4.15 GEOHAZARDS AND SEISMIC CONDITIONS

This section describes potential impacts of seismic and other geologic hazards (geohazards) on project components that could affect the environment. The Environmental Impact Statement (EIS) analysis area for geohazards ranges from the immediate vicinity of the project footprint (e.g., slope instability) to regional areas with geohazards that could affect project facilities from long distances (e.g., earthquakes, volcanoes).

The impact analysis for geologic hazards considered the following factors:

- Magnitude—impacts are assessed based on the magnitude of the impact, as indicated by the anticipated effects of various possible geologic hazard events (e.g., repairable damage to mine features, ground settlement).
- Duration—impacts are assessed based on the project phase during which they are expected to occur (e.g., certain structures removed at closure), and how long repair of potential damage or interruption of activities may last.
- Geographic extent—impacts are assessed based on the location and distribution of occurrence of the expected effects from potential geologic hazard events (e.g., distant earthquake effects on mine site and port structures).
- Potential—impacts are assessed based on the likelihood of a geologic hazard event to occur during and after project development (e.g., based on expected recurrence interval[1] for certain geologic hazards).

The impact analysis incorporates an understanding of the probability of occurrence, and of planned mitigation in the form of planning, design, construction, operations, maintenance, and surveillance that can meaningfully reduce impacts from geohazards through closure and post-closure. Based on Pebble Limited Partnership (PLP) plan documents and engineering reports, planned mitigation methods, described in Chapter 5, Mitigation (e.g., design and monitoring to withstand or detect geohazards), are considered part of the project description, and the impacts analysis includes this understanding. In some cases, planned mitigation may not be specified, but is considered typical or standard engineering practice. In cases where planned mitigation is unknown or unclear and the situation is not commonly addressed, the impact analysis takes the lack of planned mitigation into account.

The review of geohazards and seismic effects on project facilities and the related potential for effects on the environment are based on a conceptual level of design and analysis for critical structures, such as the mine site embankments. Therefore, there are uncertainties regarding the potential behavior of these structures in the event of geohazards-type impacts. This section describes how these effects would continue to be evaluated as design progresses through State permitting, following accepted industry practice and standard of care. The National Environmental Policy Act (NEPA) does not require that engineering plans are at an advanced design level; and frequently, conceptual-level design information is used to analyze impacts. Sufficient information for a complete application was submitted by the Applicant, and therefore, USACE must evaluate the application, including proceeding with the NEPA analysis. If the design changes appreciably after the NEPA process, USACE would evaluate whether permit modifications or re-evaluation under NEPA would be needed. A description of uncertainties, assumptions used in the analyses, and related risk due to the conceptual level of design are disclosed in this section (and in Appendix K4.15, Geohazards and Seismic Conditions) where they affect the impact analysis. In

[1]**Recurrence interval** (or return period) is an estimate of the probability or frequency that certain geohazards are expected to occur, based on geologic and seismologic evidence.

addition, mitigation measures that would reduce the level of uncertainty and risk are described in this section, and in Appendix M1.0, Mitigation Assessment.

This section describes the following potential impacts related to geohazards:

- Stability of major mine structures during operations and closure.
- Effects of earthquakes on project facilities.
- Effects of unstable slopes on project facilities.
- Effects of geotechnical conditions and coastal hazards on port structures and pipeline landfalls (e.g., shallow bedrock).
- Effects of tsunamis and seiches on port and ferry terminals.
- Effects of volcanoes on project facilities.

Potential impacts to the environment resulting from geohazard-caused upset conditions, such as an embankment failure, are addressed in Section 4.27, Spill Risk. Impacts from water and ice hazards, such as waves and lake ice, are discussed in Section 4.16, Surface Water Hydrology. As described in Section 3.14, Soils, permafrost has not been encountered in the mine site or other project areas based on field investigations; therefore, potential effects from permafrost hazards are not addressed in this section.

Scoping comments expressed concerns that major faults occur in the project area and may affect project facilities. Commenters requested that the EIS include detailed information about seismically active areas, geological faults and tectonic activity, and corresponding design features. They also requested information on how the project facilities, particularly the tailings storage facilities (TSFs), would withstand earthquakes; and an analysis of potential impacts from volcanic activity from Augustine Volcano, especially at Amakdedori port and along the pipeline.

4.15.1 Summary of Key Issues

Table 4.15-1: Summary of Key Issues for Geohazards and Seismic Conditions

Impact Causing Project Component	Alternative 1a	Alternative 1	Alternative 2 and Variants	Alternative 3 and Variant
Mine Site				
Tailings Storage Facility and Water Management Pond Embankment Stability	Low probability of embankment instability based on preliminary static stability analysis: FoS 1.9 to 2.0 based on downstream slopes of 2.6H:1V for TSFs and 2H:1V for WMPs; additional geotechnical and stability analyses to be incorporated into advanced design stages. Temporary repairable damage in OBE, and <1-foot displacement in MCE based on pseudo-static analysis and target seismic FoS of 1.2; would not result in effects	Same as Alternative 1a: Static FoS of 1.9 to 2.0 based on downstream slopes of 2.6H:1V for TSFs and 2H:1V for WMPs; and <1-foot displacement in MCE based on target seismic FoS of 1.2.	Downstream Bulk TSF Embankment: Design provides marginal additional static or seismic stability over Alternative 1a design. Static stability: FoS 1.9 to 2.0 based on downstream slope of 2.6H:1V for both designs. Seismic (pseudo-static) stability: downstream design has 0.04 foot less displacement than buttressed-	Same as Alternative 1a: Static FoS of 1.9 to 2.0 based on downstream slopes of 2.6H:1V for TSFs and 2H:1V for WMPs; and <1-foot displacement in MCE based on target seismic FoS of 1.2.

Table 4.15-1: Summary of Key Issues for Geohazards and Seismic Conditions

Impact Causing Project Component	Alternative 1a	Alternative 1	Alternative 2 and Variants	Alternative 3 and Variant
	on environment outside of footprint; additional seismic modeling to be conducted in final design. Duration long-term with removal of pyritic TSF and WMPs at closure and dry closure of bulk TSF.		centerline design for 2 of 4 MCEs. Other TSF and WMP Embankments: Same as those for Alternative 1a.	
Open Pit Slope Stability	Low to medium likelihood of localized unstable slopes in pit in early closure, to be mitigated through targeted groundwater depressurization while lake rises. Landslide-induced pit lake wave would not overtop rim.	Same as those for Alternative 1a.	Same as those for Alternative 1a.	Same as those for Alternative 1a.
Container Storage and Pumphouse	No variants are analyzed for this alternative: Low likelihood of earthquake toppling effects at container storage area with foundation preparation.	**Summer-Only Ferry Operations Variant:** Low likelihood of earthquake toppling effects at container storage area with foundation preparation.	**Summer-Only Ferry Operations Variant:** Same as those for Alternative 1.	**Concentrate Pipeline Variant:** Impacts at pumphouse are similar to those for Alternative 1a.
Transportation Corridor				
Ferry Terminals and Operations	Earthquakes: Low likelihood of temporary ground-shaking effects such as cracking, spreading, and settlement of terminals. Low likelihood of tsunamis, seiches, and unstable slope effects.	Ground-shaking impacts are similar to those for Alternative 1a. Landslide-induced lake tsunamis: slightly lower likelihood of impacts on Alternative 1 north ferry terminal than for Eagle Bay ferry terminal for Alternative 1a. **Summer-Only Ferry Operations Variant and Kokhanok East Ferry Terminal Variant:** Slightly lower potential for lake tsunami impacts than Alternative 1a and Alternative 1	Ground-shaking impacts are similar to those for Alternative 1. Landslide-induced tsunamis: slightly higher potential for impacts than Alternative 1a or Alternative 1 ferry terminals and crossings. **Summer-Only Ferry Operations Variant:** Slightly lower potential for lake tsunami impacts than Alternative 2 due to fewer ferry operations.	No geohazards effects because there would be no ferry terminals.

Table 4.15-1: Summary of Key Issues for Geohazards and Seismic Conditions

Impact Causing Project Component	Alternative 1a	Alternative 1	Alternative 2 and Variants	Alternative 3 and Variant
		base case due to fewer ferry operations.		
Road Construction and Operations	Unstable slopes: Minor areas along mine access road (e.g., near Roadhouse Mountain). Low likelihood of impacts expected with typical engineering and construction practices. No variants are analyzed for this alternative.	Unstable slopes: Fewer impacts along the mine access road than Alternative 1a. **Summer-Only Ferry Operations Variant and Kokhanok East Ferry Terminal Variant:** Impacts would be similar to Alternative 1a and Alternative 1 base case.	Unstable slopes: Higher likelihood of impacts along road corridor than Alternative 1a or Alternative 1; effects would be temporary and localized with engineering controls and maintenance. Liquefaction: Higher potential at Pile and Iliamna river crossings than for Alternative 1a or Alternative 1. **Summer-Only Ferry Operations Variant:** Impacts would be similar to Alternative 1. **Newhalen River North Crossing Variant:** Impacts would be similar to Alternative 1a and Alternative 2 base case.	Unstable slopes: Slightly higher likelihood of effects than Alternative 2 due to longer route in steep terrain; effects would be similar to Alternative 2 with engineering controls and maintenance. Liquefaction: Potential would be similar to Alternative 2. Landslide-induced lake tsunamis: low likelihood of effects on eastern parts of the road close to lakeshore. **Concentrate Pipeline Variant:** Low likelihood of minor spills due to unstable slopes.
		Ports		
Dock and Port Facilities Construction and Operations	Caisson dock stability: Low likelihood of stability effects on dock, assuming additional geotechnical and stability evaluations in final design. Unstable Slopes: Low likelihood of effects. Tsunamis: Low to moderate likelihood of temporary (repairable) effects such as dock or fuel tank damage, assuming additional site-specific analysis in final design. Volcanic ash from Augustine Volcano: Low	Sheet pile dock stability: Slightly higher likelihood of stability effects and damage from boulders or shallow bedrock, scour, and potential for fill escape than for Alternative 1a. Unstable Slopes: Same as Alternative 1a. Tsunamis: Slightly higher likelihood of effects on sheet pile dock due to cross-sectional area.	Sheet pile dock stability: Slightly higher likelihood and extent of stability effects than Alternative 1 sheet pile dock due to 4x larger structure and finer seabed/fill material, increased liquefaction potential, buried boulders, and 10-foot elevation change at mudline on either side of the northwestern corner of the dock. Unstable Slopes: Higher likelihood of effects than	Caisson dock stability: Slightly higher likelihood of foundation, stability, and liquefaction effects than Alternative 1a caisson dock due to finer seabed material and buried boulders; and slightly higher likelihood of these effects than Alternative 2 due to greater elevation change (12- to 15-feet) on either side of the caissons. Unstable Slopes: Higher likelihood of

Table 4.15-1: Summary of Key Issues for Geohazards and Seismic Conditions

Impact Causing Project Component	Alternative 1a	Alternative 1	Alternative 2 and Variants	Alternative 3 and Variant
	likelihood of port operations interruption.	**Pile-Supported Dock Variant:** Higher likelihood of damage/repairs needed during project life due to shallow bedrock; similar or lower likelihood of stability effects than caisson or sheet pile dock. **Summer-Only Ferry Operations Variant:** Slightly higher likelihood of debris impacts during tsunami due to increased container storage.	Alternative 1a due to steep alluvial fan material in port area. Tsunamis: Slightly lower intensity than for Alternative 1 due to lower predicted run-up elevation, though higher likelihood of local landslide-generated tsunamis. Volcanic ash from Augustine Volcano: Slightly higher likelihood of effects than for Alternative 1a and Alternative 1 during winter due to prevailing winds. **Pile-Supported Dock Variant:** Higher likelihood of damage/repairs needed during project life than other dock designs due to shallow bedrock; and lower likelihood of stability effects than sheet pile dock.	effects on port facilities than Alternative 1a and Alternative 2 due to rockslide/rockfall potential at port facilities and talus slopes adjacent to dredged material storage area. Tsunamis: Slightly higher intensity at port facilities than Alternative 2 due to higher predicted runup elevation; lower likelihood of effects on caisson dock than for Alternative 2 due to smaller cross-sectional area; likelihood of landslide-generated tsunamis similar to Alternative 2. Volcanic ash from Augustine Volcano: Impacts would be the same as those for Alternative 2. **Concentrate Pipeline Variant:** Unstable slope effects on the storage facility same as Alternative 3. Tsunamis: Same likelihood as for Alternative 3 port facilities, but with slightly higher risk of contaminant release.
Natural Gas Pipeline Corridor				
Construction and Operations—Offshore Cook Inlet	Low likelihood of pipe damage from liquefaction or exposed bedrock. Low likelihood of scour effects due to pipeline burial and minimum depth of cover (1 to 2 feet), or on-bottom	Same impacts as those for Alternative 1a.	Liquefaction, bedrock, and scour: Impacts in Cook Inlet would be similar to those for Alternative 1a. Low likelihood of active fault crossing	Liquefaction, bedrock, and scour: Impacts would be similar to those for Alternative 1a. Surface faults: Same as those for Alternative 2.

Table 4.15-1: Summary of Key Issues for Geohazards and Seismic Conditions

Impact Causing Project Component	Alternative 1a	Alternative 1	Alternative 2 and Variants	Alternative 3 and Variant
	stability analysis for segment with no cover. No active fault crossing effects expected.		(Bruin Bay fault) and displacement effects.	
Construction and Operations—Coastal Cook Inlet	Low likelihood of pipe damage from coastal hazards (e.g., boulder rafting, scour, sediment drift). Pipeline burial below mudline and depth of cover (3 to 5 feet) would be sufficient to avoid hazards.	Same impacts as those for Alternative 1a.	Similar impacts to those for Alternative 1a, except for slightly higher liquefaction potential in estuarine deposits.	Similar impacts to those for Alternative 2, except for 1 mile longer in liquefiable estuarine deposits.
Construction and Operations—Upland Areas	Low likelihood of unstable slope effects on pipeline.	Similar impacts to those for Alternative 1a. **Kokhanok East Ferry Terminal Variant:** Similar impacts to those for Alternative 1a and Alternative 1 base case.	Unstable slopes: Low-medium likelihood of effects (such as operations interruption or rupture) between Diamond Point and Roadhouse Mountain; expected to be mitigated through typical engineering controls and monitoring. Liquefaction: Higher potential for impacts than that for Alternative 1a and Alternative 1, due to more areas of wide alluvial and estuarine deposits.	Same impacts as those for Alternative 2.

Notes:
[1]Slope angle expressed as ratio of horizontal (H) distance to vertical (V) change in elevation.
FoS = Factor of Safety
H:V = horizontal/vertical
MCE = Maximum Credible Earthquake
OBE = Operating Basis Earthquake
TSF = Tailings Storage Facility
WMP = Water Management Pond

4.15.2 No Action Alternative

Under the No Action Alternative, no construction, operations, or closure activities specific to the Applicant's Preferred Alternative would occur. Although no resource development would occur, PLP would retain the ability to apply for continued mineral exploration activities under the State's authorization process, as well as any activity that would not require federal authorization. In addition, there are many valid mining claims in the area, and these lands would remain open to mineral entry and exploration by other individuals or companies.

Current State-authorized activities associated with mineral exploration and reclamation and scientific studies would be expected to continue at levels similar to recent post-exploration activity. The State requires reclaiming sites at the conclusion of their State-authorized exploration program. If reclamation approval is not granted immediately after the cessation of reclamation activities, the State may require continued authorization for ongoing monitoring and reclamation work as it deems necessary.

Effects on project components from geohazards, seismic events, and other geotechnical conditions would not occur, and no impacts on the environment would result from such effects. Natural geohazards such as those described in Section 3.15, Geohazards and Seismic Conditions, would continue to affect existing communities and infrastructure in the region.

4.15.3 Alternative 1a

4.15.3.1 Mine Site

This section describes potential effects of seismic events and other geohazards on major structures at the mine site; the ability of the structures to withstand these hazards; and the likelihood that such hazards could produce related environmental impacts. Figures in Chapter 2, Alternatives, display the mine site layout; and Table K4.15-1 in Appendix K4.15 provides the buildout dimensions of embankments and impoundments that would contain tailings, waste rock, and/or contact water at the mine site. This section also addresses potential geohazard effects on the open pit.

Embankment Construction Material

The embankments for the tailings storage and water management facilities would be constructed of rockfill and earthfill materials obtained from drilled and blasted bedrock removed from quarries A through C,[2] and the overburden in the open pit (see Chapter 2, Alternatives, Figure 2-4). Analyses were completed to determine the quantities of on-site embankment construction materials and project-related needs. Appendix K4.15 (see Table K4.15-2 and Table K4.15-3) provides embankment material quantities that would be generated by quarries A through C and the open pit overburden, as well as the embankment material needs for the relevant mine site–related facilities.

Based on the material properties, quantities, and assumptions provided by PLP (2018-RFI 015b; PLP 2019-RFI 108a; PLP 2019-RFI 008e update); the combination of quarries A through C and the open pit overburden could generate about 4 to 5 percent less compacted rockfill and earthfill material than needed to construct the embankments. These results are based on various conservative assumptions regarding bulking, compaction, and usable material reduction factors assigned in Table K4.15-2. The effect of changing these assumptions is described in Appendix K4.15. For example, if slightly higher but still reasonable bulking factors were used based on numbers in the literature (e.g., Look 2007), the calculated compacted rockfill available from the quarry and pit sources would be higher than needed for embankment rockfill and road maintenance. As described in Chapter 5, Mitigation, the material balance (surplus/deficit) would be further refined as the design and site investigation programs are advanced; and if necessary, the base elevation of the quarries would be lowered to increase earthfill and rockfill material availability (PLP 2018-RFI 015a). In particular, quarry A in the bulk TSF footprint could be expanded if needed to meet material requirements during construction without impacting the overall footprint. An expansion of quarry A would also provide additional tailings storage capacity.

[2]Quarry A is shown on Figure 2-4 in the footprint of the bulk TSF; this quarry would be developed before the construction of the bulk TSF.

Therefore, the likelihood that additional rockfill material would be needed as the project progresses, with related project footprint increases, is low.

Appendix K4.15 describes the availability of low-permeability material expected from open pit overburden stripping that may be used as liner bedding, embankment core zones, and the bulk TSF closure cover, depending on detailed design. Pit overburden deposits mainly consist of low-permeability clayey sands and gravels derived from glacial drift and glacial lake deposits (see Figure 3.13-2) that would be segregated into appropriate stockpiles based on material gradations. If additional low-permeability materials are needed, they would be sourced from embankment foundation excavations and other site preparations. It is expected that the pit and other mine excavations would provide the sufficient amount of low-permeability material to meet the requirements for these materials specified in the detailed design. For example, the estimated volume needed for both liner bedding and the bulk TSF closure cover represents about 38 to 44 percent of the pit overburden, and about 10 to 20 percent of total overburden (including other excavations).

Embankment and Impoundment Design and Construction

The embankments and impoundments could be impacted by geohazards, such as instability associated with seepage, internal erosion,[3] foundation conditions, high precipitation, and earthquakes. The embankments would therefore be designed, constructed, and operated to remain stable during these events, including under both static (non-seismic) and seismic conditions.

All embankments would be subject to State of Alaska regulations per Chapter 17 in Title 46 of the Alaska Statutes (AS 46.17) and Article 3 Dam Safety of Chapter 93 in Title 11 of the Alaska Administrative Code (11 AAC 93). The Dam Safety and Construction Unit (Dam Safety) of Alaska Department of Natural Resources (ADNR) would be responsible for "supervision" of the safety of the embankments and for administration of the Alaska Dam Safety Program (ADSP). A draft revision of *Guidelines for Cooperation with the Alaska Dam Safety Program* (dam safety guidelines) (ADNR 2017a) is in the public domain, but has not yet been formally adopted by ADNR. A portion of the dam safety guidelines regarding periodic safety inspections were formally adopted by ADNR in 2003 by reference in 18 AAC 93. Subsequent revisions to the guidelines (ADNR 2005b, 2017a) have not been adopted in regulations, and may not be enforceable under AS 46.17 or 11 AAC 93 (ADNR 2020).

