pass Creek valleys. A large landslide scar is also present at MP 136.

Significant slope instability at these locations during Construction and Operations could cause pipeline damage if it is not buried deep enough beneath sliding surfaces, and could require remedial measures or re-routing of the pipeline after construction. It is reasonable to assume that additional geotechnical engineering data collection would be conducted prior to final design of this alternative to mitigate these hazards and modify pipeline design if necessary.

The intensity, duration, extent, and context of effects would be the same as described for Alternative 2, assuming that site-specific design for specific debris flows in the Alaska Range would be completed in final design.

#### 3.3.3.7.3 OTHER GEOHAZARDS

Effects from other geohazards (dam seepage, tsunamis, volcanoes, HDD frac-out) under Alternative 6A would be the same as those described for Alternative 2 for the three project components. While Alternative 6A would have an HDD section in Dalzell Gorge to avoid slope hazards, it would not be located beneath a river and would not present a frac-out risk. In addition, a similar HDD section may be required in the Alaska Range section of Alternative 2 (Three-mile/Jones River area), making these two alternatives comparable in this regard. Thus, there would be no difference in potential direct and indirect effects or related design requirements for addressing these geohazards under Alternative 6A.

#### 3.3.3.7.4 SUMMARY OF ALTERNATIVE 6A IMPACTS

Effects from earthquakes, slope instability, and other geohazards at the Mine Site and on the Transportation Corridor facilities under Alternative 6A would be the same as those described for Alternative 2. Impacts associated with climate change would also be the same as those discussed for Alternative 2.

Under Alternative 6A, there would be roughly double the length of high-risk unstable slopes along the Alaska Range portion of the pipeline route (15 miles under Alternative 6A vs. 8 miles under Alternative 2). The effects of slope stability hazards as well as earthquakes and other geohazards under Alternative 6A (including effects for pipeline sections and associated facilities which do not change under this alternative) would be mostly the same as described for Alternative 2, assuming that site-specific design of high hazard landslides in the Alaska Range would be conducted in final design.

The design features, standard permit conditions, and BMPs most important for reducing impacts from geohazards are described under Alternative 2. Examples of additional measures being considered that are applicable to this resource are listed under Alternative 2.



### 3.3.3.8 IMPACT COMPARISON – ALL ALTERNATIVES

A comparison of impacts between alternatives is presented below in Table 3.3-5. Although there are differences among alternatives in the project components that would affect geohazards, they are relatively small. This is because all alternatives involve excavation, dam construction, road cuts, etc. in a region that is at naturally high risk for geohazards. Therefore geohazard impacts to the project are similar for all the alternatives being considered. Overall there is little difference in the range of impacts of geohazards for the various alternatives, as the scope and scale of the three project components are such that changes to a single mine structure, road, port, or pipeline route result in small changes to overall impacts.

**Table 3.3-5: Comparison of Impacts by Alternative\* for Geohazards**

<b>Impact-causing Project Component</b>	<b>Alternative2 – Proposed Action</b>	<b>Alternative3A – LNG-Powered Haul Trucks</b>	<b>Alternative3B – Diesel Pipeline</b>	<b>Alternative4 – BTC Port</b>	<b>Alternative5A – Dry Stack Tailings (Lined or Unlined)</b>	<b>Alternative6A – Dalzell Gorge Route</b>
<b>Mine Site</b>						
Earthquakes	Intensity of impacts would vary and mostly range from unmeasurable or unnoticeable, to noticeable changes in the resource character. The TSF dam design is considered robust with seismic parameters incorporated into the design; it is extremely unlikely to fail during a major earthquake. There is a low probability of acute or obvious impacts to the WRF and pit walls in post-Closure.	Slight increase in effects for the LNG plant (designed to withstand ground shaking). However, same overall intensity, duration, extent, and context of impacts as Alternative 2.	Same as Alternative 2.	Same as Alternative 2.	Slight increase in effects during Operations (ground shaking effects on dry stack above upper dam); slightly less during closure (shorter duration to stable landform). However, same overall intensity, duration, extent, and context of impacts as Alternative 2.	Same as Alternative 2
Slope Stability	Intensity of impacts would vary and mostly range from unmeasurable or unnoticeable, to noticeable changes in the resource character. Static stability analysis is incorporated into the TSF and water dam design. There is a low probability of acute or obvious effects at Lower CWD (landslide activation) and pit (crest settlement and overtopping).	Same as Alternative 2.			Slight increase in effects during Operations for both Opts.1 and 2 (potential dry stack instability); and slightly more in Opt.1 than Opt.2 (groundwater wicking and saturation at base of stack). However, same overall intensity, duration, extent, and context of impacts as Alternative 2.	

