Probabilistic Seismic Hazard Assessment

Seabee Gold Mine, Saskatchewan

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# Preface

*This report documents the probabilistic seismic hazard analysis (PSHA), site-specific ground response analysis, slope displacement modeling, and seismic design criteria developed for the tailings storage facilities (TSFs) at Seabee Gold Mine, as commissioned by SSR Mining – Seabee Gold Operation Inc.. The assessment adheres to international standards (GISTM, CDA, ANCOLD) and reflects consequence-based performance objectives across the full lifecycle of the facility.*

## *Seismic Hazard*

Probabilistic seismic hazard analysis (PSHA) was completed using a comprehensive logic tree that integrates all relevant sources, ground‑motion models, and explicit uncertainty treatment (see Methodology). Hazard at the reference‑rock condition ( m/s) is quantified by numerical integration over all earthquake scenarios (magnitude, distance, source parameters) per the total probability theorem. Source recurrence, geometry, and maximum magnitude are fully specified, with alternatives and distributions encoded at the branch level. GMPEs are mapped to tectonic regimes/source groups, with branch weights and parameter alternatives implemented as specified. Aleatory variability follows each GMPE’s sigma with exceedance computed in the lognormal framework. For every realization (unique source–GMPE branch set), hazard curves are constructed and weighted per the logic tree. Disaggregation identifies controlling magnitude, distance, and residual parameters at each hazard level.

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| Table 1: Design Ground Motions in terms of horizontal pseudo-spectral acceleration PSA [cm/s2] for return periods ranging from 475 to 9975 [yr] and structural periods Tn ranging from 0.05 to 5 [s]. Spectral ordinates were obtained assuming rock site conditions with Vs30 = 760 [m/s]. (p=mean)   | **TR[yr]** | **poe[%]** | **0.05** | **0.1** | **0.2** | **0.5** | **1** | **2** | **5** | **ID** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | 475 | 10.0 | 11 | 16 | 16 | 9 | 5 | 2 | 0 | max | | 975 | 5.0 | 20 | 29 | 26 | 15 | 8 | 3 | 1 | max | | 1,975 | 2.5 | 37 | 49 | 42 | 23 | 12 | 5 | 1 | max | | 2,475 | 2.0 | 45 | 58 | 48 | 26 | 13 | 6 | 1 | max | | 4,975 | 1.0 | 78 | 94 | 75 | 37 | 19 | 8 | 2 | max | | 9,975 | 0.5 | 129 | 150 | 114 | 54 | 26 | 11 | 3 | max | |

## *Site Response*

Dynamic site response analyses were performed for six shear‑wave velocity classes aligned with NEHRP categories (ASCE 7; NBC 2020). Site effects were incorporated via two complementary PSHA‑consistent methods: (1) an ergodic ground‑motion model (**gem**) based on Vs30‑dependent GMPEs, and (2) a site‑specific amplification model applied to reference‑rock hazard ( m/s) to capture nonlinear effects (**sdLnSaFC1**). For each site class and method, spectral accelerations and PGA were computed with full propagation of aleatory and epistemic uncertainties using Monte Carlo simulation. Site‑specific design spectra and PGA were developed for service levels with annual exceedance probabilities from 1/475 to 1/9,975, consistent with risk evaluation and consequence classification.

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| Table 2: Design Ground Motions in terms of peak ground accelerations PGA [%s] for return periods ranging from 475 to 9,975 [yr] assuming site conditions with Vs30 ranging from 200 to 1000 [m/s]. Mean values.   | **TR[yr]** | **poe[%]** | **180** | **270** | **360** | **560** | **760** | **1250** | **1500** | **ID** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | 475 | 10.0 | 14 | 11 | 10 | 9 | 7 | 8 | 8 | max | | 975 | 5.0 | 25 | 20 | 18 | 15 | 13 | 13 | 14 | max | | 1,975 | 2.5 | 42 | 33 | 29 | 25 | 21 | 22 | 23 | max | | 2,475 | 2.0 | 49 | 38 | 34 | 29 | 25 | 26 | 27 | max | | 4,975 | 1.0 | 79 | 62 | 54 | 46 | 41 | 43 | 43 | max | | 9,975 | 0.5 | 122 | 97 | 85 | 72 | 64 | 67 | 69 | max | |

## *Seismic Design Criteria*

*Seismic design criteria for tailings storage facilities were established according to the performance-based frameworks defined in GISTM (*[*2020*](#ref-gistm2020)*), CDA (*[*2021*](#ref-cda2021)*), and ANCOLD (*[*2019*](#ref-ancold2019)*). These standards specify consequence-based design earthquake levels and performance objectives for each facility and lifecycle phase, determined by population at risk, potential loss of life, and the magnitude of environmental and socio-economic impact. For each consequence class and operational phase (construction, operation, closure, and post-closure), ground-motion criteria were identified in accordance with the applicable standard. Resulting design spectra are provided for each consequence class and service level (OBE, SEE, and PCE).*

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| Table 3: Design Ground Motions for operation, closure and post-closure, defined in terms of peak ground accelerations (PGA) [cm/s2] for diffrent consequence levels (**Class**) according to the **GISTM** standard. Spectral ordinates were obtained for AEP ranging from 1/475 to 1/9975s [1/yr] and site conditions with Vs30 ranging from 180 to 1500 [m/s].   | **Standard** | **Class** | **Stage** | **180** | **270** | **360** | **560** | **760** | **1250** | **1500** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | GISTM | Low | Closure | 14 | 11 | 10 | 9 | 7 | 8 | 8 | | GISTM | Low | Operation | 14 | 11 | 10 | 9 | 7 | 8 | 8 | | GISTM | Low | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Significant | Closure | 25 | 20 | 18 | 15 | 13 | 13 | 14 | | GISTM | Significant | Operation | 25 | 20 | 18 | 15 | 13 | 13 | 14 | | GISTM | Significant | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | High | Closure | 49 | 38 | 34 | 29 | 25 | 26 | 27 | | GISTM | High | Operation | 49 | 38 | 34 | 29 | 25 | 26 | 27 | | GISTM | High | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Very High | Closure | 79 | 62 | 54 | 46 | 41 | 43 | 43 | | GISTM | Very High | Operation | 79 | 62 | 54 | 46 | 41 | 43 | 43 | | GISTM | Very High | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Extreme | Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Extreme | Operation | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Extreme | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | |

## *Slope Performance*

Slope performance is evaluated probabilistically by integrating site‑specific ground‑motion hazard, slope dynamic characterization, and a suite of Newmark‑type empirical and semi‑empirical displacement models. Shear stiffness, shear‑wave velocities, and fundamental periods are obtained from numerical models spanning representative geometries and material profiles for tailings and waste‑rock facilities. Permanent displacements are computed per scenario using a weighted ensemble of rigid‑ and flexible‑block Newmark models. Aleatory and epistemic uncertainties from ground motion, site amplification, and Newmark models are propagated via Monte Carlo simulation. Performance‑based seismic coefficients are established for different objectives by identifying the minimum yield acceleration that limits the probability of exceeding specified displacement thresholds.