The regulatory requirements are obligatory, and typically considered as the "minimum" standard of care. The intent of the ADSP is to provide for the protection of human lives, property, and the environment, including anadromous fish streams, through consistency in design approach, construction, and operation of water and TSF. The draft ADSP dam safety guidelines do not dictate how a facility is to be designed and constructed, but do describe a minimum standard of care, indicating that designs should follow a higher standard based on accepted industry standards and procedures (i.e., what a reasonable person or expert in the industry would consider foreseeable risk and the standard of care). Mitigation measures such as those described below that rely on proven engineering controls are more appropriate in reducing dam failure risk than relying on compliance with State regulatory programs (ADNR 2020; Cobb 2019; Fourie 2009; Morgenstern 2018; Silva et al. 2008).

[3]**Internal erosion**, also referred to as piping, is the formation of voids in a soil caused by the removal of material by seepage, and occurs when the hydraulic forces exerted by water seeping through the pores and cracks of the material in the embankment are sufficient to detach particles and transport them out of the embankment structure.

The current level of embankment design is considered to be at an advanced conceptual to initial preliminary level. As the design advances, it would go through preliminary and detailed design levels, terms which are explained in the draft guidelines (ADNR 2017a), and are accepted globally as state-of-practice design terminology. Prior to construction, all embankment starter dams and all embankments that would be built to their full height at the outset would undergo initial application package preparations (complete with conceptual design information), preliminary and detailed designs, final construction package preparation, safety reviews, and submittals by a qualified engineer to ADNR for Certificates of Approval to Construct a Dam. Also prior to construction, each embankment raise would undergo a separate design and safety review that would be adjusted as necessary, based on knowledge from previous raise constructions, TSF operations, and tailings characterizations, followed by submittals to ADNR for a Certificate of Approval to Modify a Dam.

Prior to operations, following the completion of each starter dam, full embankment, or embankment raise construction, a construction completion report would be submitted to ADNR for a Certificate of Approval to Operate a Dam. Therefore, no operations would be permitted to start until the construction completion report is approved and the certificate is issued. Also, all dam repairs, removals, and abandonments require separate designs, safety reviews, and submittals to ADNR for Certificates of Approval to Repair, Remove, and Abandon a Dam, respectively.

The following summarizes geohazard considerations for the design and construction of the major embankments and impoundments, including the bulk TSF, pyritic TSF, water management ponds (WMPs), and seepage collection ponds (SCPs). More detailed information is provided in Appendix K4.15.

Bulk TSF. The bulk TSF would be designed to impound the bulk tailings, and includes a main (north) embankment and a south embankment with the following design, construction, and monitoring elements to prevent geohazard-related impacts:

- Siting in a single tributary watershed surrounded by bedrock knobs to focus potential impacts in one watershed and incorporate natural containment elements.
- Foundations to be placed on competent bedrock for increased embankment stability. All overburden soils and weathered bedrock in the embankment footprint areas would be removed to expose the competent bedrock.
- Main embankment starter dam downstream-constructed[4] to a maximum height of 265 feet, followed by centerline-construction[5] of the upper 280 feet of the embankment to reduce the footprint, with a buttressed downstream slope to enhance stability (total maximum height 545 feet). This would result in an overall downstream embankment slope of 2.6 horizontal (H): 1 vertical (V), including benches, with intermediate slopes designed at 2H:1V; and a serrated near-vertical upstream face for the 280-foot-high centerline part (see Chapter 2, Alternatives, Figure 2-8).
- Main embankment operated as a permeable flow-through structure with continuous engineered filter zone to control drainage in the embankment, prevent internal erosion, and remain functional after a seismic event.

[4]**Downstream construction** is a method of dam (embankment) construction in which a rockfill dam is raised in the downstream direction by placement of fill on top of the dam crest and downstream slope of the previous raise.
[5]**Centerline construction** is a method of dam (embankment) construction in which a rockfill dam is raised by concurrent placement of fill on top of the dam crest; the upstream slope, including portions of the tailings beach; and the downstream slope of the previous raise.

- South embankment constructed using downstream methods to a maximum height of 300 feet. The downstream embankment slope would be 2.6H:1V. The upstream embankment slope would be flatter, at 3H:1V, to facilitate the placement of a liner on the slope.
- South embankment operated as an impervious structure with a liner on the upstream face (or a low-permeability core zone), combined with a grout curtain in bedrock, and engineered filter zone to protect the liner (or core) and prevent internal erosion. The upstream liner or core zone would key into a concrete plinth to form a continuous seepage barrier with the grout curtain, which would be keyed into bedrock to prevent leakage beneath the embankment.
- Tailings storage impoundment containing thickened tailings with a small pond on the surface away from the main and south embankments, covering only about one-fourth to one-third of the total surface area (see Table K4.15-1).
- Underdrains in natural tributary drainages beneath the impoundment, an aggregate drain at a topographic low point beneath the main embankment to provide a preferential seepage path from the tailings to downstream of the embankment toe, and additional underdrains running parallel to the embankment to allow for drainage of seepage collected along the embankment.
- Water management to protect all embankments from seepage pressure-related instability, with excess pond water pumped to the main SCP of the bulk TSF and/or the main WMP.
- Drainage ditches around the toes of the embankment slopes to prevent erosion and undercutting, and to allow drainage water to flow unimpeded to the SCPs.
- Diversion channels and rockfill embankment material that minimize erosion on the downstream face of the embankments.
- Freeboard to contain the entire inflow design flood above the tailings beach, and account for potential seismic deformation of the embankment crests so that water cannot overtop the embankment crests.
- Water balance model that incorporates an analysis of historic trends and extremes to account for potential climate change effects on runoff and pond size (see Section 3.16 and Appendix K3.16, Surface Water Hydrology).
- Wide tailings beach to keep pond water away from the embankments, and thereby reduce seepage pressures on the embankments and promote subsurface drainage to the main flow-through embankment with pond development against bedrock high to the southeast.
- Reduced tailings volume by using thickened tailings discharge methods that would increase the density and decrease the water content of the deposited tailings, and by additional pumping capacity to remove excess pond water to the main WMP.
- Dry closure methods to improve stability for permanent in-place closure, with a closure cover design that would minimize infiltration, regrading, and surface drainage to promote runoff, tailings consolidation, and long-term internal drainage.
- Monitoring performed during construction, operations, closure, and post-closure.

Pyritic TSF—The pyritic TSF would be designed to impound pyritic tailings, potentially acid-generating (PAG) waste rock, and metal-leaching (ML) materials in a co-placement[6] manner during operations, which would be moved to the open pit at closure. This form of tailings and waste rock co-disposal is in common use globally (Habte and Bocking 2017). Examples of existing and planned co-disposal TSF operations are discussed in Appendix K4.15.

The pyritic TSF would include a continuous embankment around the northern, eastern, and southern sides that have been named the north, east and south embankments, respectively, with the following design and construction elements to prevent geohazard-related impacts:

- The majority of the pyritic TSF would be in a single tributary valley bounded by high ground on the western side to focus potential impacts in one watershed and incorporate a natural containment element.
- North, east, and south embankments prepared by removing overburden to competent bedrock over the entire embankment footprints, and downstream-constructed to maximum heights of 335, 225, and 215 feet, respectively.
- Fully lined TSF with liner underlain by a layer of processed bedding material (sand and gravel) to protect and cushion the liner from exposed ground surface materials, and underdrains to collect and convey any seepage to the downstream SCPs.
- Liner overlain and protected by processed materials (sand and gravel) after liner installation to prevent damage to liner from punctures and damage during waste rock placement.
- Waste rock placed in a ring over the processed sand and gravel around the inside perimeter of the TSF.
- Tailings discharged into the TSF from sub-aqueous discharge points during operations to minimize oxidation and potential acid generation with the tailings surface level maintained at all times below the waste rock surface level.
- Water levels maintained on top of the tailings and waste rock for the full life of the facility, with freeboard maintained to account for inflow design flood, wave run-up, wind set-up, seismic deformation, and excess pond water pumped to the main WMP.
- Tailings, waste rock, and any impacted underlying materials moved into the open pit at closure.
- After closure, the liner removed and embankments graded/recontoured to conform to the surrounding landscape and promote natural runoff and drainage.
- Monitoring included during construction, operations, and closure.

The presence of colluvium and solifluction deposits on the sides of the impoundment that are subject to frost creep could lead to potential stretching of the upper liner on valley side slopes before it is covered. Liner deformation is expected to be minimized by placement of liner bedding material prior to installation, and placement of the protective layer and PAG waste rock fill on top of the liner that would buttress such movement.

WMPs and SCPs—Two primary WMPs would be at the mine site (the main WMP north of the pyritic TSF, and the open pit WMP) to impound contact and open pit water, respectively. The SCPs would be sited downstream of the TSF embankments, including those associated with the bulk TSF main and south embankments, and the pyritic TSF north, east, and south embankments.

[6] **Co-placement** is a co-disposal method in which tailings and waste rock are transported independently to the same storage facility, but not pre-mixed to form a single discharge stream. Examples are waste rock end-dumped into a TSF, or waste rock placed to create internal berms or retaining walls of a TSF.

The facilities would include the following design and construction elements to prevent geohazard-related impacts:

- WMPs fully lined with an engineered filter zones, and SCPs with low-permeability and engineered filter zones keyed into grout curtains, to minimize seepage and risk of internal erosion.
- Rockfill embankments to promote stability and safety under static and seismic loading conditions.
- Main WMP embankment to have a maximum height of 190 feet and 225-acre footprint prepared by removing overburden so that the embankment is constructed on competent bedrock.
- Open pit WMP embankment design concept requiring potential weak foundation conditions encountered in the overburden materials (e.g., glacial lake deposits) to be excavated.
- Pond water volumes managed through reuse in the process plant, and treatment and discharge.
- Monitoring/seepage pumpback wells downgradient to detect and capture potential liner leakage.
- At closure, the WMPs to be removed and embankments graded/recontoured to conform to the surrounding landscape and promote natural runoff and drainage.
- Monitoring included during construction, operations, closure, and post-closure.

The main WMP would be composed of a 225-acre reservoir with 190-foot-high embankment, which is in line with the largest geomembrane-lined water storage reservoirs in the world. Comparable examples in the US and worldwide are described in Appendix K4.15.

Seepage Analysis

A seepage analysis was conducted of the bulk TSF based on a 2-dimensional model (SEEP/W), which resulted in predicted seepage rates for use in the site-wide water balance model (see Section 4.16, Surface Water Hydrology), and informed the behavior of the phreatic surface for understanding the stability of the embankments. Details of the seepage model assumptions, input parameters, material layout, boundary conditions, and results are provided in Appendix K4.15.

Sensitivity analyses were run to evaluate the effect of uncertainties in various parameters. Overall seepage rates estimated for the bulk TSF main embankment at the end of operations ranged from 3.6 to 18 cubic feet per second (cfs) for the various sensitivity analyses. The results indicated that seepage flow is most sensitive to recharge rates on the tailings beach, the distance of the supernatant pond from the dams, and isotropy of the coarse tailings unit; and less sensitive to changes in tailings and bedrock hydraulic conductivity values. The increase in recharge and pond size in the sensitivity analyses demonstrates the range of seepage flow that could occur due to increased precipitation from climate change. Based on the likelihood that the tailings would be anisotropic, Knight Piésold (2019o) selected a range of 3.5 to 5.5 cfs as a best estimate for use in the mine site water balance model. In post-closure, seepage rates were estimated to be about 10 to 30 percent of those during operations depending on closure cover type, ranging from 0.3 to 1.2 cfs through the main embankment, and 0.1 to 0.6 cfs through the south embankment. The post-closure results were less sensitive to the presence or absence of a seasonal pond than the operations results (Knight Piésold 2019o; PLP 2019-RFI 006b, c).

The model also provided information on the behavior of the phreatic surface in the embankments. As shown in Figure K4.15-3, the phreatic surface next to the main embankment is expected to vary with the history of spigotting tailings in the area, as well as the ability of the tailings to segregate. Operational practices to maintain the desired width of tailings beach and keep the pond away from the dams would entail varying the tailings discharge locations, resulting in a phreatic surface elevation that could vary along the length of the embankment at any given time. The seepage model also shows how the phreatic surface is expected to decline in early closure after the end of spigotting, resulting in more stable embankment conditions in post-closure (PLP 2019-RFI 006b, -RFI 008h, and -RFI 130). Refined seepage analyses in the preliminary and detailed designs would consider tailings grain-size distribution based on additional test work, a plan for discharging tailings into the impoundment, and further analysis on the range of input parameters to assess the plausible range of flow conditions that could exist (PLP 2019-RFI 006c).

The flow-through design of the bulk TSF main embankment is intended to promote unsaturated conditions in the coarse tailings deposited near the embankment and reduce porewater pressures in the embankment fill materials. This has been identified as a Best Available Technology (BAT) principle for tailings dams by the expert panel that reviewed the Mount Polley dam failure (Morgenstern et al. 2015). As noted above, the large, continuous, engineered filter zone in the embankment is intended to control internal erosion, while promoting internal drainage and reduction of the phreatic surface, which enhances stability.

PLP 2019-RFI 006c and PLP 2019-RFI 008h provide a summary of centerline dams worldwide that are currently operating and have heights and seepage similar to what is estimated for the bulk TSF main embankment. For example, the Gibraltar and Brenda mines in British Columbia and the Montana Resources' Continental Mine in Montana have centerline or modified-centerline TSFs in the range of 385 to 750 feet in height, and seepage flows in the range of 2 to 10 cfs in operations and 1 to 2 cfs in closure. It is noted that the centerline constructions of these three example TSFs are different than the construction planned for the bulk TSF main embankment, in that the Gibraltar and Brenda TSF dams are raised by cyclone tailings sands, and the Continental TSF embankment has alluvial soils on its upstream slopes to reduce tailings migration into the rockfill. The seepage rates recorded through these three example dams that are of similar heights to the bulk TSF main embankment are similar to the seepage rates estimated through the bulk TSF main embankment.

Preliminary Static Stability Analyses

Preliminary analyses were completed to evaluate the stability of the embankments under static and non-seismic conditions based on the current conceptual levels of design. The following summarizes the static stability analysis. A more detailed discussion is presented in Appendix K4.15. The following major embankments were analyzed for static stability: bulk TSF main and south pyritic TSF north, main WMP, open pit WMP, and bulk TSF main SCP.

Input Parameters and Methods—Input parameters for the preliminary analyses were based on the results of field and office studies, and included the embankment configurations and assumed rockfill material, foundation material, and stored material parameters listed in Table K4.15-5. The analyses were completed using the software program SLOPE/W. Potential slip surfaces analyzed are shown on Figure K4.15-4 through Figure K4.15-9.

The preliminary analyses assumed homogeneous conditions for the foundation materials, with strength parameters selected based on both drillhole data and typical values in the literature (see Table K4.15-5). A summary of foundation materials and potential weak zones encountered in drillholes completed to date is provided in Appendix K4.15 for each of the major embankments. Foundation conditions would be further investigated and parameters for stability analysis refined as design progresses through State permitting (see Chapter 5, Mitigation).

Results and Target Factor of Safety—The results of the preliminary static stability analyses predicted the analyzed embankments would have a static factor of safety[7] (FoS) between 1.9 and 2.0 (see Table K4.15-6). The minimum allowable FoS is an important design factor that is determined by the Applicant based on standards of engineering practice and can be different for various components and phases of mine design (e.g., liners, underdrains, pit slope failure in post-closure). For the purpose of static loading, PLP has indicated it would meet or exceed a target FoS of 1.8 (Knight Piésold 2019p; PLP 2018-RFI 008g).

The current conceptual-level design FoS values are considered adequate for determining low probabilities of instability; for comparing different types of embankments such as downstream and centerline; and for PLP project planning. Acceptably reliable FoS values for preliminary and detailed design and final construction package purposes would be refined based on additional geotechnical investigation of tailings and embankment fill characteristics during the advanced preliminary and detailed stages of the designs.

The ADNR (2017a) draft guidelines under the ADSP do not specify a minimum FoS that must be met. The purpose of the guidelines is to outline the typical information required, while recognizing that every dam is unique. It is the dam engineer's responsibility to use an industry standard-based approach to design, which includes specification of a minimum FoS and respective analyses as part of a minimum standard of care, and to defend the design based on the level of detail in the engineering. The level of detail in engineering work has more influence on the likelihood of failure than increasing the FoS on a less detailed design (Silva et al. 2008). Therefore, there is much uncertainty in evaluating the stability of the mine site embankments based on a conceptual-level design.

There are three areas of uncertainty with respect to embankment stability at the current level of design: 1) the extent that the thickened tailings would segregate to promote coarser material and a deeper phreatic surface near the embankment; 2) the extent that pore pressures in the newly placed, potentially soft and loose tailings would reduce sufficiently to provide a stable upstream slope of the first raise; and 3) how to schedule and construct the first raise with its upstream part over tailings placed in less than 2 years (estimated time for starter dam to fill to capacity). These are discussed in more detail in Section K4.15. Uncertainties need to be resolved during the preliminary and detailed design processes and early during the first year of operations. This would minimize the potential that a deeper failure surface (up to the depth of the lowest centerline raise) could result in more of the centerline part of the embankment (below just the most recent raise) sliding into potentially undrained tailings, which could set the mass in motion with adverse consequential effects on the TSF in a downstream direction.

The technical key to addressing these three uncertainties is to obtain geotechnical characteristics of the tailings, which can only be accomplished after tailings deposition has begun and actual deposited tailings become available for geotechnical investigations and raise design, including seepage, stability, consolidation, pore-pressure, and liquefaction analyses. Initial geotechnical investigation results obtained in the first year of operations are needed to provide data on the extent of segregation that is being achieved, pore pressures within the tailings, consolidation rates of the tailings, and depth of the phreatic surface in the tailings and embankment. These geotechnical data can then be used in analyses to provide stable and safe centerline raises, starting with the first raise above the starter dam and initial deposited tailings. Chapter 5, Mitigation, describes some of the additional geotechnical investigation and engineering design

[7]**Factor of safety** is the ratio of the strength of a structure to an applied load, or the ratio of forces resisting failure to those driving failure. It can be a calculated number from a stability analysis, or a target number imposed by regulation or engineering standards and practices. An FoS of exactly 1 means that a structure would support only the applied load and no more; i.e., failure is impending. A structure with an FoS of 2 would fail at twice the design load.

work that would be completed during detailed design, including additional seepage, stability, and liquefaction analyses. Additional recommendations for tailings geotechnical investigation and stability analyses are provided in Appendix M1.0, Mitigation Assessment. The scope of additional work would be specified in an initial application package after the EIS is complete (PLP 2019-RFI 008g). The application package would include descriptions of the conceptual designs and a design scope that would "define the proposed level of work, methodologies, levels of analysis, and approaches to determine and evaluate those parameters that are required for the safe design and construction of the dam" (ADNR 2017a).

Closure—As described above and in Appendix K4.15, the long-term stability of the bulk TSF main embankment would be enhanced through a reduction in seepage flow after tailings deposition stops; removing the pond, promoting runoff, and limiting infiltration through closure cover design; and consolidation and long-term internal drainage of the tailings. As described in Chapter 5, Mitigation, analysis of tailings properties that promote internal drainage near the embankments would be completed during detailed design and monitored through operations. If required to maintain conditions that achieve long-term drainage and stability goals (i.e., reduced phreatic surface and pore pressures at the embankment), alternative drainage-enhancing features would be considered, such as vertical or horizontal drains (PLP 2019-RFI 130).