**Table 3.3-5: Comparison of Impacts by Alternative\* for Geohazards**

<b>Impact-causing Project Component</b>	<b>Alternative2 – Proposed Action</b>	<b>Alternative3A – LNG-Powered Haul Trucks</b>	<b>Alternative3B – Diesel Pipeline</b>	<b>Alternative4 – BTC Port</b>	<b>Alternative5A – Dry Stack Tailings (Lined or Unlined)</b>	<b>Alternative6A – Dalzell Gorge Route</b>
Other Geohazards (Dam Seepage and Ice Hazards)	Intensity of impacts would mostly result in noticeable changes to the resource character. Design is adequate for the expected range of geohazard conditions, assuming additional geotechnical investigation in final design and contingencies for ice damage mitigation are effective.				Higher tailings seepage in closure (both options) and higher ice hazards effects on operating pond liner than Alternative 2, but similar seepage effects on dam stability as Alternative 2. However, same overall intensity, duration, extent, and context of impacts as Alternative 2.	
<b>Transportation Corridor</b>						
Earthquakes	Intensity of impacts would range from unmeasurable or unnoticeable, to noticeable changes in the character of the resource. The design is expected to be adequate for the range of geohazard conditions.	Slightly lower risk of effects due to reduction in port fuel tanks. Overall intensity, duration, extent, and context of impacts would be the same as Alternative 2.	Slightly more effects for Tyonek dock (seismic, coastal bluff, seafloor concerns). However, overall intensity, duration, extent, and context of impacts would be the same as Alternative 2.	Slightly increased risk of effects due to two additional bridges (seismic design). However, overall intensity, duration, extent, and context of impacts would be the same as Alternative 2.	Same as Alternative 2.	Same as Alternative 2.

**Table 3.3-5: Comparison of Impacts by Alternative\* for Geohazards**

<b>Impact-causing Project Component</b>	<b>Alternative2 – Proposed Action</b>	<b>Alternative3A – LNG-Powered Haul Trucks</b>	<b>Alternative3B – Diesel Pipeline</b>	<b>Alternative4 – BTC Port</b>	<b>Alternative5A – Dry Stack Tailings (Lined or Unlined)</b>	<b>Alternative6A – Dalzell Gorge Route</b>
Slope Stability	Same as above.			More effects (3 times road length with steep slopes, and 3 times material sites). However, overall intensity, duration, extent, and context of impacts would be the same as Alternative 2.		
Other Geohazards (Tsunamis, Volcanoes)	Same as above.	Same as Alternative 2.		Same as Alternative 2.		
<b>Pipeline</b>						
Earthquakes	Same as above.	Same as Alternative 2.	More effects for tank farm and 19 mi longer pipeline (ground shaking, liquefaction). However, overall intensity, duration, extent, and context of impacts would be the same as Alternative 2.	Same as Alternative 2.	Same as Alternative 2.	More effects (7 mi longer route with high-hazard unstable slopes). However, overall intensity, duration, extent, and context of impacts would be the



**Table 3.3-5: Comparison of Impacts by Alternative\* for Geohazards**

Impact-causing Project Component	Alternative2 – Proposed Action	Alternative3A – LNG-Powered Haul Trucks	Alternative3B – Diesel Pipeline	Alternative4 – BTC Port	Alternative5A – Dry Stack Tailings (Lined or Unlined)	Alternative6A – Dalzell Gorge Route
Slope Stability	Same as above.		More effects (minor sloughing, debris slides) at 2 steep open-cut river crossings, 3 additional airstrips, and 5 additional material sites. However, overall intensity, duration, extent, and context of impacts would be the same as Alternative 2.			same as Alternative 2.
Other Geohazards (HDD frac-out, Tsunamis, Volcanoes)	Intensity of impacts would mostly range from unmeasurable or unnoticeable, to noticeable changes in the character of the resource. There is a low probability of acute or obvious effects at HDDs (frac-out impacts river water quality).		More low probability-acute/obvious effects due to additional Beluga River HDD crossing (frac-out risk unknown). However, overall intensity, duration, extent, and context of impacts would be the same as Alternative 2.			Same as Alternative 2.

**Notes:** \*Alternative 1 (No Action Alternative) is presumed to have no new impacts resulting from geohazards.