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| Table 4: Performance-based seismic coefficients [cm/s2] for different geometry scenarios (S8 to S75) and service levels (AEP 1/475 to 1/9975). Sesmic coefficients were calibrated for residual displacements 10 [mm] and averaged through different material scenarios (U1 to U8)   | **TR** | **Da** | **p** | **S8** | **S15** | **S22** | **S30** | **S38** | **S45** | **S52** | **S60** | **S68** | **S75** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | 475 | 10 | mean | 3 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | | 975 | 10 | mean | 6 | 6 | 6 | 7 | 7 | 7 | 7 | 8 | 8 | 8 | | 1,975 | 10 | mean | 10 | 11 | 11 | 12 | 12 | 13 | 13 | 13 | 14 | 14 | | 2,475 | 10 | mean | 12 | 13 | 14 | 14 | 15 | 15 | 15 | 16 | 16 | 17 | | 4,975 | 10 | mean | 21 | 22 | 23 | 24 | 24 | 25 | 25 | 26 | 27 | 27 | | 9,975 | 10 | mean | 34 | 36 | 37 | 37 | 38 | 39 | 40 | 42 | 42 | 43 | |

# Introduction

## Background

SSR Mining – Seabee Gold Operation Inc. (SSR/SGO) retained SRK Consulting (Canada)(SRK) as Engineer of Record to complete a probabilistic seismic hazard assessment (PSHA) for the Seabee Gold Mine, located near Laonil Lake, Saskatchewan, Canada. The project comprises underground mining operations and associated tailings management facilities, with the primary commodity being Gold. This assessment provides the technical basis for seismic design for all facilities at , consistent with international standards GISTM([2020](#ref-gistm2020)), ANCOLD([2019](#ref-ancold2019)) and CDA([2021](#ref-cda2021)). All seismic hazard metrics, ground-motion parameters, and design criteria presented in this report have been developed in direct response to project needs, independent review panel (ITRB) recommendations, and industry practice.

## Objectives

The primary objectives of this study are as follows:

**Seismic Hazard:** Import and deploy all source geometries, segmentation, recurrence relationships, and magnitude–frequency parameters directly from the GEM logic-tree XML without modification. Adjust ground-motion modeling parameters to ensure compatibility with the adopted source model. Quantify annual exceedance probabilities for ground-motion intensity measures at the site using probabilistic seismic hazard analysis. Compute site-specific hazard curves and uniform hazard spectra. Incorporate and propagate epistemic and aleatory uncertainties through logic-tree branching and probabilistic integration. Summarize hazard results as mean and fractile annual exceedance probability curves for defined site conditions.

**Site Response:** Compute site-specific peak ground acceleration and spectral ordinates using both ergodic ground-motion models and site-specific amplification factors. Present hazard and site response results as tables and figures summarizing peak ground acceleration values and uniform hazard spectra for all site classes and quantile levels.

**Slope Design:** Quantify permanent slope displacements using an ensemble of empirical and semi-empirical Newmark models for site-specific ground-motion scenarios. Determine performance-based seismic coefficients () for slope stability design through displacement-based inversion of Newmark model results. Calculate fundamental periods and dynamic properties of embankments based on site geometry and material parameters. Summarize slope design parameters in terms of fundamental periods, model parameters, and predicted Newmark displacements for all critical slopes at the site.

**Design Criteria:** Establish seismic design criteria for tailings storage facilities based on international standards and consequence classification. Assign design ground motions and performance objectives for all project lifecycle phases, including operation, closure, and post-closure. Determine Maximum Credible Earthquake (MCE) scenarios in accordance with regulatory requirements. Define probabilistic design spectra and associated ground motions for each operational stage and consequence rating. Present site-specific design ground motions for all relevant stages and site conditions as required by GISTM, CDA, and ANCOLD standards. Select a suite of ground motion records for advanced dynamic analyses (e.g., FLAC, PLAXIS), ensuring the suite encompasses the amplitude and frequency content consistent with site-specific hazard and design ground motions.

**Uncertainty Modeling:** Incorporate and propagate epistemic and aleatory uncertainties through logic-tree branching and probabilistic integration. Propagate all uncertainties as encoded in the GEM logic-tree structure. Propagate hazard-based and model-based uncertainties in site amplification through Monte Carlo sampling to generate fractile site response spectra. Aggregate epistemic and aleatory uncertainties in ground-motion intensity measures, site amplification, and displacement models to generate robust confidence intervals for Newmark displacements and pseudo-static seismic coefficients.

# Seismotectonic Setting

## Seismotectonic Setting – Western Canada

### Tectonics

Western Canada is situated at the convergent boundary between the North American, Pacific, and Juan de Fuca plates. The Cascadia subduction zone extends offshore from northern California through the coasts of Washington and British Columbia, terminating near the northern tip of Vancouver Island. Along this margin, the oceanic Juan de Fuca Plate is subducting beneath the North American Plate at a convergence rate of approximately 40 mm/year ([Wang et al. 2003](#ref-Wang2003)). The geometry of the Cascadia megathrust is characterized by a gently dipping interface that accommodates both interplate locking and episodic slip behavior. Geodetic measurements indicate significant plate coupling along the margin, with potential for full-length megathrust rupture.

North of Cascadia, the Queen Charlotte Fault system runs offshore and parallel to the British Columbia coastline. This structure forms the major right-lateral transform boundary between the Pacific and North American plates and has generated several of Canada’s largest instrumentally recorded earthquakes, including the 1949 8.1 and the 2012 7.8 events ([Rogers 1980](#ref-Rogers1980); [Cassidy et al. 2016](#ref-Cassidy2016)). The transform fault system is segmented and displays both strike-slip and oblique-slip kinematics, with evidence for complex rupture patterns and stress partitioning along the margin.