Comparison to Other Centerline-Constructed Dams—PLP (2019-RFI 008h) provides a summary of 11 centerline dams that are currently operating globally and have some similarities to the bulk TSF main embankment. Three of the dams are directly comparable to the planned bulk TSF main embankment with regard to centerline construction. The Constancia dam is zoned rockfill with a vertical clay core and is higher than 328 feet. The 318-foot-high Highland Valley H-H Dam is an earthfill dam with a vertical core, random fill and tailings placed upstream, and variable waste fill placed downstream. The 750-foot-high Continental dam is rockfill with a centerline-constructed segment. All three dams have configurations and materials like those planned for the bulk TSF main embankment, except that Constancia and Highland Valley H-H have vertical cores, so they are not "flow-through" dams. The engineered filter zones in the bulk TSF, consisting of graded sands and gravels, is expected to be more effective than these low-permeability core examples in lowering the phreatic surface in the embankment and promoting stability. The Continental dam has alluvium on its upstream face to prevent tailings migration into the dam, so it can be considered a partial flow-through dam. The Constancia and Highland Valley H-H dams are lower—and the Montana Resources dam is higher—than the planned bulk TSF main embankment. These dams were still being raised at the time of preparation of this EIS.

Three additional dams are described as "modified centerline" dams, or hybrids of centerline and upstream or downstream construction with rockfill raises (Alumbrera, Fort Knox, and Montana Tunnels dams), and are somewhat comparable to the bulk TSF main embankment configuration. The Alumbrera dam is described as a rockfill/earthfill dam; is projected to be 540 feet high; and has a free-draining starter dam. The Fort Knox dam is rockfill, 350 feet high, and had planned centerline raises but was raised as a downstream-to-centerline hybrid. The Montana Tunnels dam is rockfill; was permitted to 410 feet in 2008; and started downstream with raises closer to upstream than centerline. Additional discussion of comparable dams in PLP (2019-RFI 008h) is provided in Appendix K4.15.

Preliminary Seismic Stability Analysis

Active Surface Faults—The mine site is situated in a regionally seismically active area caused by the convergence of the Pacific and North American tectonic plates. A description of the known active faults in the project area is provided in Section 3.15, Geohazards and Seismic Conditions. Because no mine facilities would be constructed on top of known active surface faults, it is unlikely there would be ground surface rupture effects on embankments and other mine facilities. The type of effects that could occur in the event that facilities or infrastructure were unknowingly built on an active fault that experienced surface rupture could include pipeline rupture, instantaneous displacement (lateral or vertical offset) of roads, or cracking and shearing of embankments and buildings.

The closest potentially active fault to the mine site is the Lake Clark fault. Section 3.15 provides a summary of evidence and uncertainties in the interpretation of the recency of faulting on this structure. The closest documented surface exposure of this fault is 14 miles from the mine site. Several possible extensions of this fault have been identified based on regional geophysical data as close as 6 miles from the mine site (see Figure 3.15-2), although these are not necessarily active faults, and field studies have not shown evidence of fault offset of surficial deposits in the area (Hamilton and Klieforth 2010; Haeussler and Waythomas 2011; Koehler 2010). Evidence of repeated paleo-liquefaction events as close as 8 miles southwest of the mine site (Higman and Riordan 2019) could suggest Holocene earthquake activity on either a buried Lake Clark fault extension or deeper subduction-related seismicity.

The implication of these uncertainties for the impact analysis is discussed in Appendix K4.15. The effect of a closer location of the Lake Clark fault on ground-shaking estimates for mine structures is discussed below under Seismic Hazard Analysis. Potential impacts from possible surface rupture at the fault extensions are discussed below under Transportation Corridor, and Natural Gas Pipeline. Chapter 5, Mitigation, and Appendix M1.0, Mitigation Assessment, describe PLP plans to continue to investigate the Lake Clark fault as design progresses (PLP 2019-RFI 139), and additional fault study recommendations that would help identify (or rule out) the potential splay locations and recency of faulting closer to the mine site.

Seismic Hazard Analyses—The TSF embankments at the mine site would be regulated as Class I (high) hazard potential dams under the ADSP draft dam safety guidelines (ADNR 2017a; PLP 2017). Based on these draft guidelines, two levels of design earthquake must be established for Class I dams (see Table K4.15-7):

- *Operating Basis Earthquake* (OBE) that has a reasonable probability of occurring during the project life (return period of 150 to more than 250 years), for which structures must be designed to remain functional, with minor damage that could be easily repairable in a limited time. In other words, minor damage within allowable design criteria may be sustained at the TSF embankments following an OBE earthquake.
- *Maximum Design Earthquake* (MDE) that represents the most severe ground shaking expected at the site (return period from 2,500 years up to that of the Maximum Credible Earthquake [MCE]), for which structures must be designed to resist collapse and uncontrolled release.

The size of an earthquake that can be expected is related to its return period, or how often it would occur. Moderate to large earthquakes (such as the OBE) occur occasionally and can be expected to occur during the life of the mine. Very large earthquakes occur very infrequently (with long return periods), but it is protective to consider them as the MDE that dams would have to withstand. Earthquake(s) selected for the MDE control the design of the dams, not the more frequent OBE.

The OBE can be defined based on probabilistic evaluations, with the level of risk (probability that the magnitude of ground motion would be exceeded during a particular length of time) being determined commensurate with the hazard potential classification and location of the dam (ADNR 2017a). The MDE may be defined based on either probabilistic or deterministic evaluations, or both (ADNR 2017a).

Ground shaking from earthquakes is typically presented in terms of peak ground acceleration (PGA), measured as a fraction (or percent) of gravity (g), which represents the initial intensity of an earthquake as it is applied to a structure, such as the TSF embankments. The degree of ground shaking and structural damage expected is related to earthquake magnitude, distance from active faults, and duration of shaking. For example, small local earthquakes may cause more ground shaking than large, more distant earthquakes; and large distant earthquakes with a lower PGA but longer shaking duration may cause more damage than smaller nearby earthquakes with a higher PGA. Therefore, the selected OBE or MDE may be based on more than one earthquake scenario. Several potential earthquakes were evaluated in the probabilistic and deterministic seismic hazard analyses to develop the OBE and MDE (see Appendix K4.15).

A conservative OBE corresponding to a return period of 475 years was adopted for the Pebble TSF designs (Knight Piésold 2019d). Based on the probabilistic seismic hazard analysis (see Table K4.15-8), the estimated PGA associated with this return period is 0.16g (or 16 percent of gravity acceleration).

The MCE was selected as the MDE for the Pebble TSFs (Knight Piésold 2019d). Based on the results of the deterministic seismic hazard analysis (see Table K4.15-9), earthquake magnitudes and ground shaking associated with the MCE considered in TSF embankment design include:

- A magnitude 6.5 shallow crustal earthquake from an unknown fault assumed to occur directly beneath the mine site, with a PGA of 0.56g.
- A magnitude 8.0 intraslab subduction earthquake (similar to the source of the magnitude 7.1 Anchorage earthquake on November 30, 2018), with a PGA of 0.61g.
- A magnitude 7.5 earthquake on the Lake Clark fault, with a PGA of 0.32g.
- A magnitude 9.2 megathrust earthquake with a PGA of 0.16g.

As noted above, one of the four MCEs selected for the MDE of mine site embankments is on a similar seismic source to that which caused the November 30, 2018 Anchorage earthquake. Dam inspections conducted in the region following the Anchorage earthquake indicate results ranging from no damage to minor cracking (Cobb 2019), which is similar to expectations following an OBE. The mine site embankments would be designed to withstand an MCE from the same intraslab subduction zone source, but nearly 10 times larger than the Anchorage earthquake.

The selection of four earthquake scenarios as MCEs to be considered in embankment design appears to be appropriate and conservative for site conditions. Appendix K4.15 provides further discussion of the seismic sources, ground motion models, and probabilistic and deterministic evaluations completed for the project to evaluate potential ground shaking associated with these earthquakes. Response spectra that show how ground shaking changes with time during each of the MCEs are also provided in Appendix K4.15 (see Table K4.15-10 and Figure K4.15-12). As described below under Seismic Deformation Analysis, the four earthquakes were used as input to preliminary seismic (pseudo-static) stability analyses for the major embankments at the mine site.

The probabilistic and deterministic seismic hazard analyses would be updated in final design, incorporating best practices for analysis and updated US Geological Survey (USGS) ground motion data as available (PLP 2018-RFI 008c; PLP 2019-RFI 008h). Further analyses of the effects of these earthquakes on the embankments would include compiling acceleration time-history records from past earthquakes that match each of the MCEs, which would be used as inputs to model the behavior of the embankments during the full duration of ground shaking in a maximum earthquake (see Chapter 5, Mitigation).

Preliminary Seismic Deformation Analysis—Preliminary pseudo-static deformation analyses were completed to predict the response of the major mine site embankments to a seismic event, based on MCEs from the four potential seismic sources (faults) noted above, with magnitudes ranging from 6.5 to 9.2. The input parameters, methods, and results from these analyses are provided in PLP 2019-RFI 008g, -RFI 008i, and -RFI 130, and are summarized in Appendix K4.15.

As shown in Table 4.15-2, predicted displacements from the preliminary analyses ranged from negligible (less than 0.03 foot) to 0.23 foot for the open pit WMP under the deep intraslab earthquake scenario, although displacement estimates are minimal in either case, and would not affect the integrity of the structure.

The current design of the mine site embankments is considered conceptual, and the pseudo-static results are considered preliminary. As described in Appendix K4.15, the pseudo-static method and input parameters used do not consider pore pressures, site-specific weak zones in the foundations, dynamic response of the embankments for the full length of ground shaking, or additive effects from aftershocks, and do not provide estimates of crest settlement; therefore, uncertainties regarding these factors remain. As described in Chapter 5, Mitigation, and below under Numerical Modeling, additional seismic stability analyses and crest deformation estimates would be completed and updated for each embankment structure as design progresses and additional field data are collected to support the understanding of geotechnical and hydrogeological conditions. The estimated crest deformation/settlement values would be added to the minimum freeboard requirements for the embankments, so that the minimum required freeboard would be maintained after the MDE event.

Table 4.15-2: Preliminary Seismic Stability Analysis Results for Mine Site Embankments

MCE Earthquake Magnitude (M), Source	PGA[1]	Downstream Deformation ($D_{84\%}$ in foot)[2,3]					
		Bulk TSF Main		Bulk TSF South	Pyritic TSF North	Main WMP	Open Pit WMP
		Buttressed-Centerline Construction[4]	Downstream Construction[5]				
M9.2, Megathrust	0.16 g	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03
M8.0, Deep Intraslab	0.61 g	0.07	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	0.23
M7.5, Lake Clark Fault	0.32 g	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	0.03
M6.5, Background	0.56 g	0.07	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	Negligible, <0.03	0.20

Notes:
[1] Measured as a fraction of gravity, g
[2] Based on the pseudo-static method of Bray and Travasarou (2007)
[3] $D_{84\%}$ = maximum estimated displacement along slip surface at 84% confidence level
[4] Alternative 1a, Alternative 1, and Alternative 3
[5] Alternative 2
M = Earthquake Magnitude
MCE = Maximum Credible Earthquake
PGA = Peak Ground Acceleration
TSF = Tailings Storage Facility
WMP = Water Management Pond
Source: PLP 2019-RFI 008g, -RFI 008i, and -RFI 130

Post-Liquefaction Analysis—As noted above, the bulk TSF main embankment design would result in a serrated near-vertical upstream face at the dam crest for the upper 280 feet of the embankment that would partially rest on tailings. The potential for this configuration to liquefy was initially reviewed by geotechnical, tailings, and dam subject matter experts (SMEs) during the EIS-Phase Failure Modes and Effects Analysis (FMEA) (AECOM 2018l). As discussed, the stability analysis results rely mostly on the strength of rockfill materials directly beneath and downstream of successive raises in the core zone and buttresses versus on the strength of the tailings (see Figure 2-8). In other words, regardless of the low strength assigned to the tailings, the overall embankment did not fail in a downstream direction. Therefore, the SMEs concluded that the likelihood of global instability of the buttressed centerline embankment design would be very low.

Additional preliminary stability analyses were subsequently conducted on the bulk TSF main embankment to further evaluate the potential effects of tailings liquefaction on embankment stability. The methods, input parameters, and results of these analyses are provided in Knight Piésold (2019p) and PLP 2019-RFI 008g, -RFI 008h, and -RFI 130, and are summarized in Appendix K4.15.

As shown in Table K4.15-11, three cases evaluated stability in an upstream direction from the portion of embankment rockfill that is centerline-raised on top of tailings beach material, and six cases evaluated the effect of tailings liquefaction on global embankment stability in a downstream direction. The different cases looked at the effects of: 1) reducing the volume of tailings post-liquefaction due to expulsion of porewater and contraction of the solid particles; 2) varying the depth of liquefaction from 100 feet to the full depth of the tailings; 3) increasing the phreatic surface by assuming that the engineered filter zone is fully blocked; 4) evaluating a slip surface that extends through both the tailings and about half of the embankment; and 5) downstream construction (versus modified centerline).

In all upstream cases, tailings liquefaction would result in some deformation of the embankment rockfill, particularly the upstream edge of rockfill that is constructed on top of the tailings beach. Based on these simulations, the deformations are expected to be constrained in the upstream zone of the dam due to the blocking effect of the tailings, and would not compromise the overall integrity of the embankment. There would be some near-surface deformation effects in the tailings, but their movement in an upstream direction would be limited by the tailings mass in the impoundment. In all downstream cases evaluated, tailings liquefaction did not affect the global stability of the embankment, and the FoS remained well above the target of 1.2 (selected based on Canadian Dam Association [CDA 2014] guidelines).

The current design of the bulk TSF main embankment is considered conceptual, and the post-liquefaction results preliminary. Several sources of uncertainty in these analyses were identified in an independent review by AECOM (2019n). These include the unknown ability of the thickened tailings to segregate into a coarse fraction at the tailings beach, and ensure proper drainage (reduction of the phreatic surface) through the upstream embankment shell and engineered filter zone; the stability of the embankment in the event of liquefaction under static conditions, and during the full duration of ground shaking; methods that take pore pressures into account; additional cases with slip surfaces through both the tailings and embankment; and additional cases that evaluate shallow phreatic surfaces, including where drainage is impeded throughout the downstream rockfill shell.

Given the uncertainties described above, conclusions that post-liquefaction stability would remain above the target FoS are preliminary, and would require additional analysis in the future to demonstrate with confidence. It is acknowledged that some of these analyses can only be completed using numerical modeling techniques, which PLP has committed to completing as design progresses (as described below and in Chapter 5, Mitigation). Recommendations are provided in Appendix M1.0, Mitigation Assessment, for incorporating the above uncertainties into the future liquefaction stability analyses. Further mitigation measures or design changes may be warranted to reduce uncertainties and improve stability, and could be implemented as the design proceeds through ADSP permitting reviews.

Numerical Modeling—The above seismic stability analyses are considered preliminary. As described in Appendix K.4.15, dynamic response analyses using numerical modeling methods would be required to further evaluate potential amplification of seismic waves as they propagate through the foundation material, tailings deposit, and embankments. The application of numerical modeling to the design at its current stage would be inappropriate, because it would rely on ongoing geotechnical analyses and State permitting reviews that have not yet been completed. As described in Chapter 5, Mitigation, PLP has committed to conducting additional detailed modeling, including deformation, settlement, and liquefaction analyses, as well as additional geotechnical investigation and tailings testing to refine input parameters, as part of the ongoing design of the TSFs and other embankments. Additional detailed modeling, including analyses using Fast Lagrangian Analysis of Continua (FLAC) numerical modeling software, would be completed during detailed design of the facilities to better define embankment displacement estimates (PLP 2018-RFI 008a; PLP 2019-RFI 008g).

Post-Closure Phase—As described in Appendix K4.15, the mine site embankments would be designed to withstand an earthquake with a return period up to 10,000 years. Preliminary static, pseudo-static, and post-liquefaction stability analyses have been completed based on end-of-operations conditions when the pond, tailings, and phreatic surfaces would be at their maximum or highest condition. Given that tailings would continue to consolidate, runoff from the closure cover would be promoted, infiltration restricted, and the phreatic surface would drop over time; the results of these analyses are expected to be protective of conditions following closure (PLP 2019-RFI 008g, -RFI 130). As described in Chapter 5, Mitigation, stability and seepage analyses specific to the closure conditions of the facility would be conducted during detailed closure design and would include an independent panel review. These analyses would be updated as required under State permitting throughout the latter stages of operations. Regular mass stability and seepage monitoring would continue throughout closure (PLP 2019-RFI 130; PLP 2019-RFI 135).

Monitoring and Emergency Action Plan (EAP) Requirements—As described in Appendix K4.15, monitoring would be included in all phases of the life of the mine site embankments. This would include construction quality assurance and control plans to assure that the embankments are built according to approved designs; an Operations and Maintenance (O&M) manual describing water management procedures, monitoring, and embankment inspections; and monitoring for mass stability and seepage after closure. An EAP would be prepared per draft ADSP guidelines that would include a dam break analysis with inundation maps and a description of actions to be taken in the event of a dam failure.

Summary of Stability Effects—As described in Section 4.1, Introduction to Environmental Consequences, NEPA requires that potential effects of a project be analyzed in relation to certain factors such as magnitude, duration, extent, and likelihood. The following summary is intended to provide a description of mine site geohazard effects on embankment stability in terms of these factors and is not intended to be a summary of technical engineering and design issues. Due to the conceptual stage of design, the stability analyses described above are considered preliminary, and would require additional analyses in the future to demonstrate with confidence. Uncertainties that remain are described above and in Appendix K4.15, along with mitigations such as future detailed analyses that would be conducted as design progresses to reduce uncertainties.

The magnitude of direct effects on mine embankments from earthquakes, floods, static loading, slope failure, and foundation conditions could range considerably, and are directly related to the likelihood of occurrence. Effects may not be measurable where designs are adequate for expected geohazards with a moderate likelihood of occurrence, such as embankment displacements from moderate earthquakes, large precipitation events, or known unstable foundation conditions that are removed in construction. In terms of duration, effects in the event of an OBE could include damage that would be repairable in the short-term (e.g., months). In the event of an MDE, effects could range up to damage that would not be easily repairable, but would not be expected to lead to structural collapse or uncontrolled release of contaminated materials. Assuming that facilities are planned, designed, constructed, operated, maintained, and surveilled as proposed, in terms of extent, potential damage to facilities and indirect effects on the environment would be expected to remain within the footprint of the mine site. As described in Chapter 5, Mitigation, PLP would establish an independent review board to review embankment designs and stability analyses as engineering analysis progresses.

The duration of effects would vary depending on the facility and likelihood of geohazard occurrence. In the case of earthquake damage that would be easily repairable, impacts would be infrequent, but not longer than the life of the mine for facilities that would be removed at closure (e.g., embankments at the pyritic TSF). Impacts could occur in perpetuity for structures that would remain in place (e.g., bulk TSF embankments). Based on the conceptual designs, and assuming current standard of engineering practice would be followed, the likelihood of global instability of the major embankments was considered to be very low (i.e., less than 1 in 10,000 probability) by geotechnical experts in the EIS-Phase FMEA (AECOM 2018l). Indirect effects on other downstream resources in the unlikely event of an embankment spill or release are discussed in Section 4.27, Spill Risk.

Open Pit Slopes

Unstable pit slopes could lead to operations disruptions, safety hazards to workers, or potential slumping of the pit rim in closure. The location of the water table with respect to the open pit slopes is an important factor in determining their stability during operations and closure. During operations, the water table would be kept back away from the slopes through groundwater pumping or active drains to maintain stability for active mining operations. During closure, dewatering pumps would eventually be turned off, and the water table and pit lake would rise.

Slope Stability Modeling—Numerical modeling was completed by SRK (2012, 2018c, 2019b) and PLP (2018-RFI 023a, 2019-RFI 023b) to predict the stability of four sections of the open pit walls with known weak rock conditions under five water table scenarios in late operations, early closure, and post-closure (Table 4.15-3, Figure K4.15-14, and Figure K4.15-15). As described in Appendix K4.15, the analyses evaluated both static and seismic conditions, and included modeling of disturbance factor zones that represent the predicted bedrock damage caused by blast damage, as well as rock mass relaxation[8] and crustal rebound[9] due to the excavation of the open pit (Hoek 2012). Long-term chemical weathering was taken into account in the assignment of rock strengths (SRK 2019b). The modeling targeted a minimum acceptable FoS for the open pit walls of 1.3 for static conditions, and 1.05 for dynamic (earthquake) conditions. These values recognize that there would only be a single entry into the pit, and any instability involving the ramp could impact the operations. After closure, the target FoS for static conditions would be reduced to 1.1 due to the lack of access required into the pit, but this would be further reviewed during detailed design.

[8] **Rock mass relaxation** is the unloading of rock stresses due to the removal of bedrock (e.g., underground mines and/or open pits).

[9] **Crustal rebound** is the rise of a land mass due to removing an overlying weight or mass, such as excavating bedrock during open pit mining, which could be significant enough to be measurable, and therefore included in the computer modeling.

In terms of magnitude, the modeling results (Table 4.15-3) showed an FoS greater than target values for three of four pit sections (B through D), indicating they would be stable under both static and earthquake loadings. An FoS below target values (indicating potentially unstable conditions) was determined for Section A through the northwestern side of the pit under both static and dynamic loadings in early closure after dewatering ceases ("pumps off" scenario in Table 4.15-3), but before the lake has risen. The unstable results for Section A are associated with weak rock near faults in the lower part of the pit. The results of the continued "active drains" scenario in early closure (see Figure K4.15-16) suggest that with continued depressurization in the localized area of Section A during early closure activities (e.g., backfilling), the pit wall would be stable. The results of the half-full pit lake scenario (see Figure K4.15-17) indicate Section A would be stable after the lake provides a buttressing effect on the lower slopes.

Table 4.15-3: Pit Wall Stability Modeling Results

Section	Operations EoM	Early Closure Phase 1—Active Drains	Closure Phases 1 and 2—Pumps Off	Closure Phase 2—Half Pit Lake	Closure Phases 3 and 4—Final Pit Lake	Operations EoM	Early Closure Phase 1—Active Drains	Closure Phases 1 and 2—Pumps Off	Closure Phase 2—Half Pit Lake	Closure Phases 3 and 4—Final Pit Lake
	Static FoS					Dynamic[1] FoS				
A	1.3	1.3	**0.8**[2]	1.4	1.4	1.2	1.3	**0.7**	1.4	1.4
B	1.6	1.6	1.4	1.4	1.9	1.4	1.4	1.2	1.2	1.7
C	1.4	1.4	1.2	1.2	2.2	1.2	1.2	1.1	1.1	2.0
D	1.4	1.4	1.4	1.3	1.4	1.2	1.2	1.2	1.2	1.1

Notes:
[1] Dynamic stability due to earthquake loading, based on a PGA of half 0.14 g (similar to 1-in-475-year earthquake, Table K4.15-8); use of half PGA derived from documented experiences at open pit mines (Read and Stacey 2009; Azhari 2016).
[2] Bold = potential unstable condition
EoM = end of mine
FoS = factor of safety
Source: PLP 2019-RFI 023b

Sensitivity Analyses—Sensitivity analyses were conducted that evaluated effects on pit wall stability from: 1) increasing earthquake ground shaking levels; and 2) reducing rock strength parameters. These were conducted only on the scenario in Table 4.15-1 with the worst results (i.e., after dewatering ceases ["pumps off" scenario]), but before the lake has risen. The results of increasing ground shaking levels (see Table K4.15-14) indicate that in addition to the unstable condition at Section A during early closure described above, Section D reaches an unstable FoS (below the target criteria of 1.05) at ground shaking levels above a PGA of 0.20g, which is roughly equivalent to the 1-in-1,000-year earthquake (see Table K4.15-8).

Results of reducing the rock strength input parameters in the model by 25 percent showed the potential for increased risk of movement associated with a fault zone higher up in Section A. The approximate area of potential instability reaches about 650 feet back from the pit rim (see Figure K4.15-18 and Figure K4.15-19) and could affect soils and wetlands in this area. The risk of failure along Section A would be highest during the Phase 2 closure period, when the water table is rebounding, but before the lake provides additional buttressing capacity above the backfilled material. This period of time is estimated to be about 15 years in early closure. The results are considered conservative in that they do not take into account the buttressing effect of approximately 1,000 feet of backfilled tailings and waste rock that would be placed in the pit as the water table rebounds (Knight Piésold 2018d: Figure 5.1), although about 450 feet of Section A weak rock would remain exposed above the final lake level without any dewatering or buttressing by the lake. Additional effects from physical weathering (freeze/thaw) could include sloughing at bench crests and inter-ramp slopes but are not expected to result in deep-seated failure (SRK 2019b).

Landslide-Induced Pit Lake Wave—An analysis was conducted to examine the effect of a potential earthquake-induced landslide into the full pit lake in post-closure, and the likelihood that such an event could create a tsunami wave that overtops the pit rim. Tsunamis were computed for two potential landslide scenarios, along Section A and Section D (see Figure K4.15-14), selected because they exhibited the lowest FoS' in the dynamic (seismic) stability analysis in Table 4.15-3 for the full pit lake scenario. The methods, input parameters, and results of the tsunami modeling are presented in AECOM (2019p, 2020) and described in Appendix K4.15. Initial maximum wave amplitudes of about 300 feet (see Figure K4.15-20) were estimated and propagated across and around the lake in the model. These do not overtop the rim, although they reach close to the rim in the slide scenario for Section A.

Other Geohazard Considerations

Quality Assurance/Quality Control (QA/QC)—A Construction QA/QC Plan would be developed to assure all quarries, embankments, impoundments, and liners are constructed and operated in accordance with the approved designs and specifications. The plan would specify actions for approving embankment materials, construction methodology, field testing, surveying, monitoring, and documentation. ADNR (2017a) guidelines provide details on plan requirements, personnel responsible for QA/QC, key inspection items, and required post-construction document submittals.

Mining-Induced Seismicity—Induced seismicity refers to earthquakes and tremors that are thought to be caused by human activity through altering the stresses and strains in the earth's crust. Mining-related activities such as rock mass relaxation, crustal rebound, blasting associated with the excavation of an open pit, dewatering that can reduce load on faults and weaken them, introduction of fluid pressure such as a pit lake, and mass shifts such as rock removal from an open pit or accumulations behind dams, have the potential to generate induced seismicity. Induced seismicity can be associated with altering subsurface porewater pressure in a region known to be cross-cut by faults, as is the case at the Pebble open pit, whether they are active or not. Because some of these are opposing effects (e.g., dewatering versus increased pore pressure), they are complex conditions that are difficult to predict (Klose 2012; McGarr et al. 2002).

The USGS (2018f) compiled a list of mining-related induced seismicity in the US over the 27-year period between 1973 and 2000, during which there was a total of 47 seismic events attributable to mining-related induced seismicity. The recorded tremors were generally small, ranging in magnitude between 2.0 and 4.8. One of the events occurred at the Usibelli Coal Mine in Alaska, with a magnitude 3.3 attributed to blasting, and possibly concurrent rock mass relaxation. Like the Pebble mine site, Usibelli Coal Mine is an open pit operation situated in a seismically active area (WSM 2018). Induced seismicity has been reported in areas of open pits or quarries in the range of less than magnitude 2.5 to 4.6, and higher levels have been documented at large impounded reservoirs in the range of magnitude 4.3 to 6.5 (McGarr et al. 2002).

The open pit slope analysis above assumed seismic conditions that are greater than the highest-magnitude mining-related induced seismic event cited above. For example, the highest ground shaking used in the pit wall seismic sensitivity analysis was 0.30g (see Table K4.15-14), which is roughly equivalent to an earthquake of magnitude 7.5 on the Lake Clark fault (see Appendix K4.15, Table K4.15-9). In addition, the pseudo-static (seismic) stability analysis performed in support of the mine site design (Table 4.15-2) took into consideration unknown shallow crustal earthquakes (Knight Piésold 2019d) up to magnitude 6.5, which is similar to how a large mining-related induced seismic event would likely behave.

Seismic Impacts on Contact Water Pipelines—The EIS-Phase FMEA reviewed the likelihood of a release occurring at the mine site in the event of rupture of a contact water pipeline during an earthquake. These are discussed in AECOM (2018l) and in Section 4.27, Spill Risk. The possibility that such an event could shut down the bulk TSF main SCP reclaim pipeline in post-closure and cause the SCP to fill to the point of overflowing is analyzed in PLP 2019-RFI 130 and described in Appendix K4.15. Based on a range of inflow (dry to wet) and starting water level conditions, it could take anywhere between 3 weeks and 15 months for the SCP to reach capacity after a reclaim pipeline shutdown. As described in Chapter 5, Mitigation, personnel and redundant equipment would be maintained on site throughout post-closure, so that repairs could be conducted as needed (PLP 2019-RFI 130).

Seismic Impacts on Hydrogeology—The potential exists for impacts on hydrogeology resulting from a seismic event, such as changes in groundwater levels, volumes, chemistry, and the location of seeps. However, these types of changes also commonly occur in the absence of seismic events due to other factors such as weather conditions (e.g., precipitation, temperatures) and changes in water chemistry (e.g., precipitation of naturally occurring constituents and/or bacteria in the water).

Groundwater conditions would be monitored throughout all stages of the mine project for both flow and chemistry purposes (PLP 2019-RFI 135; PLP 2019g) (see Section 4.17, Groundwater Hydrology, and Section 4.18, Water and Sediment Quality). The ADSP draft dam safety guidelines include a requirement for an “extraordinary inspection” for impacts if a major earthquake were to occur during project operations (ADNR 2017a). The O&M manual also has specific requirements regarding inspections after a major earthquake. The inspection would identify adherence to design criteria for all major structures to ensure they continue to perform as designed. Changes to the groundwater monitoring program, facility design, and/or operation would be implemented as necessary to ensure protection of the environment.

4.15.3.2 Transportation Corridor

Earthquakes—Surface Faulting and Ground Shaking

The transportation corridor would not cross any known active surface faults (see Section 3.15, Geohazards and Seismic Conditions, Figure 3.15-1). Possible splays of the Lake Clark fault cross the mine access road between the mine site and Eagle Bay terminal (see Figure 3.15-2), and a trace of the Bruin Bay fault zone crosses the port access road within several miles of the Amakdedori port site. However, there is no evidence of Holocene offset at the surface at these locations (Haeussler and Waythomas 2011; Hamilton and Klieforth 2010; Koehler 2010; Koehler et al. 2013; Plafker et al. 1994). Therefore, effects on the road from surface fault displacement are considered unlikely to occur.

As described in Section 3.15, major earthquakes can cause liquefaction along the road corridor in areas of shallow groundwater and liquefiable-type sediments such as silty fine sands. Effects could be like those of the November 2018 Anchorage earthquake, in which a number of roads experienced effects such as buckling, lateral spreading, cracking, ground settlement, and roadbed collapse. These effects could occur at drainages described in Section 3.18, Water and Sediment Quality, as having fine sand and silt substrates along the mine and port access roads and could cause temporary disruption in operations until repairs can be made.

Earthquakes can also cause damage to bridges such as shearing of pilings from liquefaction, settlement, or lateral spreading (Ledezma et al. 2011). However, these types of effects are unlikely to occur at the Newhalen and Gibraltar river bridge crossings, because these drainages contain incised bedrock and boulder-cobble substrates and banks that are not likely to be subject to liquefaction (PLP 2019e, 2020d).

The magnitude of impacts on ferry terminals from ground shaking in the event of a major earthquake would include direct effects such as cracking, spreading, and settlement of terminal platforms, or damage to the ferry during construction. However, because the terminals would not include fuel tank storage facilities, indirect effects on the environment from tank rupture would not be expected.

Seiches and Tsunamis

Earthquake-induced seiches can damage shoreline structures, boats, and moored vessels in enclosed waterbodies, particularly if the natural period of a moored ship matches that of a seiche (Kabiri-Samani 2013). The historical occurrence of seiches in Iliamna Lake is unknown (see Section 3.15, Geohazards and Seismic Conditions) (PLP 2018-RFI 013). In terms of magnitude, seiches several feet high have been documented in Southeast Alaska and in harbors in the Pacific Northwest during past major Alaska earthquakes (McGarr et al. 1968; Barberopoulou et al. 2004; CBJ 2018). However, seiches are more likely to occur in these narrow bodies of water than in Iliamna Lake. A preliminary estimate of seiche potential in Iliamna Lake was conducted based on a 60- by 15-mile area representing the wide part of the lake where the ferry would operate under Alternative 1a (AECOM 2018d). The results indicate the natural oscillation period of an earthquake-induced seiche would fall well outside the period range where earthquake ground motions carry the most energy, suggesting that earthquake-induced seiches would not be expected to occur, or would be on the order of inches. In comparison, wind waves on Iliamna Lake have been documented up to about 6 feet (USACE 2009a).

Tsunamis could also occur in Iliamna Lake from an earthquake-triggered landslide. Examples of landslide-induced tsunami predictions for other inland waterbodies in Alaska include Bradley Lake on Kenai Peninsula and Lynn Canal at Skagway, where wave heights of 10 to 20 feet have been suggested (CASA 1982; Stone & Webster 1987). Well-known rockslide-induced tsunamis with runups well over 100 feet have been documented in saltwater on Uminak Island and Lituya Bay, Alaska (Rozell 2019; Ward and Day 2010).

Although steep slope deposits do not occur near any of the Iliamna Lake infrastructure under Alternative 1a, earthquake-triggered landslides may be possible along coastal areas at the eastern end of the lake, where steep slope deposits occur along Knutson and Pile bays, and potentially steep underwater delta deposits have built up near the mouth of Pile River (Higman and Riordan 2019). A landslide-induced tsunami originating in these areas would be expected to dissipate to the west, as the lake widens away from enclosed bays and islands, which can reflect and trap wave energy locally. Recommendations are provided in Appendix M1.0, Mitigation Assessment, to further evaluate the likelihood of a landslide-induced tsunami originating in the eastern end of the lake to affect the Eagle Bay terminal.

Tsunamis could also occur in Iliamna Lake from underwater fault offset or tectonic tilting, although the likelihood of these occurring is considered less than that of a landslide-induced tsunami. Active faults have not been mapped crossing Iliamna Lake (see Figure 3.15-1). Uncertainties regarding the recency of activity on the closest potentially active fault to Iliamna Lake, the Lake Clark fault, are discussed above under Mine Site, and in Section 3.15 and Appendix K4.15, Geohazards and Seismic Conditions. The potential for tectonic tilting during a magnitude 9 megathrust subduction zone event (similar to the 1964 earthquake) was estimated based on the USGS Slab2 geometric model (Hayes 2018). Tilting during such an event is predicted to be minimal in the Iliamna Lake area (uplift on the order of 1 foot or less) due to the depth of the megathrust in this area (see Figure K4.15-11).

Unstable Slopes

In terms of potential extent of impacts from unstable slopes, several small areas of unstable slope deposits occur along the mine access road: about 2 miles and 6 miles east of the mine site, 3 to 6 miles west of the Newhalen River bridge, on the southern side of Roadhouse Mountain near Eagle Bay, and the southern end of the port access road (see Section 3.15, Geohazards and Seismic Conditions) (Detterman and Reed 1973; Hamilton and Klieforth 2010). Over-steepened, potentially unstable slopes could also be created during the development of the geologic material sites. Landslides can be triggered by earthquakes, exacerbated by precipitation increases caused by climate change, and can cause downstream effects from erosion and sedimentation (e.g., Fan et al. 2019). As described in Appendix K3.1, Introduction to Affected Environment, traditional ecological knowledge (TEK) suggests that increased precipitation and freeze-thaw events have been occurring in the region due to climate change, which could cause increased erosion or risk of landslides along these areas of the transportation corridor.

Typical engineering and construction practices such as engineered cuts, benching, and drainage controls (see Chapter 2, Alternatives, Figure 2-20) would be used at these locations to minimize the potential for landslide impacts on the roads, material sites, and disruption of truck haulage. Therefore, if such effects were to infrequently occur, the duration and extent of impacts on the project and related effects on environmental resources would be easily repairable in the short-term, and of limited extent in the immediate vicinity of the road footprint.

As discussed in Section 3.15, the Eagle Bay ferry terminal location is underlain by bedrock, and the south ferry terminal location by both bedrock and surficial deposits consisting of beach and lake terrace sand and gravel, neither of which is prone to unstable slope conditions (see Figure 2-25).

Based on the topography along the road corridor of Alternative 1a, avalanches would not be expected to occur during mine operations.

4.15.3.3 Amakdedori Port

Earthquakes

Seismic Hazard Analysis—Site-specific seismic hazard analyses were conducted for the Amakdedori port site as described in Appendix K4.15. In terms of magnitude, the predicted ground shaking at the port would be roughly double that predicted for the mine site, reflecting the closer proximity of the port to potential subduction zone earthquakes and the Bruin Bay fault (see Figure 3.15-1 and Figure 3.15-2; and Figure K4.15-10 and Figure K4.15-11). The caisson dock would be designed to withstand an OBE with a return period of 475 years, and an MDE with a return period of 2,475 years (Knight Piésold 2013; PLP 2020-RFI 160). As described in Table K4.15-14, an MDE with this return period would have a PGA of 0.51g at the Amakdedori port site based on the probabilistic analysis. The deterministic analysis (see Table K4.15-15) shows that the biggest contributors to seismic hazard at the port site are the Bruin Bay fault, with a maximum credible acceleration of 1.04g, and the deep intraslab subduction event (similar to the source of the November 2018 Anchorage earthquake) with a PGA of 0.96g.

The type of damage that could occur on a caisson structure during an earthquake might be similar to that experienced during the 1995 Kobe, Japan earthquake (magnitude 7.2), in which concrete caisson walls were among the port structures affected. Ground shaking during the Kobe, Japan earthquake was similar to that predicted for the MDE at Amakdedori (PGA of about 0.5g). The

Kobe structures experienced displacements up to 10 feet laterally and 6 feet vertically due to caissons tilting and pushing out foundation soil, which consisted of imported loose decomposed granite (Nozu et al. 2004).

Caissons are routinely used in high seismic areas throughout the coast of California, Washington, and British Columbia, such as at the Deltaport and Roberts Bank terminals in the Vancouver, BC area, where designs consider loading from seismic accelerations and supporting ground conditions (PLP 2020-RFI 160). As described further below, ground shaking estimates from the seismic hazard analyses are typically used as input to stability analyses to identify how much facility deformation would result during a major earthquake. As a result of these analyses, designs are modified as projects progress to final design to avoid the possibility of global stability failure. As described in Chapter 5, Mitigation, the seismic hazard analyses would be updated in final design, and the geometry and location of the Bruin Bay fault relative to the port site would be further evaluated to refine the deterministic analysis (Knight Piésold 2013, 2019d; PLP 2018-RFI 008c).