Inland from the subduction and transform boundaries, the Canadian Cordillera encompasses a wide zone of crustal deformation, including the Intermontane Belt, the Omineca Belt, and the Rocky Mountain Trench. This region is characterized by distributed active faults, transpressional and transtensional tectonics, and inherited structural features dating to the Mesozoic and early Cenozoic orogenies. Geologic mapping and paleoseismic studies have identified Quaternary surface faulting along several crustal structures in southern British Columbia, including the Leech River, Devils Mountain, Fraser River, and others, each representing potential sources for moderate to large crustal earthquakes ([Journeay et al. 2021](#ref-Journeay2021)). The Peace River Arch and Northern Rocky Mountains also show evidence of intraplate deformation, with low slip rates but episodic seismicity.

Offshore, the Explorer Plate, a remnant fragment of the Farallon Plate, interacts with the northern end of the Juan de Fuca system, further complicating local plate tectonic relationships. The geometry and segmentation of these offshore plates play a critical role in the distribution of seismic hazard along the entire western Canadian margin.

### Seismicity

Seismicity in Western Canada is dominated by activity associated with the Cascadia subduction zone, the Queen Charlotte Fault system, and distributed crustal faults of the Cordillera. The subduction interface has produced great earthquakes, including the full-margin rupture of the Cascadia megathrust in January 1700, identified through Japanese tsunami records and paleoseismic deposits in coastal marshes and offshore turbidites ([Satake et al. 1996](#ref-Satake1996); [Goldfinger, Nelson, and Johnson 2012](#ref-Goldfinger2012)). This event is estimated at 8.7–9.2, with recurrence intervals on the order of 500 to 600 years. No similar megathrust event has been instrumentally recorded in the region, but slow slip events and episodic tremor indicate ongoing strain accumulation and release on the subduction interface.

The Queen Charlotte Fault has produced several large earthquakes during the instrumental period, including the 1949 8.1 event—the largest Canadian earthquake on record—and the 2012 7.8 event near Haida Gwaii ([Rogers 1980](#ref-Rogers1980); [Cassidy et al. 2016](#ref-Cassidy2016)). Both events ruptured significant lengths of the fault and generated strong ground motions along the coast and on adjacent islands. Other notable offshore earthquakes include the 2013 7.5 Craig, Alaska earthquake and numerous 6–7 events along the fault system, which have caused shaking and damage in coastal communities.

In the onshore Cordillera, crustal seismicity is concentrated in southern British Columbia, Vancouver Island, and the Puget Lowland. The 1946 7.3 Vancouver Island earthquake remains the largest onshore event recorded in Canada, with a felt area extending into the northwestern United States and documented liquefaction and surface deformation in the epicentral region ([Cassidy et al. 2016](#ref-Cassidy2016)). Moderate-magnitude earthquakes ( 5–6.5) occur regularly along mapped and inferred faults, with focal depths typically less than 30 km. The 2001 6.8 Nisqually earthquake, although centered in Washington, generated strong shaking across southern British Columbia and demonstrated the regional significance of subcrustal intraslab earthquakes within the Juan de Fuca Plate.

The Intermountain Belt, Peace River Arch, and foothills of the Rocky Mountains are sites of distributed intraplate seismicity, typically with lower event rates and magnitudes compared to the plate margin. However, historical earthquakes in the 5–6 range have caused ground shaking and minor damage in these regions ([Halchuk et al. 2023](#ref-Halchuk2023)).

Earthquake monitoring is conducted by Natural Resources Canada (NRCan) through a network of broadband and short-period stations, providing dense coverage in the populated regions of southern British Columbia and sparser coverage in the north and interior ([Natural Resources Canada 2024b](#ref-NRCANNet)). The NRCan and USGS earthquake catalogs together provide a comprehensive instrumental record, with magnitude reporting standardized to for hazard assessment purposes ([Natural Resources Canada 2024a](#ref-NRCANCat); [United States Geological Survey 2024](#ref-USGS)). The instrumental catalog is generally complete for events since the early 20th century, with macroseismic and historical records supplementing the documentation of pre-instrumental earthquakes.

## Source Model

Seismic‑source characterization adheres to the classical PSHA workflow first outlined by Cornell ([Cornell 1968](#ref-Cornell1968)) and implemented under SSHAC guidelines ([Senior Seismic Hazard Analysis Committee 1997](#ref-SSHAC1997)). Fault and background (areal) sources are delineated from geologic, geophysical, and seismological evidence, with segmentation retained where data justify discrete structural blocks. Coordinates, strike, dip, length , width , and seismogenic depth enter the XML logic tree unchanged, ensuring OpenQuake constructs finite rupture planes without geometric reinterpretation.

All sources employ the truncated Gutenberg–Richter relationship with project‑specific and bounds stored as logic‑tree attributes. The and coefficients derive from declustered catalogs that satisfy completeness criteria consistent with geotechnical‐earthquake‑engineering practice ([Kramer 1997](#ref-Kramer1997)). Maximum magnitudes for fault sources rely on the empirical relationships of Wells & Coppersmith ([Wells and Coppersmith 1994](#ref-Wells1994)); areal‑source values reflect historical seismicity and regional analogs. Characteristic and time‑dependent (renewal) recurrence formulations appear as alternative branches where paleoseismic evidence exists, following Schwartz & Coppersmith ([Schwartz and Coppersmith 1984](#ref-Schwartz1984)) and Youngs & Coppersmith ([Youngs and Coppersmith 1985](#ref-Youngs1985)). Activity rates are computed by moment balance, , using published slip‑rate estimates.

Rupture dimensions scale with magnitude according to the same Wells & Coppersmith regressions. Multi‑segment ruptures are allowed when mapping or paleoseismic studies support connectivity. OpenQuake calculates rupture‑to‑site distances—, , and others—directly from the imported planes, maintaining consistency with the selected GMPE set. Unless supplanted by a renewal branch, temporal occurrence is modeled as a stationary Poisson process, the canonical PSHA assumption.