In the event of major earthquake damage that temporarily disrupts operations at the port, emergency supplies and equipment would be transferred to onshore infrastructure by landing on Amakdedori beach with a barge or landing craft (PLP 2020-RFI 160).

Container Toppling—It is possible that stacked containers at the port could topple over in a major earthquake and rupture, releasing some of the concentrate. The likelihood of this occurring is considered relatively low, similar to the return periods of major earthquakes at the port site. No toppling effects were reported from stacked containers at the Port of Alaska during the November 2018 magnitude 7.1 Anchorage earthquake, in which PGAs of about 0.3g were recorded near the port (Walker and Murren 2019). This would be similar to the shaking predicted for a 1-in-475-year earthquake at Amakdedori (see Table K4.15-14). In other words, it would likely take a major earthquake with lower likelihood of occurrence to create a toppling hazard.

The concentrate containers would be 6 feet high (shorter than the industry standard of 8 feet) and would be stacked up to three containers high (shorter than the industry standard of five or six containers). Locked pins that fit the containers together would add to stability during ground shaking. In the event that toppling and container spillage does occur, effects on the environment are expected to be similar to those described in Section 4.27, Spill Risk, for concentrate spills.

Stability of Caisson Dock—The types of geohazards impacts that could affect the stability of the caisson dock include foundation or slope conditions, erosion at the base of the caissons, and structural instability such as tilting, cracking, or shearing as a result of seismic loading. Liquefaction of the seabed could also cause damage, although the foundation conditions at the Amakdedori site (described below) may be too coarse and inhomogeneous for liquefaction to occur. Icing and waves that would increase loading on the dock are discussed in Section 4.16, Surface Water Hydrology.

At closure, some of the port facilities would be reconfigured to support a smaller operation, with some terminal facilities and port infrastructure being decommissioned (SRK 2019d). Therefore, the duration of geohazards impacts on the dock would be long-term, lasting throughout operations and possibly into post-closure.

Foundation Conditions—As described in Section 3.15, Geohazards and Seismic Conditions, information on foundation materials at the dock site is limited, but suggests that subsurface deposits consist primarily of sand and gravel with shallow bedrock and buried boulders. Subsurface conditions such as buried sensitive clay layers, which are known to occur elsewhere in Cook Inlet (e.g., at the Port of Alaska) and could increase the risk of sliding failure in an earthquake, likely do not exist at the Amakdedori port site.

Additional geotechnical investigation would be conducted as the project design progresses, and would likely include completing boreholes, rock cores, in situ tests that measure density and other properties (standard penetration tests [SPTs] or cone penetrometer tests [CPTs]), and additional geophysical surveys. It is anticipated that there would be at least one CPT or SPT per caisson location along with representative boreholes along the length of the structure (PLP 2020-RFI 160).

Prior to installing the prefabricated caissons, the seafloor would be prepared to create a level compact surface by excavating 2 to 3 feet of sediment and temporarily storing it on a barge, which would be used to backfill the caissons. Although geotechnical conditions at the port site could be variable, bedrock may be sufficiently deep to allow for sediment excavation during ground preparation (Terrasond 2019).

Stability Analyses—The port design is currently at a conceptual level and stability analyses have not been conducted for the caisson dock. A stability analysis that takes both static and seismic loads into account would be considered state-of-practice for this type of structure in this seismic setting (Alikhani et al. 2003; Matsui et al. 2001). The marine structural design would be developed in general conformance with design and reference standards such those published by American Society of Civil Engineers (ASCE 2014, 2017a), American Association of State Highway and Transportation Officials (AASHTO 2020), and British Standards Institution (BSI 2012) for maritime works; would incorporate industry design and checking standards supervised by professional engineers; and an independent structural/quality review process to ensure conformance with applicable codes and standards (PLP 2020-RFI 160).

PLP would establish appropriate design methodology once the geotechnical program is complete. Liquefaction assessment would be completed initially to determine the type of modeling needed for assessing lateral soil spreading in the event of an earthquake. If more detailed slope stability analysis is necessary, FLAC software may be used to estimate soil movement and overall performance of the structure. Conventional geotechnical design methodologies would be used to determine other parameters applicable to design of the caisson and bridge supporting structures, such as ground-bearing capacity, lateral-slope-sliding resistance, and estimated settlement. Should the seabed conditions be found to be susceptible to liquefaction, ground improvement work would be considered during the design process (PLP 2020-RFI 160).

Erosion Potential—Another potential hazard is that of erosion from currents undermining the base of the caissons. Nearshore tidal and inlet circulation currents are known to occur in Kamishak Bay, as well as seafloor scour near areas of shallow bedrock (Intecsea 2019). The likelihood of scour undercutting the caisson foundation is considered low, given that ground preparation below mudline at the footprint of each caisson would partially key them into the seafloor, and adjacent sediment is expected to backfill around the base of the caissons. The addition of armoring material (e.g., rock rubble) around the foundation could also be considered, depending on the results of further analyses in detailed design.

Environmental Effects—Based on the prefabricated box design of the concrete caissons, release of fill material would not occur from erosion at the base, but could occur in the event of shearing or cracking of the caisson columns during a major earthquake. The fill material for the caissons would be sourced from excavated seafloor material, as well as from a local geologic materials site (blasted granitic material) or imported by ship (PLP 2018-RFI 005), and could range from sand and gravel material (the same as that present on the seafloor) to rockfill. In the event of loss of fill from the caissons, the released material could cause a temporary turbidity plume in the water column. Dock damage in the event of a major earthquake could also disrupt barging and concentrate lightering activities, potentially causing a buildup of concentrate containers at the port and ferry terminals. Additional analyses during detailed design would confirm that port construction, operations, and closure would be protective of the environment.

Unstable Slopes

The Amakdedori port site is underlain by raised beach terrace deposits consisting of sand and gravel (see Section 3.15, Geohazards and Seismic Conditions), which are not prone to unstable slope conditions. The prefabricated concrete caissons would be constructed of different heights to account for the seafloor slope ranging from about -2 feet mean lower-low water (MLLW) at the inshore end of the trestle to about -20 feet MLLW at the offshore end of the dock (about a 1 percent slope). Because the dock and trestle would be constructed to a final elevation of +40 feet MLLW with bridge beams spanning the distance between caissons, the total height of the individual caissons would range from 44 to 63 feet (final elevation + water depth + seafloor excavation). The seafloor slope and different caisson heights would be accounted for in future stability analyses described above (PLP 2020-RFI 160).

Tsunamis

Predicted Runup Elevations and Probabilities—Recent tsunami modeling for lower Cook Inlet (ASCE 2017b) predicts a run-up elevation in the Amakdedori area in the range of 26 to 30 feet above mean high water (MHW), or about 39 to 44 feet above MLLW, with potential seismically induced regional subsidence of about 1 foot, for a very large earthquake with a 2,500-year return period (PLP 2019-RFI 112a). These estimates are based on probabilistic modeling of tectonic sources (e.g., from the megathrust offshore of Kodiak Island), and do not include potential local landslide-induced sources. As discussed in Section 3.15, Geohazards and Seismic Conditions, debris avalanches on the flanks of Augustine Volcano are also estimated to be capable of generating local tsunami wave amplitudes in the range of 5 to 60 feet (Waythomas et al. 2006).

The 2,500-year return period event is the "maximum considered tsunami" in the ASCE (2017a) standards, which specify that certain structures be designed so that they can provide essential functions immediately following this event. The probability of this size tsunami occurring over the life of the port is roughly 1 in 35, assuming the port needs to be operational through closure phase 3, for a total of 70 years (20 years of operations, plus 50 years of closure). Older modeling by Crawford (1987) predicts run-up elevations in the Amakdedori area for smaller, more frequent, medium to large earthquakes (100- to 500-year events) of about 19 to 30 feet MLLW. The probability of a landslide-induced tsunami occurring over the same project life may be as high as 1 in 2, based on past frequency of Augustine Volcano debris avalanches reaching the ocean about every 150 to 200 years (Waythomas et al. 2006), although this estimate does not take into account tsunami size or directional origin (debris avalanche location around the island).

Port Impacts Site-Specific Tsunami Design—If unmitigated, effects from a large tsunami could include risks to worker safety, equipment, and structures, such as the fuel storage tanks, concentrate containers, caisson dock and trestle, trucks, and cranes. Damage during a tsunami could result from initial wave crushing or buoyancy failures, which can cause tipping or sliding of fuel storage and concentrate containers (Brooker 2011). The cross-sectional area of the caissons supporting the dock would be exposed to the hydrodynamic impact of a tsunami wave, and a critical loading condition would be the very low water level during the "retreat phase" of the tsunami, during which the stabilizing effect of water on the outside of the caissons would be absent or diminished. For more frequent smaller tsunamis, predicted run-up elevations would be below that of the port facilities, and the magnitude and extent of impacts on terminal facilities and related effects on the environment would be expected to be similar to waves from large storm events (see Section 4.16, Surface Water Hydrology).

The elevation of the terminal patio and caisson dock was raised to 40 feet MLLW since the Draft EIS to account for tsunami runup potential (PLP 2019b; 2019-RFI 112a). Prior to the final design phase of the project, a formal site-specific tsunami study would be conducted in accordance with ASCE (2017a) standards to provide site-specific maximum run-up, inundation, and current velocity that would be incorporated into final design. The detailed tsunami analysis would include numerical modeling of wave impacts from both seismic and volcanic sources, such as the effects from debris avalanches on Augustine Island. The final terminal elevation would be revisited in final design based on these analyses (PLP 2019-RFI 112; PLP 2019-RFI 112a).

The port diesel fuel facility would be designed to withstand the largest design event. The concrete containment barrier wall around the fuel tank farm (see Figure 2-32) would be designed to protect against tsunami run-up. The effects of potential spill releases from project facilities are discussed in Section 4.27, Spill Risk. A risk analysis would be undertaken for other port components to determine the associated risk level and associated design event. Structures would be designed to withstand tsunami forces, protect against debris impacts such as container interactions, resist uplift, and ensure that scour does not form that could undermine structures (PLP 2019-RFI 112).

In addition to design mitigation, other measures would be employed to reduce risk to personnel, such as early warning systems, vertical evacuation structures, and operational procedures and training on when to move to higher ground and secure critical equipment (PLP 2019-RFI 112).

Impacts to Vessels—Tsunamis can create shipping hazards such as strong currents or areas of sub-tidal rocks exposed by wave drawdown, such as those documented north of the port and offshore of Augustine Island (Intecsea 2019). Some boat damage could result from barge/wharf or barge/ship collisions if loading and lightering activities at the wharf or offshore mooring locations coincide with the arrival of a tsunami wave. However, tsunami warning infrastructure, which typically sends warnings within minutes (NOAA 2018e), may provide enough time to move vessels to avoid these impacts. Advance warning of the potential for local landslide-generated tsunamis from Augustine Volcano is expected to be longer due to tracking of volcanic activity by Alaska Volcano Observatory (AVO).

Impacts to vessels at the two lightering locations would be analyzed during the site-specific tsunami studies to understand the response if a vessel happened to be in place during an event. For the majority of potential events, the vessels would not remain moored. Operational procedures would be in place so that if volcanic activity is predicted or a tsunami warning issued, vessels would cease lightering operations and move to safer locations in deeper water (PLP 2019-RFI 112).

Summary of Tsunami Impacts—In relation to NEPA factors described in Section 4.1, Introduction to Environmental Consequences, the likelihood of a large tsunami occurring at the port ranges from low (i.e., 1 in 35) to moderate over the life of the port, depending on the results of future site-specific tsunami analysis that would evaluate both seismic and landslide sources. The intensity of impacts could range from minimal disruption of activities or boat damage, to terminal flooding and damage to infrastructure, although critical infrastructure such as the fuel tank farm would be expected to remain intact with mitigation in final design. Infrastructure damage would be localized in the near vicinity of the port and mooring sites. The duration of impacts could range from hours to months in the event repairs are required.

Volcanoes

A number of active volcanoes have erupted in the last few decades within about 100 miles of the project area (see Figure 3.15-5). Of particular potential concern is Augustine Volcano, approximately 20 miles east-northeast of the Amakdedori port site. The magnitude of impacts from any of the active volcanoes could include ash clouds transported by wind, and fallout that disrupts construction and operations of project components, depending on prevailing wind direction and plume height. Volcanic ash particles are particularly abrasive, corrosive, and pervasive.

In terms of duration and extent, based on past frequency of eruptions of about 1 in 35 years (Miller et al. 1998), ashfall effects could occur once or twice over the life of the port. Impacts from a volcanic plume could affect both the port facilities and moored ships. The magnitude and extent of direct effects could include damage to equipment, engines, and compressor stations; and disruption of staffing, shipping, and fuel supplies. The duration of effects would be temporary, potentially lasting several days per incident. Ashfall effects on the project would not be expected to result in indirect effects from the facilities on other environmental resources. Typical practices to minimize the effects of an ashfall event would include a vulnerability analysis of facilities and equipment, and hazard planning.

Potential effects from volcanic debris avalanches that flow into Cook Inlet are described above under Tsunamis. The likelihood of these flows reaching the port facilities is considered low.

4.15.3.4 Natural Gas Pipeline Corridor

Earthquakes and Surface Faults

As described above for the transportation corridor, the natural gas pipeline corridor would not cross any known active surface faults (see Figure 3.15-1). Therefore, direct effects on the pipeline from surface fault displacements would not be expected to occur. There is a small possibility that surface displacement could occur on faults previously unrecognized as active, such as splays of the Lake Clark fault (see Figure 3.15-2), causing rupture or other damage to the pipeline. Recommendations are provided in Appendix M1.0, Mitigation Assessment, to conduct special design for fault crossings that may be found to be active in the future.

A major earthquake could cause liquefaction in unfrozen lowlands, stream crossings, and marine areas with fine sandy soils. This condition has the potential to cause buried pipelines to become buoyant; which, if not properly accounted for in design, could lead to pipe flotation and possible damage. The loss of soil shear strength during liquefaction could also lead to permanent ground movements through lateral spreading, flow failure, and settlement. Control measures for liquefaction and buoyancy (e.g., estimation of lateral spreading, use of select compacted backfill, increased cover depth, swamp weights, and post-earthquake inspection) are considered typical state-of-practice for high-liquefaction areas so that design deflection and stress on the pipe would not be exceeded. The use of thicker-walled pipe in marine areas would also help reduce the effects of liquefaction in Cook Inlet and Iliamna Lake.

As described in Chapter 5, Mitigation, additional seismic and liquefaction analyses would be conducted during detailed design to further evaluate the design implications of possible loss of pipeline support from liquefaction (NanaWP and Intecsea 2019a). Therefore, pipe rupture and potential related environmental effects in the event of liquefaction is considered unlikely. If pipe damage were to occur, the extent would be expected to be limited to the immediate vicinity of the liquefaction. The duration of impacts would be short-term, assuming the pipeline could be repaired in a timeframe of days to months.

Unstable Slopes

An unstable bluff roughly 200 feet high exists between the Anchor Point compressor station on Kenai Peninsula and Cook Inlet. As described in Section 3.15, Geohazards and Seismic Conditions, a recent landslide scarp lies within 100 feet of the pipeline route, and bluff retreat rates range as high as 3 feet per year near the town of Kenai. To avoid the bluff, the pipeline would be constructed using horizontal directional drilling (HDD) from the compressor station to the pipeline's emergence point on the Cook Inlet seafloor to the west.

The HDD would begin at an elevation of about 207 feet on the eastern side of Sterling Highway and drop down to an elevation of -12 feet MLLW or deeper[10] in accordance with the Pipeline and Hazardous Materials Safety Administration requirements (PLP 2018-RFI 011). (The exact water depth at which the pipeline would emerge at the seafloor has not been determined, but would be deep enough to avoid navigational hazards [(PLP 2020d]). The downslope elevation of the recent landslide lies about half-way down the vegetated bluff slope (see Figure 3.17-16), which determines how deep the slip surface goes in cross-section view. Based on the typical HDD cross-section in Figure 2-40, there would be about 150 feet of vertical distance between the base of this landslide and the HDD pipeline. Although landslide conditions could vary over time, and the actual HDD location would vary based on the actual angle it follows between entry and exit points, this example illustrates that the HDD methods would likely avoid any existing or future landslides.During the life of the project, the steep bluff at Cook Inlet would likely continue to erode and retreat landward as a result of natural causes. Bluff erosion could become more pronounced in the event of increased precipitation due to climate change; however, even at the high end of historic retreat rates reported at Kenai to the north, the bluff edge is unlikely to reach the HDD entry point hundreds of feet to the east during the project life. With the use of HDD methods, the pipeline is expected to pass well below and landward of the bluff, and avoid the unstable slope hazards (PLP 2018-RFI 011). Therefore, potential impacts on the project and related effects on the environment from this geohazard are expected to be minimal because of this avoidance.

Coastal and Offshore Hazards

Seabed and lake bottom hazards such as movement of boulders on the seafloor, scour from tides and ice, shoreline sediment drift, uneven bottom conditions, or shifting sand waves can cause damage to submerged pipelines, as has occurred with existing oil and gas infrastructure in other areas of Cook Inlet. The minimum depth of pipeline cover above the 12-foot water depth would range from 3 to 5 feet, which would reduce potential effects from these hazards (NanaWP and Intecsea 2019b). The depth of cover west of the HDD installation location, and below the 12-foot water depth on the sides of Cook Inlet and in Iliamna Lake, would be on the order of 1 to 2 feet, which is also considered sufficient to ensure that the top of the pipeline lies below the mudline and avoids these hazards.

About an 11-mile segment of the pipeline route southeast of Augustine Island would not need to be buried to avoid seafloor hazards. This segment would be between 59 and 70 miles west of the eastern side of Cook Inlet in water depths ranging from 155 to 221 feet, which is well below the depth of ice gouge and ice-rafted boulders. Surveys in this area indicate the presence of fine- to medium-grained sand at the seafloor with no boulder fields, outcropping bedrock, or evidence of surface fault or fold deformation near the seafloor, and few third-party risks such as vessel interaction or anchoring (NanaWP and IntecSea 2019a, b). Although the potential for bottom scour was identified in this area, the heavy wall design of the pipeline is predicted to be stable based on on-bottom stability analysis using standard industry software (NanaWP and Intecsea 2019b; PRCI 2019).

In Iliamna Lake, some areas of the lake bottom with uneven terrain in water depths of 33 to 131 feet would require supporting berms to prevent pipeline damage from unacceptable free spanning (see Figure 2-46). The berms would be constructed of engineered fill and rock derived from onshore material sites. The heavy wall pipe is not anticipated to require anchoring to prevent lateral movement off the berms; although if operations monitoring indicates otherwise, mitigation

[10]An 1,800-foot HDD pipeline would exit at approximately -12 feet MLLW, while a 2,000-foot HDD would exit at approximately -18 to -24 feet MLLW. Current technology can accommodate a 2,000-foot HDD for a 12-inch-diameter pipeline (PLP 2018-RFI 011).

might include placement of concrete saddle weights or similar weighting method (PLP 2020-RFI 164).

As described in Chapter 5, Mitigation, industry best practices for inspection and maintenance, such as pigging and offshore remote surveys, would be used during construction and operations to ensure the integrity of the pipeline in the event of loss of cover or support (NanaWP and Intecsea 2019c).

Volcanoes

Effects on the pipeline from Augustine Volcano could include flows during an eruption or debris avalanches reaching the pipeline, and areas of shallow bedrock related to past Augustine Volcano flows that could create construction and operations challenges. Ashfall impacts to aboveground pipeline infrastructure such as the compressor station could also occur from any of the active Cook Inlet volcanoes (see Figure 3.15-5). Ashfall effects would be the same as described above under Amakdedori port.

Based on a preliminary study of past debris avalanches from Augustine Volcano, a 7.5-mile standoff distance between Augustine Island and the pipeline was established to avoid this hazard. This would be confirmed in future design to quantify the probability of a debris avalanche reaching the pipeline, considering seabed gradient. As described above under “Tsunamis,” slope failure at Augustine Volcano could also trigger a tsunami, which could affect pipeline construction and infrastructure at the shore crossings (NanaWP and Intecsea 2019a).

Areas of shallow bedrock and adjacent scour have been mapped along the pipeline route southwest of Augustine Volcano. Embedment and pipeline stability may be challenging in these areas, with the potential for pipeline spans or float-up to occur, and/or pipeline walking, buckling, or vibration. Potential mitigations that could be applied include rock dumping to stabilize the pipeline, strakes (fins that reduce vibration), increased wall thickness, improved weld criteria, or coating design. As described in Chapter 5, Mitigation, additional site-specific investigation and engineering analyses would be conducted to support detailed design and mitigation plans (Intecsea 2019; NanaWP and Intecsea 2019a).

4.15.4 Alternative 1

4.15.4.1 Mine Site

Under Alternative 1, the magnitude, duration, extent, and likelihood of impacts at the mine site would be the same as those described under Alternative 1a. The following section describes impacts for the mine site that would be different under a ferry variant.

Summer-Only Ferry Operations Variant

Under the Summer-Only Ferry Operations Variant, copper-gold concentrate would be stored in shipping containers at the mine site during the winter at a storage area northeast of the pyritic TSF (see Figure 2-59). Based on the surficial geology map (see Figure 3.13-2), the copper-gold concentrate storage area is primarily underlain by surficial glacial outwash deposits, which generally consist of a mixture of sand- to gravel-sized material. The glacial outwash appears to thin to the northeast, with possible bedrock exposed near the northeastern boundary of the storage area.

During a large earthquake, the potential would exist for the stacked shipping containers to be impacted by differential settlement of the underlying glacial outwash due to being thicker to the southwest than the northeast, potentially resulting in toppling of the containers. The likelihood would depend on the magnitude and duration of the seismic event, height of container stacking, density of foundation materials, and other factors. The impact would likely be mitigated through further investigation and foundation preparation such as compaction of near-surface materials.

4.15.4.2 Transportation Corridor

The effects of earthquakes and seiches on the transportation corridor under Alternative 1 would be the same as described above for Alternative 1a.

Lake Tsunamis

As discussed above under "Transportation Corridor" for Alternative 1a, it is possible that an earthquake-triggered landslide could occur along coastal areas at the eastern end of the lake and generate a tsunami wave. Such an event would be expected to dissipate to the west as the lake widens away from enclosed bays and islands, and would be slightly less likely to have an effect on the Alternative 1 north ferry terminal further to the west than the Eagle Bay terminal under Alternative 1a, although the likelihood of occurrence is considered low in both cases. Recommendations are provided in Appendix M1.0, Mitigation Assessment, to further evaluate the likelihood of landslide-induced tsunamis originating in the eastern end of the lake to affect the ferry terminals.

Unstable Slopes

In terms of potential extent of impacts from unstable slopes, several small areas of unstable slope deposits occur along the Alternative 1 mine access road: about 2 and 6 miles east of the mine site on the northern side of Koktuli Mountain (along the portion of the road corridor common to all alternatives); and near the junction between the mine access and Iliamna spur roads (see Section 3.15, Geohazards and Seismic Conditions) (Detterman and Reed 1973; Hamilton and Klieforth 2010). Over-steepened, potentially unstable slopes could also be created during the development of the geologic material sites. There are slightly fewer areas of steep slope deposits along the Alternative 1 roads as compared to Alternative 1a, because the Alternative 1 mine access road avoids the steep section near Roundhouse Mountain.

The Alternative 1 north ferry terminal location is underlain by surficial deposits consisting of beach and lake terrace sand and gravel, which are not prone to unstable slope conditions (see Figure 3.13-4; Section 3.15, Geohazards and Seismic Conditions; and Figure 2-53 and Figure 2-54). Slope conditions at the south ferry terminal and along the port access road would be the same as described under Alternative 1a.

As described above under Alternative 1a, typical engineering and construction practices such as engineered cuts, benching, and drainage controls would be used to minimize the potential for landslide impacts on the roads, material sites, and disruption of truck haulage. Therefore, if such effects were to infrequently occur, the duration and extent of impacts on the project and related effects on environmental resources would be easily repairable in the short-term, and would be of limited extent in the immediate vicinity of the road footprint.

Based on topography along the Alternative 1 road corridor, avalanches would not be expected to occur during mine operations.

Summer-Only Ferry Operations Variant

There would be no difference in the magnitude and extent of geohazard-related impacts under the Summer-Only Ferry Operations Variant. Differences related to lake ice hazards (for year-round versus summer-only ferry operations) are discussed under Section 4.16, Surface Water Hydrology.

Kokhanok East Ferry Terminal Variant

As described in Section 3.15, Geohazards and Seismic Conditions, the Kokhanok East Ferry Terminal Variant location would be underlain by beach deposits near the shoreline and volcanic bedrock farther upslope. The magnitude and potential for seismic effects and unstable slope impacts would be expected to be similar to those of the south ferry terminal location west of Kokhanok (under Alternative 1a and Alternative 1).

4.15.4.3 Amakdedori Port

Stability of Sheet Pile Dock

The port design for Alternative 1 would be to construct a solid earth-filled causeway leading to a sheet pile dock structure filled with granular material. An assessment of the static and seismic stability of the sheet pile dock design is presented in Appendix K4.15 and summarized below. As described above under Alternative 1a, existing geotechnical information regarding foundation materials for the offshore components at the port is limited, and suggests that subsurface deposits would consist primarily of sand and gravel, with possible buried boulders and shallow bedrock. Additional geotechnical investigation would be conducted as the project design progresses (PLP 2018-RFI 005).

The types of geohazards impacts that could affect the rockfill causeway and sheet pile dock and have the potential to result in adverse impacts to the environment include:

- Damage to the sheet pile wall during installation due to the presence of boulders or shallow bedrock in the nearshore sediment, which could result in release of fill during operations.
- Structural instability and potential failure of the sheet pile wharf as a result of seismic loading, foundation conditions, liquefaction, erosion at the base of the sheet pile, icing increasing gravity load on the sheets, and corrosion requiring regular monitoring of cathodic protection systems.

Like the caisson dock design under Alternative 1a, with additional field investigation and detailed stability analyses, the sheet pile dock could be designed to withstand geohazards impacts and be protective of the environment. In comparison to the caisson design, the sheet pile dock is more likely to be damaged during construction if boulders or shallow bedrock are present in the subsurface. The sheet pile variant is more likely to lead to a release of fill in the event of construction damage or scour around the base of the sheet pile. Depending on foundation conditions, it is possbile that the sheet pile dock could be more susceptible to instability during a major earthquake, although future seismic stability analyses are expected to mitigate these effects under either alternative. There would be less seafloor disturbance (no excavation) required prior to sheet pile dock installation, but its footprint would cover about a five times larger area of the seafloor than the caisson dock (PLP 2019b).

In the event of a tsunami, the sheet pile bulkhead would potentially have more cross-sectional area exposed to hydrodynamic and drawdown forces than the caisson design, depending on wave direction. Wave impacts and flooding of the earthfill causeway during a tsunami would be expected to cause little damage and erosion, because riprap would be used to protect the sides, and would be designed to resist tide buoyancy and storm impacts (PLP 2018-RFI 093).

With additional geotechnical investigation and stability analyses, the sheet pile dock design would be refined to address the potential for failure that could lead to adverse impacts on the environment. In the event of geohazard-related dock damage, the extent of possible fill release to the environment would generally be limited to the close vicinity of the dock footprint. As with the caisson dock, some of the port facilities would be reconfigured at closure to support a smaller operation, with some terminal facilities and port infrastructure being decommissioned (SRK 2019d). Therefore, the duration of geohazards impacts on the dock would be long-term, lasting throughout operations and possibly into post-closure.

Stability of Pile-Supported Dock Variant

A pile-supported dock is considered as a variant under Alternative 1 to minimize in-water impacts. The pile-supported dock would be constructed on trestle and dock piles (see Figure 2-63). The footprint area of this variant would be smaller than the other designs, about 5 percent of that required for the caisson trestle and dock, and about 1 percent of that required for the sheet pile variant (PLP 2019b).

As with the caisson and sheet pile docks, detailed engineering analysis has not been completed in support of initial design. Due to the potential for shallow bedrock at Amakdedori, the piles would likely require socketing into bedrock. They may also be more susceptible to marine and icing conditions compared to other dock designs, and would likely require more maintenance, repair, and possible replacement during the project life (PLP 2019b).

The stability of a pile-supported dock is typically a function of structural design details and pile-soil interaction. The current state-of-practice is to use bending in the pile to resist lateral loads (e.g., wind, seismic, vessel impacts, and mooring loads), which may control pile embedment depths. Static stability analysis is typically conducted to determine the ability of the dock to accommodate and control maximum displacements from these loads, as well as global stability issues such as liquefaction. The survivability of a pile-supported structure in a large earthquake is generally considered better than bulkhead-type structures, which do not perform well in major earthquakes, and are difficult to repair. Sections of the existing Port of Alaska pile-supported dock in Anchorage survived the 1964 earthquake, but experienced some damage during the November 2018 Anchorage earthquake, due partly to operating well past its original 35-year design life. The dock experienced spiral weld failure near mudline and cracking of vertical seams during the 2018 earthquake, but no global failure. However, the dock may be at risk of progressive collapse in a future earthquake due to its age, corrosion, and 2018 earthquake damage (Brehmer 2019).

In terms of magnitude of impacts, the pile-supported dock would likely experience more construction difficulties due to shallow bedrock or boulders in the subsurface than the caisson or sheet pile dock designs; have metal corrosion concerns like the sheet pile dock; and ice-related impacts that could be worse due to exposure of the piles to the elements (PLP 2018-RFI 071, PLP 2019b). As with the caisson and sheet pile designs, additional geotechnical investigation and stability analysis would be performed during final design, and the results would provide a better understanding of dock behavior in response to geohazards, and how much shallow bedrock would hinder pile installation.

Based on the conceptual level of design and experience with similar structures, given appropriate maintenance attention, the likelihood of stability issues would be generally considered low with the pile-supported dock, and survivability in a major earthquake generally greater than the sheet pile dock. Unlike the sheet pile dock, the pile-supported dock would not have the potential to release fill into the marine environment as a result of geohazard-related events. In the event of potential geohazard-related impacts to the pile-supported dock, the duration of effects would range from temporary (e.g., ice loads that would be repairable) to long-term, requiring weeks or months to repair, and the extent would likely be limited to the footprint of the structure.

Summer-Only Ferry Operations Variant

This variant would require increased storage capacity for concentrate containers at the port during the non-summer season. Therefore, in the event of a tsunami, there could be an increased risk of container damage or movement, or debris impacts involving containers. Like Alternative 1a, the terminal would be designed to withstand tsunami forces and protect against debris impacts (PLP 2019-RFI 112).

There would be no difference in other geohazard-related impacts under this variant for this component.

4.15.4.4 Natural Gas Pipeline Corridor

Geohazard-related impacts for the pipeline component under Alternative 1 would have similar effects to those described for Alternative 1a.

Kokhanok East Ferry Terminal Variant

There would be no difference in geohazard-related impacts under this variant for this component.

4.15.5 Alternative 2—North Road and Ferry with Downstream Dams

4.15.5.1 Mine Site—Downstream Embankment

The bulk TSF main embankment under Alternative 2 would be constructed using downstream raises (see Figure 2-64 through Figure 2-66), as compared to the buttressed-centerline design under the other alternatives (PLP 2020d; PLP 2018-RFI 075). Under Alternative 2, the overall downstream slope would be 2.6H:1V, which would be the same as the buttressed-centerline-constructed embankment under the other alternatives. The upstream slope of the main embankment under Alternative 2 would be 2H:1V, versus the upstream slope under the other alternatives that would be a serrated near-vertical upstream face at the dam crest for the upper 280 feet, and partially rest on tailings (see Figure 2-8).

Preliminary Static Stability Analyses—As described in Appendix K4.15, the preliminary static stability analysis for the downstream-constructed main embankment calculated an FoS value on the order of 1.9 to 2.0 under static loading conditions, similar to that of the buttressed-centerline design (see Table K4.15-6), thereby offering minimal additional stability or resilience over the design in the other alternatives. A schematic section of the main embankment at its ultimate height with the predicted potential slip surface is shown in Figure 2-66.

As with the FoS values for the modified centerline embankment, the downstream embankment FoS values are considered adequate for the current conceptual levels of design, for determining low probabilities of instability, and for comparing downstream and centerline embankments. Acceptably reliable FoS values for preliminary and detailed design and final construction package purposes would be refined, based on additional geotechnical investigation of tailings and embankment fill characteristics, during the advanced preliminary and detailed stages of the designs.

The bulk TSF main embankment under Alternative 2 would be raised approximately 25 feet higher (embankment height approximately 570 feet) than the design in the other alternatives to provide equivalent bulk TSF storage capacity. The embankment fill would increase from 78 million cubic yards (yd^3) to 124 million yd^3, and the impoundment footprint area would increase by 119 acres (PLP 2018-RFI 075a). This would result in increased impacts on other resources such as material sites, substrate, and wetlands (see Section 4.13, Geology; Section 4.18, Water and Sediment

Quality; and Section 4.22, Wetlands and Other Waters/Special Aquatic Sites), but would not change the global stability of the embankment.

Preliminary Seismic Stability Analyses—Preliminary pseudo-static (seismic) and post-liquefaction stability analyses were completed for the downstream alternative using the same methods and input parameters described above for the bulk TSF buttressed-centerline embankment (under Alternative 1a, "Seismic Stability Analyses") and in Appendix K4.15. The results of the pseudo-static analysis shown in Table K4.15-11 predict negligible displacement (less than 0.3 foot) of the Alternative 2 embankment in a downstream direction under all earthquake scenarios. In comparison, the results for buttressed-centerline construction (Alternative 1a) are slightly higher (by 0.04 foot) for the two MCEs with the highest ground shaking predictions (deep intraslab and background earthquakes), although displacement estimates are minimal in either case, and would not affect the integrity of the structure. This difference would not be measurable under field conditions and indicates effectively no detectable difference in stability between the two designs (PLP 2020-RFI 071d).

The post-liquefaction stability analysis evaluated the stability of the Alternative 2 embankment in a downstream direction in the event the tailings liquefy in an earthquake. The results (Table 4.15-2 and Table K4.15-11) showed that, like the downstream cases evaluated for the buttressed-centerline dam, tailings liquefaction does not affect the global stability of the embankment, and the FoS remains well above the target of 1.2.

Uncertainties regarding the pseudo-static and post-liquefaction analyses would be similar to those described in Appendix K4.15, and above for the modified-centerline embankment (under Alternative 1a, "Seismic Stability Analyses"). In particular, uncertainties regarding whether tailings would segregate and provide a coarser deposit close to the embankment, resulting in a lower phreatic surface near the embankment, could also affect the stability of the downstream dam. Like the modified-centerline embankment, should Alternative 2 move forward into a further preliminary-level design phase, additional geotechnical evaluation and numerical modeling would still need to be conducted to further evaluate the seismic stability of the embankment, which would reduce these uncertainties.

Data compiled in the late 1900s on global dam failures by several agencies, including the US Environmental Protection Agency, US Committee of Large Dams, and United Nations Environment Program, show that dams built by downstream or centerline construction methods are safer than dams built by upstream construction methods, especially under seismic shaking (ICOLD 2001). Subsequent updated studies by Rico et al. (2007a) and Azam and Li (2010) confirmed these findings. Centerline construction was not cited in Mount Polley TSF failure investigative reports (Morgenstern et al. 2015; Hoffman 2015) as being a causative factor in the failure.

Post-Closure—Like the buttressed-centerline embankment, the downstream-constructed embankment under Alternative 2 would be designed to withstand an earthquake with a return period up to 10,000 years. The preliminary static, pseudo-static, and post-liquefaction stability analyses completed for the downstream alternative are based on end-of-operations conditions when the pond, tailings, and phreatic surfaces would be at their maximum or highest condition. Given that the tailings would continue to consolidate, runoff from the closure cover would be promoted, infiltration restricted, and the phreatic surface expected to drop over time; the results of these analyses would be protective of conditions following closure. Also like the buttressed-centerline alternative, stability and seepage analyses specific to closure conditions would be conducted during detailed closure design and would be updated as required under State permitting throughout the latter stages of operations.

Summer-Only Ferry Operations Variant

There would be no difference in geohazard-related impacts under this variant for this component.

4.15.5.2 Transportation Corridor

Mine and Port Access Roads

Earthquakes—The access roads under Alternative 2 would not cross any known active faults (see Figure 2-64, Figure 2-68, Figure 2-69, and Figure 3.15-1). The location and possible activity of Lake Clark fault splays and Bruin Bay fault that cross the Alternative 2 roads are described under Alternative 1a.

Wide, low-gradient stream crossings or estuaries along the Alternative 2 road, such as at the Pile and Iliamna river crossings or the road along Iliamna Bay, may be subject to liquefaction. The magnitude, duration, and extent of potential impacts on most of the road route related to liquefaction would be similar to those described for Alternative 1a. However, liquefaction or other ground failure effects on bridges across these rivers and the road embankment along Iliamna Bay are more likely to occur than at river crossings with incised bedrock or gravel substrates, such as at the Newhalen and Gibraltar river crossings under Alternative 1a.

Potential tsunami-related impacts in Iliamna Bay would be expected to be less severe than at the Amakdedori port site because Iliamna Bay is more protected and shallower than Amakdedori. However, more transportation infrastructure could be exposed to tsunamis under Alternative 2 with the access road from Williamsport to Diamond Point, lying adjacent to Iliamna Bay.

Unstable Slopes—Several areas of unstable solifluction, colluvium, and landslide deposits have been mapped along the mine access road west of Newhalen River, in the area northwest of Eagle Bay on the flanks of Roadhouse Mountain, along the lakefront south of Knutson Mountain, and at the head of Lonesome Bay. Steep alluvial fan and talus deposits also occur in incised valleys crossed by the eastern portion of the route east of Pile Bay (see Section 3.15, Geohazards and Seismic Conditions) (Detterman and Reed 1973; Hamilton and Klieforth 2010). As described in Appendix K3.1, Introduction to Affected Environment, TEK indicates that the steep slopes and valleys between Pile Bay and Williamsport are well known for landslide and avalanche risks (INL 2019). Rockslides and rockfall hazards could occur in this area where exposed bedrock and road cuts would be likely. Rockfall is evident along the steep coastal slopes between Williamsport and Diamond Point; therefore, unstable slopes and rockfall hazards would also be expected along this waterfront section of the road.

As noted above, landslides can be earthquake-triggered; could become more frequent with increased precipitation due to climate change; and could create related erosion and sedimentation effects downstream. Given the numerous steep unstable slopes at the eastern end of the lake, these types of related effects are more likely to occur under Alternative 2 than under Alternative 1a or Alternative 1.

Typical engineering and construction practices, such as engineered cuts, benching, drainage controls, and road maintenance, would be used to manage unstable slopes and reduce the potential for landslide impacts during construction, and disruption of truck haulage. Several locations along the existing Williamsport-Pile Bay Road would be rerouted under this alternative to avoid steep slopes, including approximately the eastern third of this area, and a short road segment close to Pile Bay. Unstable slopes could also lead to an increase in the likelihood of spills (Section 4.27, Spill Risk, provides an analysis of spill impacts from a truck spill scenario). The likelihood of such effects occurring would be expected to be greater for Alternative 2 as compared to Alternative 1a or Alternative 1, because there would be more areas of unstable slopes associated with the transportation corridors under Alternative 2. However, in terms of duration and extent, with appropriate designed engineering controls in place during construction and operations, impacts on the project and related effects on environmental resources would be repairable over the short-term, and limited to the immediate vicinity of the road footprint.