### Source Model adopted for Seabee Gold Mine (SHM6)

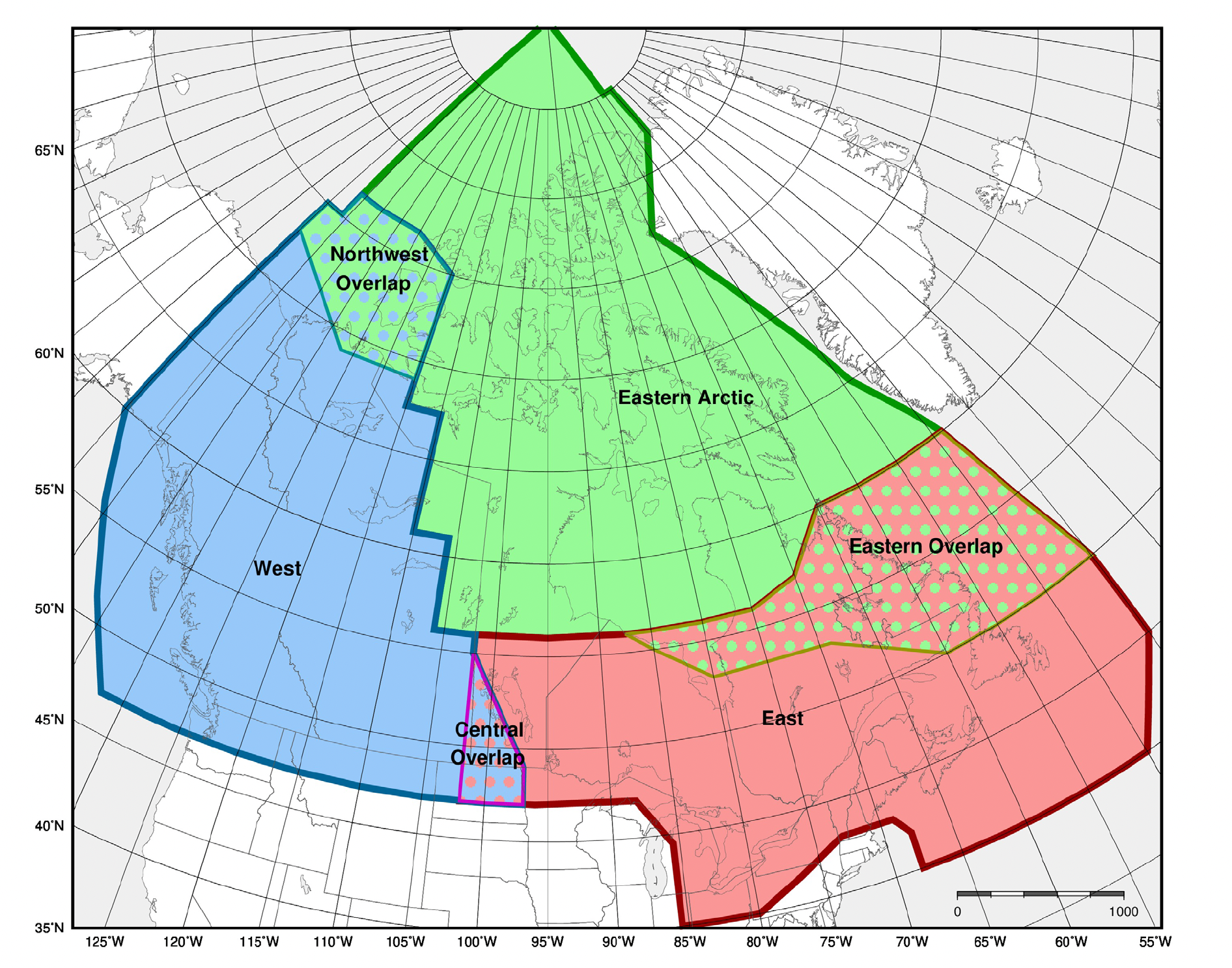
The 6th Generation Seismic Hazard Model of Canada (CanadaSHM6) is the current national standard for probabilistic seismic hazard analysis in Canada ([Adams et al. 2020](#ref-Adams2020); [Global Earthquake Model Foundation (GEM) 2018](#ref-GEM2018); [National Research Council of Canada 2020b](#ref-NRC2020)). This model forms the basis of the seismic hazard values incorporated in the 2020 National Building Code of Canada ([2020b](#ref-NRC2020)). CanadaSHM6 is structured as three regional sub-models, each developed to capture the distinct tectonic and seismogenic environments across the country: the Western Model, the East Arctic Model, and the Southeastern Model. The project site is located within the Western Model domain.

Seismicity rates in CanadaSHM6 are represented using a truncated Gutenberg–Richter recurrence relationship, consistent with international PSHA best practice ([Gutenberg and Richter 1944](#ref-Gutenberg1944); [Cornell 1968](#ref-Cornell1968)). Earthquake frequency is defined by incremental magnitude bins, parameterized by the Gutenberg–Richter and values. The use of truncated Gutenberg–Richter models, with upper () and lower () bounds, ensures physical plausibility and alignment with regional seismotectonic knowledge ([Youngs and Coppersmith 1985](#ref-Youngs1985)).

CanadaSHM6 employs a source-specific approach, discretizing seismicity rates spatially and by source type (fault, subduction interface, intraslab, crustal, and background zones). Source parameters, including geometry, slip rates, and recurrence parameters, are detailed for each fault or zone. Table B1 in Appendix A lists the active source faults and their key attributes within the Western Model, in accordance with the methodology published by Natural Resources Canada ([Adams et al. 2020](#ref-Adams2020)).

The regionalization adopted in CanadaSHM6, and the segmentation of the Western, East Arctic, and Southeastern sub-models, reflects current understanding of Canadian tectonics and is consistent with practices in other modern national hazard models ([2018](#ref-GEM2018), [2021](#ref-GEM2021); [2015](#ref-Field2015)). The Western Model is characterized by active plate boundary faulting, crustal seismicity, and subduction-related sources, with detailed treatment of fault segmentation, maximum magnitudes, and slip rates. These approaches are consistent with the standards adopted in the 2014 US National Seismic Hazard Model and GEM regional models ([Petersen et al. 2014](#ref-Petersen2014); [Global Earthquake Model Foundation (GEM) 2018](#ref-GEM2018)).

The following picture illustrates the spatial extent and tectonic boundaries of the regional models in CanadaSHM6 ([Geological Survey of Canada 2020](#ref-GSC2020)). Methodology, source parameterization, and recurrence models are fully aligned with published Canadian and international peer-reviewed frameworks.