The potential exists for avalanches to occur along portions of the road alignment between Williamsport and Pile Bay. The occurrence of avalanches and landslides could become more frequent over time if climate change causes increased precipitation as rain or snow. Avalanches are expected to be managed using relevant best management practices (BMPs) such as hazard mapping, forecasting, and blasting if necessary. Recommendations are provided in Appendix M1.0, Mitigation Assessment, to also consider the use of snow sheds along this portion of the road, which could protect against both avalanches and rockfall. In terms of duration and extent, if avalanches were to occur, they would temporarily impact a local portion of the road until the snow could be removed.

Eagle Bay to Pile Bay Ferry

The magnitude, duration, and extent of potential impacts on the ferry terminals related to ground shaking and the potential for tsunamis in Iliamna Lake would be similar to those described above under Alternative 1a for the Eagle Bay ferry terminal. Although the potential for seiche occurrence is considered unlikely (AECOM 2018j), the eastern end of the lake has steeper slopes and is narrower and deeper than the area of Alternative 1a or Alternative 1; factors that can increase the likelihood of an earthquake-triggered landslide-induced tsunami occurring from either a subaerial or submerged source and impacting shore-based infrastructure. Recommendations are provided in Appendix M1.0, Mitigation Assessment, to further evaluate the likelihood of landslide-induced tsunamis originating in the eastern end of the lake to affect the ferry terminals.

Summer-Only Ferry Operations Variant

Under the Summer-Only Ferry Operations Variant, road traffic would be concentrated during the 6-month transportation season, which would include rainy months. Because heavy rain is often a trigger for slope failure, the potential for these impacts on road traffic and spill potential could be slightly greater under this variant, but would be balanced by fewer avalanche impacts due to lack of winter season traffic. Lake ice hazards are discussed under Section 4.16, Surface Water Hydrology.

There would be less potential for impacts to the ferry to occur from landslide-induced tsunamis under this variant than Alternative 2 due to less ferry traffic.

Newhalen River North Crossing Variant

There would be no difference in geohazard-related impacts under this variant for this component compared to the Newhalen River south crossing.

4.15.5.3 Diamond Point Port

The Diamond Point port facility would use the same design concept as the Amakdedori port sheet pile dock under Alternative 1 (see Figure 2-71 and Figure 2-72), although with a footprint about four times bigger than the sheet pile dock at Amakdedori (PLP 2018-RFI 071).

Earthquakes

As discussed in Appendix K4.15, ground shaking potential in the Diamond Point area is slightly greater than at Amakdedori, based on probabilistic seismic hazard predictions that evaluate the potential for earthquakes from all sources (see Table K4.15-14), but is lower than Amakdedori for an earthquake generated specifically on the Bruin Bay fault (see Table K4.15-15). The likelihood of liquefaction effects on dock stability may be higher under this alternative due to the presence of finer-grained sediments in Iliamna Bay as compared to Amakdedori. The Bruin Bay Fault extends along the western shore of Cook Inlet near both the Amakdedori and Diamond Point port sites. Although there is no evidence for Holocene offset at the surface, this fault is associated with several small to moderate earthquakes up to M7.3 in 1943 (Stevens and Craw 2003).

Stability of Sheet Pile Dock

The sheet pile dock at Diamond Point would have the same potential to result in adverse impacts to the environment during construction and operations as discussed above for the Alternative 1 sheet pile dock at Amakdedori, and in Appendix K4.15.

The magnitude of potential impacts for the Alternative 2 sheet pile dock could be greater than the Alternative 1 sheet pile dock due to the larger footprint and fill volume required for the Alternative 2 dock, and possible higher likelihood of boulders in the subsurface with related risk of short embedment or sheet pile damage. In addition, there could be added complexity to foundation condition effects, dock stability, and construction issues near the northwestern corner of the dock, where a 350-foot-long section of the dock would be installed immediately adjacent to the dredged turning basin (see Figure 2-71), resulting in a 10-foot elevation change at the seafloor on either side of the dock and along the southern dock face to the east at the edge of the dredged area. Foundation conditions could be different on either side of the dock in this area or along the dock face, and construction in this area may require varying sheet pile heights or embedment depths.

As described in Section 4.18, Water and Sediment Quality, substrate conditions are generally finer-grained in Iliamna Bay than in Kamishak Bay. Because dock fill would partly consist of dredged material, in the event that potential geohazard-related impacts cause a release of fill to the marine environment, the extent of redeposition could be greater than under Alternative 1, and could range widely depending on season, tides, and wave conditions (e.g., from the close vicinity of the dock structure to the mouth of Iliamna Bay).

As with Amakdedori port, some of the Diamond Point port facilities may be reconfigured at closure to support a smaller operation with some terminal facilities being decommissioned (SRK 2019d), although it is possible that the Diamond Point port would be operated after mine closure by another entity (see Chapter 2, Alternatives). Therefore, the duration of potential geohazards-related impacts would be long-term, and the extent would generally be limited to the close vicinity of the dock footprint. With additional geotechnical investigation and stability analyses, the sheet pile dock design would be refined to address the potential for failure that could lead to adverse impacts on the environment (PLP 2018-RFI 005).

Stability of Pile-Supported Dock Variant

The Pile-Supported Dock Variant for the Diamond Point port would have potential geohazard-related impacts similar in magnitude, duration, and extent as the Pile-Supported Dock Variant at the Amakdedori port under Alternative 1. The offshore foundation conditions would likely be different than the Amakdedori site but are also likely to include buried boulders and/or areas of shallow bedrock, which could affect the constructability and overall performance of the pile-supported system. If this variant is chosen, field conditions would be further investigated in support of final design.

Unstable Slopes

As described above for the Alternative 2 transportation corridor, steep unstable slopes, rockfall, and avalanche hazards would be expected along the Diamond Point-Williamsport waterfront section of the road leading to Diamond Point port, and could also be present along the slopes above the port terminal and dredge material storage areas where steep alluvial fan deposits have been mapped (see Figure 2-64) (Detterman and Reed 1973). The potential for slope instability could be exacerbated in the event of increased precipitation due to climate change.

Typical engineering and construction practices such as foundation improvements, benching, and drainage controls, are expected to be employed during port design to manage unstable slopes and reduce the potential for impacts on the terminal and material storage areas. The material storage areas would be constructed with berms on their downslope sides, which are expected to prevent downslope movement or erosion effects from the storage areas.

Tsunamis

The magnitude, duration, extent, and potential for tsunami impacts at the Diamond Point port site would be similar or slightly less than those at the Amakdedori port site under Alternative 1a and Alternative 1. The predicted run-up elevation for the 2,500-year event is slightly less for Diamond Point (36 to 39 feet MLLW) than at Amakdedori (39 to 44 feet MLLW) (see Section 3.15, Geohazards and Seismic Conditions). The potential for landslide-generated tsunamis from Augustine Volcano affecting the port site and lightering locations would be considered similar to Amakdedori, because historic events have occurred radially around Augustine Volcano (see Figure 3.15-5). However, the potential for local landslide-generated events originating from the slopes of Cottonwood, Iliamna, or Iniskin bays could be greater under Alternative 2 than at Amakdedori due to the presence of steep slopes and narrower bodies of water in this area. The engineering analyses and mitigation in final design that would occur at Amakdedori based on ASCE (2017a) industry standards (PLP 2018-RFI 112; PLP 2018-RFI 112a) would be the same for Diamond Point, assuming the additional infrastructure at this port site (dredge material storage area and roads) would be included in the site-specific tsunami analysis.

Volcanoes

The Diamond Point port location would be approximately the same distance from volcanoes in the area, including Augustine Volcano, as the Amakdedori port under Alternative 1a and Alternative 1. Therefore, the likelihood of impacts occurring would be similar, with the magnitude, duration, and extent of impacts dependent on the severity of an ash cloud and the wind direction at the time of an eruption. In winter, the magnitude, duration, and extent of potential impacts from Augustine Volcano on the Alternative 2 port site could be greater than at Amakdedori due to dominant northwesterly winds in this area (Knight Piésold 2018g).

4.15.5.4 Natural Gas Pipeline Corridor

Referring to Figure 2-73, natural gas pipeline construction under Alternative 2 would follow a different corridor route west of Cook Inlet, and would therefore encounter different geology and related potential geohazards than Alternative 1a and Alternative 1 (see Section 3.13, Geology and Section 3.15, Geohazards and Seismic Conditions).

Earthquakes and Surface Faults

In western Cook Inlet, the Alternative 2 pipeline would be routed to Ursus Cove to avoid known rocks and boulders at the mouth of Iliamna Bay (PLP 2018-RFI 063). At about 3 miles before making landfall, the pipeline would cross a mapped fault trace of the potentially active Bruin Bay fault (see Figure 3.15-1). Additional field investigation prior to final design (e.g., an offshore geophysical survey or onshore fault study at Ursus Head where the fault is mapped as having an upland component), would be needed to identify whether the fault is active and whether potential displacement mitigation in design would be necessary, if this alternative were to be selected.

As described above under the Alternative 2 transportation corridor, the potential for liquefaction effects along the Alternative 2 pipeline route may be higher in some areas where the route crosses wide alluvial or estuarine deposits, such as the Pile Bay and Iliamna river crossings, along the Diamond Point-Williamsport section of road, and between Ursus Cove and Cottonwood Bay. The type of impacts on the pipeline in the event of liquefaction are described above under Alternative 1a.

Unstable Slopes

Steep unstable slopes are a known hazard to pipeline integrity, and have been known to cause operation interruptions and ruptures in other mountainous areas of the world (e.g., the Andes, Eastern Europe, and Sakhalin Island) (Lee et al. 2016). Unstable slopes mapped between Ursus Cove and Pile Bay, and for the Alternative 2 route west of Eagle Bay, are discussed above under the Alternative 2 transportation corridor. The pipeline segment between Pile Bay and Eagle Bay crosses areas of exposed steep bedrock with the potential for rock instability, and alluvial fan and talus deposits, which could be unstable on steeper slopes. The corridor would avoid mapped landslide deposits on the flanks of Knutson and Roadhouse mountains.

Typical engineering and construction practices such as engineered cuts, rock stabilization, benching, and drainage controls would likely be used at these locations to reduce the potential for rockslide and landslide impacts to the pipeline. Additional measures, such as long-term slope monitoring and inspections, may be necessary in select areas. With these controls, the likelihood of slope failures occurring during construction and operations that would affect pipeline integrity would be expected to be minimal. In terms of duration and extent, related effects on environmental resources would also be expected to be minimal, repairable in the short-term, and limited to the immediate vicinity of the pipeline right-of-way (ROW).

Coastal Hazards

The depth of the pipeline as it approaches Ursus Cove from Cook Inlet, as well as the underwater crossing of the bay to Diamond Point, would be sufficient to ensure that the top of the pipeline lies below the mudline. The minimum depth of cover above the 12-foot water depth would be 3 feet, which would be expected to reduce potential effects from coastal hazards, such as shoreline drift, ice gouge, or ice-rafting of surface boulders.

4.15.6 Alternative 3—North Road Only

Under Alternative 3 and its variants, the magnitude, duration, extent, and likelihood of impacts at the mine site (including concentrate pumphouse) would be the same as those for Alternative 1a. The impacts from the natural gas pipeline corridor would be the same as those described under Alternative 2. The following section describes impacts for the transportation corridor and port that would be different under Alternative 3 and its variants.

4.15.6.1 Transportation Corridor

All Road Routes, Mine Site to Port

Geohazards-related impacts resulting from construction and operation of the Alternative 3 north access road from Diamond Point to the mine site would be generally the same as the combination of road and natural gas pipeline corridors described under Alternative 2. However, the likelihood of slope stability issues occurring along the all-road route would be higher between Eagle Bay and Pile Bay than under Alternative 2, due to the wider road ROW (compared to the Alternative 2 pipeline-only in this area), and greater need for engineering controls (such as wider cut-and-fills) to mitigate potential slope impacts. There would also be a slightly higher likelihood of spills due to the longer road route through steep terrain (Section 4.27, Spill Risk provides an analysis of spill impacts from a truck spill scenario), and greater potential for avalanche impacts to occur that would be preventable using relevant BMPs described above for Alternative 2.

Typical engineering controls and BMPs described above would reduce the likelihood of slope failures and avalanches occurring along the all-road route. In terms of duration and extent, related effects on environmental resources would be expected to be repairable over the short-term (days or weeks), and limited in extent to the immediate vicinity of the access road ROW footprint. Recommendations are provided in Appendix M1.0, Mitigation Assessment, to also consider the use of snow sheds along the road to protect against avalanches.

The likelihood of a potential landslide-induced tsunami in Iliamna Lake impacting Alternative 3 shore-based infrastructure would be less than other alternatives because there would be no ferry terminals, but there could be effects on the road where it is close to shore along the eastern part of the lake. Recommendations are provided in Appendix M1.0, Mitigation Assessment, to further evaluate the likelihood of landslide-induced tsunami originating in the eastern end of the lake to affect the transportation route.

Concentrate Pipeline Variant

Because the concentrate pipeline would be installed in the same trench as the natural gas pipeline, the magnitude, duration, extent, and likelihood of impacts from geohazards, such as unstable slopes, would be similar to the Alternative 2 natural gas pipeline corridor and Alternative 3 all-road route. There would be a slightly higher likelihood of minor spills due to the additional potential contaminant source from the concentrate pipeline along steep terrain, which would be partially mitigated through leak detection systems (Section 4.27, Spill Risk, provides analysis of spill impacts from a concentrate spill scenario).

4.15.6.2 Port North of Diamond Point

Geohazard-related impacts would generally have a similar magnitude, duration, extent, and likelihood as those described for Alternative 2, except for the effects described below for the shore-based port facilities, caisson dock, and concentrate storage.

Port Facilities

Unstable Slopes—The shore-based port facilities under Alternative 3 would be located several miles north of the Alternative 2 port location in a narrow strip of surficial deposits backed by steep cliffs. Rockslides and rockfall are more likely to occur here than at the Alternative 2 port, conditions which would be exacerbated during a major earthquake. Cut slopes into bedrock would be necessary during construction to accommodate the port facilities (see Figure 2-81). Typical rock slope design and maintenance techniques such as benching and drainage controls would be incorporated into final design and operations to mitigate this impact.

Tsunamis—The magnitude, duration, extent, and potential for tsunami impacts at the Alternative 3 port facilities site would be similar or greater than those at the Amakdedori port site and the Alternative 2 Dimond Point port site. The predicted run-up elevation for the 2,500-year event is greater at the Alternative 3 port site (45 to 47 feet MLLW) than at the Alternative 2 port site (36 to 39 MLLW) and at Amakdedori (39 to 44 feet MLLW) (see Section 3.15, Geohazards and Seismic Conditions). The potential for landslide-generated tsunamis affecting the port site would be similar to Alternative 2. The engineering analyses and mitigation in final design that would occur at Amakdedori based on ASCE (2017a) industry standards (PLP 2018-RFI 112; PLP 2018-RFI 112a) are expected to be the same under Alternative 3.

Caisson Dock

The dock under Alternative 3 would be constructed in a similar manner as described for the caisson dock under Alternative 1a at Amakdedori, with similar-sized individual caisson footprints and separations between caissons, which would support a concrete deck. The Diamond Point causeway is shorter than at Amakdedori and would require fewer 60-foot by 60-foot caissons, but more of the larger 60-foot by 120-foot caissons than at Amakdedori.

Foundation Conditions and Dock Stability—The causeway under Alternative 3 would be constructed in shallower water than at Amakdedori or at Diamond Point under Alternative 2, extending from shore to about -4 feet MLLW. The Alternative 3 dock caissons would be placed in water depths of -18 feet MLLW, along the sides of a turning basin dredged into native seabed materials ranging from -3 to -6 feet MLLW.

As described in Section 3.15, Geohazards and Seismic Conditions, foundation conditions for the caissons under Alternative 3 would likely include mostly silt with less than 30 percent sand and gravel and occasional boulders. Bedrock is not expected to be present to a depth of more than 100 feet (PLP 2020d). Any boulders encountered in the dredge basin and channel would be removed and used in shore-based construction or placed in the dredge stockpile. Prior to installing the caissons under Alternative 3, the seafloor would be prepared by excavating approximately 5 feet of sediment below the turning basin to create a level, compact surface; this would be a 2- to 3-foot deeper foundation excavation than at Amakdedori, likely due to the presence of finer deposits in Iliamna Bay. The caissons would be backfilled with coarse material separated from the dredged sediments plus additional coarse material from onshore quarries, sized to achieve proper compaction to avoid settlement.

The finer-grained seafloor material at Diamond Point, possible presence of buried boulders in the subsurface, and 12- to 15-foot elevation change between the northwest and southeast sides of the caissons present potentially more complex geotechnical conditions for stability analysis and founding the caissons than at Amakdedori, although conditions would be similar to those described for the Alternative 2 sheet pile dock. It is expected that additional geotechnical investigation would be conducted as the project design advances to confirm foundation conditions, and that these conditions would be at least partly mitigated by the deeper foundation excavation and caisson placement. The types of geotechnical investigations and stability analyses conducted for the dock under Alternative 3 are expected to be similar to those described under Alternative 1a and in PLP 2020-RFI 160. As with Alternative 1a, ground improvement work would be considered during the design process if necessary, based on the additional investigation and analyses (PLP 2020-RFI 160).

Earthquakes—Ground-shaking potential and the likelihood of active surface fault displacement under Alternative 3 would be similar to that described under Alternative 2. There could be a higher risk of liquefaction effects on the caisson dock under Alternative 3 than described for the caisson dock at Amakdedori (under Alternative 1a), due to the finer-grained seabed material in Iliamna Bay. Liquefaction assessment would be completed in the early stages of design to determine which modeling methodologies are required for lateral spreading in a seismic event. Dredge slopes of 4H:1V are proposed to address sediment stability and the potential for seismic-induced slumping on the sides of the turning basin (PLP 2020d). If more detailed slope stability analysis is required, FLAC software may be used to estimate the soil movements and overall performance of the structure, and ground improvements may be considered during the design development process (PLP 2020-RFI 160).

Tsunamis—Impacts to the caisson dock under Alternative 3 would be similar to those described under the Alternative 1a caisson dock. In the event of a tsunami, the caisson dock would potentially have less cross-sectional area exposed to hydrodynamic and drawdown forces, and possibly be less susceptible to damage in a tsunami, than the sheet pile design under Alternative 2.

Other Impacts—Erosion potential at the base of the caissons due to tidal currents would be similar to that described for the caisson dock under Alternative 1a. Seafloor sediment at the Alternative 3 dock is more likely to build up on the dredged basin side of the caissons than erode, due to tidal currents and the 12- to 15-foot elevation change between the native seabed and dredge basin. Maintenance dredging would be conducted on a periodic basis to keep the channel and basin open as required for vessel draft.

The likelihood of impacts from the release of fill material in the event of shearing or cracking of the caisson columns would be similar to those described for the caisson dock under Alternative 1a. Unlike Amakdedori, however, any released material would likely be coarser than the surrounding seabed sediment and would be derived from a combination of subsea and onshore sources.

Concentrate Pipeline Variant

Due to the presence of steep bedrock cliffs adjacent to the footprint of the concentrate storage facility, the potential for unstable slopes and rockfall would exist during construction and operation. If this variant were selected, the final design would typically include a geotechnical investigation to confirm foundation and slope conditions to ensure the facility construction and operation would mitigate unstable slopes.