Regional Models of the CanadaSHM6. Source: Natural Resources Canada (2020)

# Seismic Hazard

## Methodology

### Hazard Model

Probabilistic Seismic Hazard Analysis (PSHA) is a methodology that quantifies earthquake ground-motion hazard at a site in terms of probabilities, by integrating over all possible earthqu akes that could affect the site ([Cornell 1968](#ref-Cornell1968); [McGuire 2004](#ref-McGuire2004)). Unlike deterministic approaches that focus on a single scenario, PSHA considers the full range of magnitudes, locations, and associated ground motions, providing a comprehensive description of the rate at which various levels of shaking may be exceeded. In probabilistic seismic hazard analysis (PSHA), the annual frequency (rate) of exceeding a ground-motion level at a site is obtained by integrating over all possible earthquake scenarios. For a single seismic source with an annual occurrence rate of earthquakes (above a minimum magnitude ) denoted (where ), the rate of exceedance of a ground-motion level (e.g. a peak ground acceleration or spectral acceleration) is given by the hazard integral:

where is earthquake magnitude, is the source-to-site distance, and are the probability density functions describing the distribution of magnitudes and distances for source , and is the conditional probability that the ground-motion parameter exceeds level given an event of magnitude at distance . is the *maximum considered magnitude* for source , which is related to the maximum credible earthquake MCE of that source (see [Section 5.1.2](#sec-MCE))

The functions and describe the normalized distributions of earthquake magnitudes and locations within source . This formulation is a direct application of the total probability theorem in continuous form, integrating over all magnitudes and locations of earthquakes from source that could contribute to exceedance of level . All *aleatory variability* in earthquake occurrence, earthquake size,earthquake location and ground motion is thus accounted for through the integration.([McGuire 2004](#ref-McGuire2004); [Baker, Bradley, and Stafford 2021](#ref-Baker2021)). If multiple seismic sources contribute to the hazard at the site, the total annual exceedance rate is obtained by summing the contributions from all sources. Assuming independent sources (indexed by ), each with its own occurrence rate and distributions as above, the overall exceedance frequency for level is:

where is evaluated for each source via the hazard integral. This linear superposition is valid under the assumption that earthquake occurrences in different sources are independent (typically modeled as independent Poisson processes. The result is a seismic hazard curve which quantifies the rate at which various ground-motion levels are exceeded at the site. Because independent seismic sources are assumed (and each follows a Poisson occurrence model), their contributions to are additive. This means the total hazard is the sum of hazards from each source, and the Poisson assumption ensures that the occurrence of multiple events in a year is rare and that the time between events is memoryless ([Cornell 1968](#ref-Cornell1968)).

### Ground-Motion Prediction Equations(GMPE)

The term in the hazard integral, represents the ground-motion model’s probabilistic prediction of exceedance and can be obtained by ground-Motion prediction equations (GMPEs). GMPEs – also called attenuation relationships – are empirical or semi-empirical models that predict the statistical distribution of ground-motion intensity measures (e.g. PGA, spectral acceleration) given an earthquake’s characteristics. A GMPE typically provides a median predicted ground motion (often in log10 or loge units) as a function of parameters such as magnitude , source-to-site distance , and site condition, along with an estimate of **aleatory variability** (the standard deviation of the logarithmic residuals) ([D. M. Boore and Atkinson 2008](#ref-Boore2008)). A *generic form* of a GMPE for an arbitrary intensity measure might be

where and are indicator variables for fault type (e.g. reverse/normal) and site class (e.g. rock or soil), and is a standard normal (zero-mean) random variable representing the random scatter (with being the log-standard-deviation) ([D. M. Boore and Atkinson 2008](#ref-Boore2008))[[1]](#footnote-1). GMPEs condense large datasets of recorded ground motions to capture how intensity decays with distance and increases with magnitude, under average site conditions. Modern GMPEs are derived regionally to reflect different attenuation characteristics, and often include nonlinear site response effects for strong shaking. Typically, ground-motion prediction equations (GMPEs) assume that the logarithm of is normally distributed for given and , characterized by a median (or mean in log-units) and a standard deviation (sigma). If denotes the median intensity (in linear units) for magnitude at distance (from the chosen GMPE) and is the standard deviation of , then the conditional exceedance probability can be expressed as:

where is the standard normal cumulative distribution function. This formulation is equivalent to integrating the lognormal ground-motion distribution from to infinity. The quantity

represents the number of standard deviations by which exceeds the median prediction for scenario . Substituting this into the hazard integral yields an explicit form of the integrand in terms of the standard normal tail probability. All ground-motion aleatory variability is captured through this probability term, ensuring that the hazard integral considers not just the median shaking from an event but the full distribution of possible ground-motion levels. In practical terms, for each scenario (with given ), we calculate the number of standard deviations that lies *above* the median prediction. High values (say ) correspond to low probabilities (rare ground motion outliers), whereas corresponds to the median ground motion (50% exceedance chance given the event).

### Annual Exceedance Probability (AEP)

PSHA results are often expressed in terms of **annual exceedance probability (AEP)** and **return period**, which are reciprocals in the context of a Poisson process. The annual exceedance frequency obtained from the hazard integral can be interpreted as the mean number of times per year that intensity is exceeded. If we assume that exceedance events follow a Poisson process in time (a standard assumption in PSHA, owing to the Poissonian occurrence of earthquakes), then the probability of at least one exceedance in a single year is[[2]](#footnote-2)

This is the **annual exceedance probability (AEP)**. By extension, the probability of exceedance in years is assuming stationarity and independence year-to-year. This relation allows conversion between a mean annual rate and a probability over years. [[3]](#footnote-3) The **return period** (or **mean return interval**) is defined as the reciprocal of the annual frequency of exceedance, usually expressed in years . For low probabilities, return period is approximately the inverse of AEP as well, since for small . Return period is a statistical average, not a guarantee of time between exceedances. Return period is a statistical average, not a guarantee of time between exceedances.

### Hazard Disaggregation

Probabilistic seismic hazard analysis (PSHA) aggregates the contributions of all plausible earthquake scenarios, obscuring the specific combinations of magnitude (), source-to-site distance (), and ground-motion residual () that control site hazard. **Hazard disaggregation** quantifies the relative contribution of each scenario class—binned by magnitude , distance , and residual —to the annual frequency of exceedance at a specified ground-motion threshold ([Bazzurro and Cornell 1999](#ref-Bazzurro1999)). Disaggregation is performed at a target hazard level, defined by an annual exceedance probability or return period. This produces conditional probability distributions over scenario bins and identifies controlling scenario parameters responsible for site hazard ([Kramer 1997](#ref-Kramer1997)). The ground-motion exceedance probability and total annual exceedance rate are defined in the hazard analysis (see [eq. 3.1](#eq-A1), [eq. 3.2](#eq-A2)). The joint exceedance rate for scenario from source is

where is the standard normal probability density function and is the indicator function. Integration over yields the two-dimensional (–) formulation. Partitioning parameter space into magnitude bins , distance bins , and residual bins , the contribution from bin is

The conditional probability that exceedance of is produced by this bin is given by [eq. 3.5](#eq-D3). The **modal scenario** is the bin for which attains its maximum. Hazard disaggregation applies Bayes’ theorem to the joint distribution of scenario parameters and ground-motion exceedance:

The marginal distribution for quantifies the proportion of exceedance attributable to median or above-median ground-motion residuals. Marginal probabilities are obtained by double summation over the other indices:

Disaggregation provides a data-driven basis required for rigorous scenario selection in engineering analysis. The *modal scenario* and the highest-probability bins identified through disaggregation correspond to the earthquake characteristics most responsible for exceedance at the design ground-motion level.

### Uncertainty Model

Uncertainty in seismic hazard assessment is managed exclusively through the explicit logic-tree structure and scenario sampling specified in the adopted OpenQuake-compatible XML files for both source characterization and ground-motion modeling. All modeling of epistemic and aleatory uncertainty is determined at the parameter and branch level in these files, with no reinterpretation, analyst adjustment, or external weighting applied at any stage of the process. The approach is consistent with SSHAC Level 3/4 protocols ([Senior Seismic Hazard Analysis Committee 1997](#ref-SSHAC1997)), the formal PSHA methodology, and the workflow equations established in this report.

Epistemic uncertainty is represented entirely by the logic-tree branches defined in the input files. In source modeling, this includes alternative parameterizations and models for each tectonic regime, source type, and fault or area source. All parameter alternatives—such as magnitude–frequency distribution (, , , ), recurrence model (Gutenberg–Richter, characteristic, hybrid), segmentation, nodal plane orientation, hypocentral depth, and seismogenic geometry—are explicitly branched with weights as prescribed in the XML. The parameterization, binning, branching, and branch weights are fully specified in the project input files and documented in the appendices. There is no deviation from the logic-tree structure or parameter values as adopted for the project. All tectonic regime assignments, source groupings, minimum and maximum magnitudes, and frequency distribution binning are implemented exactly as documented. The model includes any specified uncertainty in nodal plane or depth distributions, aspect ratios, and scaling relationships, as well as segmentation or connectivity for complex sources, with all values and alternatives encoded in the logic-tree XML.

Ground-motion model epistemic uncertainty is handled by an explicit GMPE logic-tree. Each source group or tectonic region is mapped to a prescribed set of GMPEs, with all branches and weights inherited from the XML. This includes any alternatives in median, sigma, or adjustment factors, and all IMT-specific or regime-specific branching. The mapping between sources, tectonic regimes, and GMPEs is fully determined by the logic-tree schema and is not subject to analyst reinterpretation. Branch weights, model assignments, and parameter alternatives are maintained as in the input files.

Aleatory uncertainty is handled entirely within each epistemic branch. In the source model, aleatory variability is introduced through the stochastic occurrence of earthquakes, scenario sampling from the magnitude–frequency distributions, and random selection of rupture geometry, location, and size, following the equations and formalism defined in the **Hazard Model** section (specifically, Equation [3.1](#eq-A1) for annual exceedance rate, Equation [3.2](#eq-A2) for GMPE exceedance probability, and the scenario disaggregation equations [3.3](#eq-D1), [3.4](#eq-D2), and [3.5](#eq-D3)]). All scenario sampling and integration is performed using the binning and parameter values specified in the project logic-tree XMLs and appendices, with no adjustments or resampling beyond what is encoded in the files. For each ground-motion calculation, the standard deviation () from the GMPE logic-tree branch is applied directly, and residuals are sampled or integrated in accordance with the GMPE formulation.

For each realization, defined by a unique combination of source and ground-motion model branches, annual frequencies of exceedance, hazard curves, and spectral ordinates are computed numerically as detailed in the methodology, using all input parameters, scenario bins, and integration domains as specified in the XMLs. Each realization uses the GMPE and sigma of its assigned branch for all ground-motion calculations and does not mix or average sigma values across branches. The exceedance probability for each scenario is computed using the formal lognormal formulation defined by the GMPE, with the median and sigma fixed per branch.

Hazard curves are calculated and retained for every realization , each weighted by the branch weight specified in the XMLs. Outputs for all sites, periods, and intensity measure types are retained for every branch and realization, allowing for full traceability and audit. The aggregation of epistemic uncertainty is performed by direct numerical procedure. For each exceedance probability (e.g., ) or ground-motion value , the set of realization-specific results is sorted in ascending order of . The cumulative sum of branch weights is then used to identify the -quantile following equation [3.9](#eq-U1),where corresponds to the desired fractile (e.g., 0.16 for 16th percentile, 0.5 for median, 0.84 for 84th percentile). This procedure is implemented numerically for all reported hazard fractiles. For a fixed intensity , the quantile exceedance probability is given by equation [3.10](#eq-U2), where is the exceedance probability at for branch . Mean hazard curves are computed as the arithmetic mean of exceedance probabilities across all branches ([eq. 3.11](#eq-U3)). All aggregation and quantile extraction procedures are performed using only the set of realization-level results and branch weights, as implemented in OpenQuake ([Foundation 2024](#ref-OpenQuakeManual2024))

## Seismic Hazard for Seabee Gold Mine

### Pseudo-Spectral Accelerations (PSA) in Rock

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
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| Table 3.1: Design Ground Motions in terms of horizontal pseudo-spectral acceleration PSA [cm/s2] for return periods ranging from 475 to 9975 [yr] and structural periods Tn ranging from 0.05 to 5 [s]. Spectral ordinates were obtained assuming rock site conditions with Vs30 = 760 [m/s]. (p=mean)   | **TR[yr]** | **poe[%]** | **0.05** | **0.1** | **0.2** | **0.5** | **1** | **2** | **5** | **ID** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | 475 | 10.0 | 11 | 16 | 16 | 9 | 5 | 2 | 0 | max | | 975 | 5.0 | 20 | 29 | 26 | 15 | 8 | 3 | 1 | max | | 1,975 | 2.5 | 37 | 49 | 42 | 23 | 12 | 5 | 1 | max | | 2,475 | 2.0 | 45 | 58 | 48 | 26 | 13 | 6 | 1 | max | | 4,975 | 1.0 | 78 | 94 | 75 | 37 | 19 | 8 | 2 | max | | 9,975 | 0.5 | 129 | 150 | 114 | 54 | 26 | 11 | 3 | max | |

### Hazard Results from NBC 2020

The 2020 National Building Code of Canada (NBCC 2020) provides the authoritative national framework for seismic hazard parameters in Canadian engineering design. Seismic hazard values are derived from the Sixth Generation National Seismic Hazard Model for Canada ([Adams et al. 2020](#ref-Adams2020); [2020](#ref-Halchuk2020)), which applies a probabilistic seismic hazard analysis (PSHA) methodology consistent with international standards ([Geological Survey of Canada 2020](#ref-GSC2020)).