As noted above under Alternative 3 "Port Facilities," impacts from a tsunami at this location would be similar to or greater than at the Amakdedori port and the Diamond Point port site under Alternative 2 due to a higher predicted runup elevation. If a tsunami were to occur, it would have a higher potential to result in a contaminant release to the marine environment under this variant, because this variant includes bulk storage of concentrate and the others do not. Section 4.27, Spill Risk, provides analysis of spill impacts from a concentrate spill scenario. The duration of impacts could range from hours to months in the event repairs would be required. As described in Chapter 5, Mitigation, practices that would minimize these effects would include site-specific tsunami analysis and design, incorporation of flooding into design (e.g., tie-downs), emergency action planning with tsunami escape routes, or consideration of design changes to facility armoring and elevation.

4.15.6.3 Natural Gas Pipeline Corridor

Geohazard-related impacts would have a similar magnitude, duration, extent, and likelihood as those described for Alternative 2. The offshore portion of the Alternative 3 pipeline is about 1 mile longer than that of Alternative 2, and thus, would be slightly more likely to encounter coastal hazards such as boulders on mudflats or liquefaction effects.

4.15.7 Cumulative Effects

Seismic and other geologic hazards (geohazards) range from slope instability in the immediate vicinity of the project footprint to earthquakes and volcanoes in the region that could affect project facilities from long distances (see Section 4.27, Spill Risk, for a discussion of risk of dam failure). The cumulative effects analysis area for geohazards encompasses the footprint of the Pebble Project, including alternatives and variants, the expanded mine footprint (including road, pipeline, and port facilities), and any other reasonably foreseeable future actions (RFFAs) in the vicinity of the project that would result in potential synergistic and interactive effects. In this area, a nexus may exist between the project and other RFFAs that could contribute cumulatively to geologic hazards-related impacts. Section 4.1, Introduction to Environmental Consequences, details the comprehensive set of past, present, and RFFAs considered for evaluation as applicable. Several the actions would be considered to have no potential of contributing to cumulative geologic hazard effects in the analysis area. These include activities that may occur in the analysis area, but are unlikely to result in any appreciable cumulative effect with regard to geohazards, or actions outside of the geologic hazards cumulative effects analysis area.

4.15.7.1 Past and Present Actions

Past and present actions in the analysis area would not be expected to contribute cumulatively to geologic hazards. Although past or current actions in the analysis area have included some minor earthworks, the effects are minor both in magnitude and extent, and are not expected to be a significant factor in increased geologic hazards. Similarly, although there have been past volcanic and earthquake events in the region, they have not contributed to any increased geologic hazard risk in current conditions.

4.15.7.2 Reasonably Foreseeable Future Actions

RFFAs in the analysis area that would involve earthworks resulting in possible geohazards-related impacts and that could contribute cumulatively to geohazards include the Pebble Project expansion scenario; mining exploration activities for Pebble South and Groundhog mineral prospects; onshore and offshore oil and gas development; and Lake and Peninsula transportation and infrastructure projects such as road improvements and continued development of the Diamond Point Rock Quarry.

The No Action Alternative would not contribute to cumulative geologic hazard effects.

Collectively, the project alternatives with RFFA contribution to increased geohazards are summarized in Table 4.15-4.

Table 4.15-4: Contribution to Cumulative Effects from Geohazards

Reasonably Foreseeable Future Actions	Alternative 1a	Alternative 1 and Variants	Alternative 2 and Variants	Alternative 3 and Variant
Pebble Project Expansion Scenario	**Mine Site:** The mine site footprint would have a larger open pit and new facilities to store tailings and waste rock, which would contribute to cumulative effects on and from geohazards through removal of overburden and bedrock, and construction of potentially unstable embankments, stockpiles, and pit walls. The expansion scenario and associated infrastructure would be similar for all alternatives. New facilities requiring consideration of static and seismic stability in design would include a southern bulk TSF with flow-through embankment containing an additional 4.6 billion tons of tails; a southern PAG TSF containing 0.6 billion tons of additional pyritic tails; northern and southern WRFs containing an additional 17 billion tons of NAG and PAG waste rock; and water/seepage collection ponds downgradient from these storage facilities (see Table 4.1-2) (PLP 2018-RFI 062). If the potential for expansion is foreseen before closure of the original pyritic TSF, filling the open pit would be reconsidered and the original pyritic TSF would likely remain in its currently planned form. If expansion occurred after closure and transfer of the original pyritic TSF materials to the pit, tailings removal from the pit and transport/placement techniques used at other mine closures could be considered (e.g., Tundra Mine in Northwest Territories, Centralia Mine in Washington). The new TSFs and southern WRF would be sited in geomorphically constricted valleys between exposed bedrock ridges south of the TSFs and pit that drain towards the SFK. The northern WRF would be sited in a broader area of glacial deposits draining towards UTC with an exposed bedrock ridge on the northern side. Based on geologic maps of the area (see Figure 3.13-1 through Figure 3.13-4 in Section 3.13, Geology; and Figure 3.17-1 in Section 3.17,	**Mine Site:** Identical to Alternative 1a. **Other Facilities:** Similar to Alternative 1a, except that the portion of the mine access road from about 10 miles west of the Newhalen River to the Eagle Bay area would not already be in place, and would be constructed along with the rest of the north access road from Eagle Bay area to the new Iniskin Bay port. **Magnitude:** Cumulative impacts from geohazards would be similar to that of the Alternative 1a. **Duration/Extent:** The duration and extent of cumulative impacts from geohazards would be similar to those of Alternative 1a. **Contribution:** The contribution to cumulative effects would be similar to that of Alternative 1a.	**Mine Site:** Similar to the Alternative 1a. **Other Facilities:** The north access road would be extended east from the Eagle Bay ferry terminal to the Iniskin Peninsula. Concentrate and diesel pipelines would be constructed along the north road alignment, all of which would be extended to a new deepwater port site at Iniskin Bay. There would be increased unstable slopes along the Eagle Bay-to-Pile Bay segment, and both unstable slopes and potential liquefaction effects along the extended Williamsport-to-Iniskin Bay segment and deepwater port, due to the presence of steep talus deposits and wide alluvial/estuarine valleys in this area (see Figure 3.13-4 in Section 4.13, Geology). **Magnitude:** Cumulative geohazard impacts from mine expansion would be less than that of the Alternative 1a overall, given that a portion of the north road and all of the gas pipeline would already be	**Mine Site:** Identical to the Alternative 1a. **Other Facilities:** Overall expansion would use the existing north access road; concentrate and diesel pipelines would be constructed along the existing road alignment, all of which would be extended to a new deepwater port site at Iniskin Bay. Like Alternative 2, there could be increased unstable slopes and liquefiable areas along the new pipeline/road segment between Williamsport and Iniskin Bay. **Magnitude:** Cumulative geohazard impacts from mine expansion would be less than that of the other alternatives overall, given that the north access road and gas pipeline would already be constructed. However, there would be more critical facilities concentrated in areas of unstable slopes and liquefiable ground, increasing the likelihood of disruptions to transportation/pipeline

Table 4.15-4: Contribution to Cumulative Effects from Geohazards

Reasonably Foreseeable Future Actions	Alternative 1a	Alternative 1 and Variants	Alternative 2 and Variants	Alternative 3 and Variant
	Groundwater Hydrology), foundation conditions at the new embankments and WRFs would likely be similar to those of the proposed facilities; i.e., fractured and faulted Cretaceous granodiorite and younger volcanics overlain by mostly glacial moraine deposits with minor areas of colluvium and solifluction deposits. Like the proposed embankments, the new TSF embankments would likely be founded on bedrock. There could be increased stability concerns for the WRFs and embankments of smaller ponds if founded on potentially unsuitable overburden, which would be addressed during detailed design under ADNR permitting. The new facilities may be closer to potentially active traces of the Lake Clark fault (see Figure 3.15-2 in Section 3.15, Geohazards), particularly in the case of the northern WRF, and would require additional seismic hazard analysis and possibly additional surface fault investigations. The magnitude of potential geohazard-related impacts would be higher than that of the project, due to added stability risk and potential cumulative effects on the SFK and UTC drainages from the new TSFs, WRFs, and larger pit that would be required in the Pebble Project expansion scenario. There would be about 60 years of additional design life for certain structures (e.g., pyritic TSF, main WMP, and port) that would need to remain beyond their original design life to wait for the pit to be available for backfill, which would require additional consideration in stability analyses, engineering reviews, and potential structural mitigations as operations and closure design advances. **Other Facilities:** A north access road and concentrate and diesel pipelines would be constructed under all alternatives with the Pebble Project expansion scenario, extending along the Alternative 3 road alignment from the Eagle Bay area		constructed, and the south access road would not be needed. However, there would be more critical facilities (e.g., roads and pipelines) concentrated in areas of unstable slopes and liquefiable ground, increasing the likelihood of disruptions to transportation/pipeline systems and spill risk during earthquakes. **Duration/Extent:** Cumulative impacts from geohazards would be similar in duration to the other alternatives, although affecting a smaller area and fewer watersheds than Alternative 1a and Alternative 1. **Contribution:** The contribution to cumulative impacts would be similar to the other alternatives, although affecting a smaller area and fewer watersheds than Alternative 1a and Alternative 1.	systems and spill risk during earthquakes. **Duration/Extent:** The duration of cumulative impacts from geohazards would be similar to that of the other alternatives, although affecting a smaller area and fewer watersheds. **Contribution:** The contribution to cumulative impacts would be similar to the other alternatives, although affecting a smaller area and fewer watersheds.

Table 4.15-4: Contribution to Cumulative Effects from Geohazards

Reasonably Foreseeable Future Actions	Alternative 1a	Alternative 1 and Variants	Alternative 2 and Variants	Alternative 3 and Variant
	to the Pile Bay terminus of the Williamsport-Pile Bay Road, then to a new deepwater port site at Iniskin Bay. The potential for geohazard impacts along the transportation corridor, ports, and pipeline would increase under the Pebble Project expansion scenario, because both the north and south access corridors and two ports would be used under all alternatives. This would add the effects of unstable slopes along the north access road to those of Alternative 1a. In addition, the development of the second port at Iniskin Bay (under all alternatives) would increase the likelihood of impacts from dock instability, volcanic ashfall, and tsunamis. In the case of tsunamis, the likelihood of a large tsunami of tectonic origin with a 2,500-year return period occurring would increase due to the longer life of the project, with the probability of occurrence at either port roughly 1 in 17, assuming the ports would be functioning for approximately 148 years total (98 years of operations, plus 50 years of closure activities). The likelihood of a landslide-induced tsunami from an Augustine volcanic debris slide could be higher, depending on the results of site-specific tsunami analysis, which would be conducted in final design (see Chapter 5, Mitigation). **Magnitude:** The Pebble Project expansion scenario would impact a footprint approximately four times larger than Alternative 1a, much of which would include new facilities with potential stability impacts such as TSF embankments, WRFs, and pit walls. **Duration/Extent:** The duration and extent of cumulative impacts to geohazards would vary from temporary (e.g., slope instability during construction) to long-term (e.g., instability from additional earthworks and mine facilities during operations) to permanent (e.g., regional risk to expanded bulk TSFs from earthquakes or volcanoes). The extent of			

Table 4.15-4: Contribution to Cumulative Effects from Geohazards

Reasonably Foreseeable Future Actions	Alternative 1a	Alternative 1 and Variants	Alternative 2 and Variants	Alternative 3 and Variant
	cumulative effects would be in the immediate vicinity of Pebble Project expansion scenario and along the additional transportation segments and port site. **Contribution:** The removal and storage of overburden, rock, and tails, and the extension of the road and pipeline system into steep terrain contributes to the cumulative effects of geohazards such as slope instability. However, these areas are relatively undeveloped, and effects would be limited to the close vicinity of the project footprint, which is a relatively small area in the affected watersheds.			
Other Mineral Exploration Projects	**Magnitude:** Mining exploration activities, including additional borehole drilling, road and pad construction, and development of temporary camp facilities, would contribute a small amount of slope instability at discrete locations, depending on landowner permitting and restoration requirements. Mineral exploration at the Pebble South and Groundhog prospects could have a minor cumulative effect on geologic hazards, depending on the extent of infrastructure development that was to occur. Under any pre-development exploration scenario, effects on geologic hazards would be expected to be temporary and minor, and limited to potential cumulative effects on infrastructure shared with the Pebble Project. **Duration/Extent:** Exploration activities typically occur at a discrete location for one season, although a multi-year program could expand the geographic area affected in a specific mineral prospect. Table 4.1-1 in Section 4.1, Introduction to Environmental Consequences, identifies seven mineral prospects in the EIS analysis area where exploratory drilling is anticipated (four of which are in relatively close proximity to the Pebble Project).	Impacts would be similar to those for Alternative 1a.	Impacts would be similar to those for Alternative 1a.	Impacts would be similar to those for Alternative 1a.

Table 4.15-4: Contribution to Cumulative Effects from Geohazards

Reasonably Foreseeable Future Actions	Alternative 1a	Alternative 1 and Variants	Alternative 2 and Variants	Alternative 3 and Variant
	Contribution: Exploration activities could contribute to cumulative effects of slope instability, although the areal extent of disturbance would be a relatively small portion of the Kvichak/Nushagak watersheds. Assuming compliance with permit requirements, contributions to slope instability would be minimal.			
Oil and Gas Exploration and Development	**Magnitude:** Oil and gas exploration activities in LPB and lower Cook Inlet federal lease areas could involve geophysical exploration; and in limited cases, exploratory drilling (see Table 4.1-1 and Figure 4.1-1 in Section 4.1, Introduction to Environmental Consequences). Onshore geophysical exploration would involve temporary overland activities, with permit conditions that avoid or minimize soil disturbance. Should it occur, onshore exploratory drilling would involve the construction of temporary pads and support facilities, with permit conditions to minimize disturbance to geohazards and restore drill sites after exploration activities have ceased. Offshore exploration activities that occur in the area of the pipeline could increase natural or man-made hazards to the Pebble pipeline or existing fiber-optic cables (Intecsea 2019; NanaWP and Intecsea 2019a), such as scour/erosion or anchor damage with increased boat traffic. **Duration/Extent:** Geophysical exploration and exploratory drilling are typically single-season temporary activities. The 2013 Bristol Bay Plan Amendment shows 13 oil and gas wells drilled on the western Alaska Peninsula, and a cluster of three wells near Iniskin Bay. Historic and active offshore leases in lower Cook Inlet overlap the Pebble natural gas pipeline route in the center and eastern side of the inlet. It is possible that additional geophysical testing and exploratory drilling could occur in the EIS	Impacts would be similar to those for Alternative 1a.	Impacts would be similar to those for Alternative 1a.	Impacts would be similar to those for Alternative 1a.

Table 4.15-4: Contribution to Cumulative Effects from Geohazards

Reasonably Foreseeable Future Actions	Alternative 1a	Alternative 1 and Variants	Alternative 2 and Variants	Alternative 3 and Variant
	analysis area; however, based on historic activity, this is not expected to be intensive. **Contribution:** Onshore oil and gas exploration activities would be required to minimize surface disturbance, and could occur in the analysis area, but distant from the Pebble Project. Offshore activities would be required by the Bureau of Safety and Environmental Enforcement to have mitigation plans in place for avoidance of damage to existing infrastructure (NanaWP and Intecsea 2019a). The project would have minimal contribution to cumulative effects from these activities.			
Road Improvement and Community Development Projects	**Magnitude:** Road improvement projects and continued use of Diamond Point Rock Quarry could have limited impacts on geologic hazards, and contribute to cumulative effects in the overall analysis area, but there would be no cumulative effects on infrastructure shared with the project. LPB and State of Alaska transportation, infrastructure, and energy projects that include possible upgrades to the Williamsport-Pile Bay Road could cause potential reduction in geohazards in the analysis area, but would not have combined effects with the transportation corridor under Alternative 1a. Likewise, the Diamond Point Rock Quarry could have an effect on geologic hazards such as slope instability and rockfall, although these would be expected to be limited to the immediate area around the quarry site, and not have any combined effects with Pebble infrastructure. **Duration/Extent:** Disturbance from road construction would typically occur over a single construction season. Contributions for quarrying activities at Diamond Point would be long-term, for the life of quarry operations. Geographic extent would be	Impacts would be similar to those for Alternative 1a, but less than those for Alternative 2 and Alternative 3, due to lack of effects on infrastructure shared with the Pebble Project.	**Magnitude:** LPB and State of Alaska transportation, infrastructure, and energy projects include possible upgrades to the Williamsport-Pile Bay Road, which is the same alignment that would be used under Alternative 2. If selected, the net magnitude and geographic extent of unstable slope effects may be relatively low, because the mine access road would already be rerouted or upgraded for maintaining slopes. If the road were to be further widened as part of a transportation improvement project, there would likely be additional impacts. The footprint of the Diamond Point rock quarry overlaps	Impacts would be the same as those for Alternative 2.

Table 4.15-4: Contribution to Cumulative Effects from Geohazards

Reasonably Foreseeable Future Actions	Alternative 1a	Alternative 1 and Variants	Alternative 2 and Variants	Alternative 3 and Variant
	limited to the vicinity of the Williamsport-Pile Bay Road, communities, and Diamond Point. **Contribution:** Road construction and quarry use could have effects on slope stability in the analysis area, but would be removed from the project, which would have minimal contribution to cumulative effects.		with the Diamond Point port footprint in Alternative 2; therefore, there could be a relatively minor net increase in geohazard impacts, such as unstable slopes on shore-based infrastructure or dock stability effects on the marine environment. **Duration/Extent:** These effects are expected to be temporary and repairable, and minor in extent, limited to the immediate areas around the quarry site and roads. The estimated area that would be affected at Diamond Point is approximately 140 acres (ADNR 2014a). **Contribution:** Road construction and quarry use could have cumulative effects on slope stability in areas of project infrastructure overlap. The Pebble Project under Alternative 2 is expected to have minimal contribution to cumulative effects.	

Table 4.15-4: Contribution to Cumulative Effects from Geohazards

Reasonably Foreseeable Future Actions	Alternative 1a	Alternative 1 and Variants	Alternative 2 and Variants	Alternative 3 and Variant
Summary of Project contribution to Cumulative Effects	Primary factors contributing to cumulative geohazards effects include: • Increased potential for stability impacts under Pebble Project expansion scenario from new embankments, storage areas, and pit walls, and extension of roads and pipelines into unstable terrain. • Minor effects from the Pebble Project combined with mineral and oil/gas exploration projects, road improvements, and continued quarry development. Overall, the contribution of Alternative 1a to cumulative geohazards effects, when taking other past, present, and RFFAs into account, would be minor in terms of magnitude, duration, and extent, given industry design standards and permit requirements to mitigate hazards to man-made facilities at the Pebble Project expansion scenario, protection of slope stability along roads, and mitigation plans for avoidance of offshore hazards.	Impacts would be similar to those for Alternative 1a.	Impacts would be similar to those for Alternative 1a and Alternative 1, although less area/watersheds would be affected by mine expansion, and more critical facilities would be concentrated in areas of unstable terrain. Minor effects from Alternative 2 combined with road improvement projects and Diamond Point Rock Quarry.	Impacts would be similar to those for Alternative 2.

Notes:
TSF = Tailings Storage Facility
WMP = Water Management Pond
WRF = waste rock facilities
NAG = non-acid generation
SFK = South Fork Koktuli
UTC = Upper Talarik Creek
PAG = potentially acid-generating
ADNR = Alaska Department of Natural Resources
LPB = Lake and Peninsula Borough
RFFAs = Reasonably Foreseeable Future Actions