Official NBCC hazard values are distributed via the Natural Resources Canada online hazard tool ([Natural Resources Canada 2021](#ref-NaturalResourcesCanada2021c)). The tool accepts site location (latitude/longitude or address) and site condition (either NBCC site class or time-averaged shear-wave velocity, ). Output parameters include mean ground motion values for the principal annual exceedance probabilities (AEP): 0.002 ( years), 0.001 ( years), and 0.0004 ( years), corresponding to ordinary, important, and post-disaster structures, respectively ([National Research Council of Canada 2020b](#ref-NRC2020); [2020](#ref-Adams2020)). For each AEP, spectral accelerations are provided at peak ground acceleration (PGA), and structural periods ranging from 0.05 to 5 sec., assuming 5 % critical damping.

Site-specific PSHA results are compared directly with NBCC 2020 outputs for the matched location and condition. [[4]](#footnote-4) The NBCC tool enables interpolation at arbitrary locations and supports “mean” fractile output for code-compliant spectral values. [[5]](#footnote-5) The NBCC hazard tool is accessible at: <https://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php>. Agreement between the PSHA and NBCC 2020 values confirms consistency with the national standard model and supports use of the results for code-based engineering design. Small differences observed, specially in soft soils, are attributed to the site response model employed to complement GMPE models with limited support for different shear-wave velocities.

[Reference values from NBC2020](https://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php?code=nbc2020&latitude=55.66679&longitude=-103.41482&siteDesignation=XV&siteDesignationXV=760)

## Summary

Probabilistic seismic hazard analysis (PSHA) was completed using a comprehensive logic tree that integrates all relevant sources, ground‑motion models, and explicit uncertainty treatment (see Methodology). Hazard at the reference‑rock condition ( m/s) is quantified by numerical integration over all earthquake scenarios (magnitude, distance, source parameters) per the total probability theorem. Source recurrence, geometry, and maximum magnitude are fully specified, with alternatives and distributions encoded at the branch level. GMPEs are mapped to tectonic regimes/source groups, with branch weights and parameter alternatives implemented as specified. Aleatory variability follows each GMPE’s sigma with exceedance computed in the lognormal framework. For every realization (unique source–GMPE branch set), hazard curves are constructed and weighted per the logic tree. Disaggregation identifies controlling magnitude, distance, and residual parameters at each hazard level.

# Site Response

## Methodology

### Site-Response Model

Site effects were introduced in the probabilistic seismic hazard analysis through two complementary model. The first used an ergodic ground-motion framework in which a logic tree of -dependent GMPEs was evaluated for shear-wave velocities ranging from 200 m/s to 1100 m/s. For every branch, *openQuake* computed spectral accelerations at the required service levels. Epistemic uncertainty was represented by the branch weights prescribed in the logic tree, and aleatory variability followed the total standard deviation of each GMPE. Because the site term is embedded empirically within the GMPEs, the resulting uniform-hazard spectra remain ergodic.

The second model is an amplification model based on reference-rock hazard curve at m/s and transformed it to the target condition by applying period-dependent, nonlinear amplification factors ,according to expression [4.1](#eq-AF). The amplification model followed the formulation of Stewart and co-authors, which expresses as a log-normal random variable whose mean and dispersion depend on oscillator period, shear-wave velocity, and input PGA ([Stewart et al. 2020](#ref-Stewart2020); [Hashash et al. 2020](#ref-Hashash2020)). For , the model captures nonlinear deamplification at short periods and amplification at intermediate-to-long periods; for stiffer sites, the amplification asymptotically approaches unity.

### Uncertainty Model

Uncertainty in the site-specific spectra arises from two statistically independent sources: hazard uncertainty in and model uncertainty in . Both contributions were propagated by Monte-Carlo simulation conducted separately at each oscillator period. For a given , synthetic values of were generated so that their empirical distribution reproduced the fractiles of the reference-rock hazard curve. Model-to-model epistemic variability was preserved by assigning logic-tree weights to the alternative amplification parameter sets proposed in the literature ([Stewart et al. 2020](#ref-Stewart2020); [Hashash et al. 2020](#ref-Hashash2020)). Weighted quantiles of the synthetic values furnished the final mean and fractile estimates of the site-modified uniform-hazard spectrum, maintaining explicit separation of aleatory and epistemic components throughout the analysis.

## Site Response for Seabee Gold Mine

### Peak-Ground Acceleration

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Table 4.1: Design Ground Motions in terms of peak ground accelerations PGA [cm/s2] for return periods ranging from 475 to 9975 [yr] Spectral ordinates were obtained assuming site conditions with Vs30 ranging from 180 to 1500 [m/s]. (p=mean)   | **TR[yr]** | **poe[%]** | **180** | **270** | **360** | **560** | **760** | **1250** | **1500** | **ID** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | 475 | 10.0 | 14 | 11 | 10 | 9 | 7 | 8 | 8 | max | | 975 | 5.0 | 25 | 20 | 18 | 15 | 13 | 13 | 14 | max | | 1,975 | 2.5 | 42 | 33 | 29 | 25 | 21 | 22 | 23 | max | | 2,475 | 2.0 | 49 | 38 | 34 | 29 | 25 | 26 | 27 | max | | 4,975 | 1.0 | 79 | 62 | 54 | 46 | 41 | 43 | 43 | max | | 9,975 | 0.5 | 122 | 97 | 85 | 72 | 64 | 67 | 69 | max | |

## Summary

Dynamic site response analyses were performed for six shear‑wave velocity classes aligned with NEHRP categories (ASCE 7; NBC 2020). Site effects were incorporated via two complementary PSHA‑consistent methods: (1) an ergodic ground‑motion model (**gem**) based on Vs30‑dependent GMPEs, and (2) a site‑specific amplification model applied to reference‑rock hazard ( m/s) to capture nonlinear effects (**sdLnSaFC1**). For each site class and method, spectral accelerations and PGA were computed with full propagation of aleatory and epistemic uncertainties using Monte Carlo simulation. Site‑specific design spectra and PGA were developed for service levels with annual exceedance probabilities from 1/475 to 1/9,975, consistent with risk evaluation and consequence classification.

Vs30 characterization is described in Appendix [B.1](#sec-vs30_appendix); the site‑response framework, uncertainty treatment, and parameters are detailed in Appendix [C.1](#sec-AF_appendix).

# Design Criteria

## Design Criteria

### Seismic Design Criteria for Tailings Storage Facilities (TSF)

The seismic design of tailings storage facilities (TSFs) is established by international standards, including ([Global Tailings Review (ICMM, UNEP, PRI) 2020](#ref-gistm2020)), ([Canadian Dam Association (CDA) 2021](#ref-cda2021)), and ([Australian National Committee on Large Dams (ANCOLD) 2019](#ref-ancold2019)). These frameworks define performance objectives and design earthquake criteria on the basis of consequence classification and, where applicable, by lifecycle phase. Consequence classes are determined according to population at risk (PAR), anticipated loss of life, and the magnitude of environmental and socio-economic impacts. A detailed breakdown of classification criteria and associated population thresholds is provided in Appendix [D.2](#sec-consequence_appendix), with tabulated requirements in Tables [D.1](#tbl-gistm-seismic), [D.2](#tbl-cda-seismic), and [D.3](#tbl-ancold-seismic).

For each consequence class, design ground motions are assigned in accordance with the specified standard and, where relevant, the operational phase. Under GISTM ([2020](#ref-gistm2020)), a single earthquake design ground motion (EDGM) is specified for each phase; the Maximum Credible Earthquake (MCE) must also be evaluated, and governs if it produces higher demand than the probabilistic event. The CDA standard ([2021](#ref-cda2021)) employs the Safety Evaluation Earthquake (SEE) as the design basis, with the MCE applied as an upper bound for facilities with high consequence ratings; risk-based approaches may be permitted for lower classes during closure or transition phases, provided that post-closure requirements are ultimately satisfied. ANCOLD ([2019](#ref-ancold2019)) applies a dual-level methodology, assigning both an Operating Basis Earthquake (OBE) and Safety Evaluation Earthquake (SEE) by consequence category, with the most severe criteria (MCE or the 1/10,000-year event, 85th percentile) mandated for Extreme-class facilities and required for all dams prior to abandonment ((post-closure).

In all cases, the post-closure or abandonment phase requires compliance with the most stringent seismic criteria. Facilities must demonstrate resistance to either the MCE or the probabilistic 1/10,000-year event whichever governs, as stipulated in the relevant code. Technical requirements and operational considerations for each phase, including construction staging, ongoing surveillance, and the exclusion of temporary features from post-closure design, are provided in Appendix [D.1](#sec-lifecycle_appendix).

The following section presents the design ground motions and response spectra as required by each applicable standard and consequence rating.

### Maximum Credible Earthquake (MCE)

The **Maximum Credible Earthquake (MCE)** is defined as the largest earthquake considered physically plausible on a given seismic source, based on all available geological, geophysical, and seismological evidence ([Commission 2012](#ref-usnrc2012); [International Commission on Large Dams (ICOLD) 2016](#ref-icold2016)). In the design of critical infrastructure—such as tailings storage facilities and large dams—the MCE represents an upper-bound scenario that structures must be able to withstand without catastrophic failure (often corresponding to the Safety Evaluation Earthquake for high-consequence dams) ([2016](#ref-icold2016); [2021](#ref-cda2021); [2020](#ref-gistm2020)).

The MCE approach is entirely **deterministic**, defined without regard to an earthquake’s recurrence rate or exceedance probability. It relies on expert judgment to select scenario parameters such as magnitude, rupture location, and the level of shaking to consider (commonly the median or 84th-percentile ground motion). Historically, the MCE has been a cornerstone of deterministic safety evaluations and remains a mandated check in most jurisdictions for critical facilities, ensuring explicit consideration of the worst credible earthquake event.

MCE estimates can be defined with confidence in well-characterized tectonic settings but become more speculative in regions where fault mapping is incomplete or seismicity is diffuse. In these cases, defining a deterministic scenario carries high epistemic uncertainty due to a lack of universally accepted criteria for selecting ground-motion parameters. For background or area sources (i.e., seismic sources not associated with an individual mapped fault), the analogous MCE scenario is typically taken as the largest possible earthquake () occurring at the closest feasible distance to the site[[6]](#footnote-6). Assigning an MCE for such diffuse sources is inherently more uncertain and involves greater epistemic uncertainty than for well-identified faults ([Commission 2012](#ref-usnrc2012); [Baker, Bradley, and Stafford 2021](#ref-Baker2021)).

When a fault’s characteristics (geometry, activity rate, and ) are fully represented in a probabilistic seismic hazard model, **the MCE is inherently included in the hazard estimate**. In regions dominated by a single major seismic source, the largest events on that source often control the long-period portion of the hazard; in such cases, the deterministic MCE scenario corresponds to the extreme upper tail of the hazard curve ([Baker, Bradley, and Stafford 2021](#ref-Baker2021)).

In regions where seismic hazard is controlled by subduction megathrusts, the explicit determination of an MCE scenario is required. Paleoseismic, historical, and geodetic evidence along margins such as Cascadia, Chile, and northeastern Japan demonstrates quasi-periodic recurrence intervals on the order of 70 – 200 years for events of Mw 8.5 ([Goldfinger, Nelson, and Johnson 2012](#ref-Goldfinger2012); [Mori and Kanamori 2011](#ref-Mori2011); [Simons et al. 2011](#ref-Simons2011)). These intervals are substantially shorter than the return periods that underpin conventional probabilistic design levels (e.g., 475 years for 10 % probability of exceedance in 50 years or 2 475 years for 2 % in 50 years). Consequently, the annual exceedance probability (AEP) associated with the ground motions is markedly higher than that implied by standard design spectra (e.g.: a characteristic recurrence of 100 years corresponds to ≈ 40 % probability of occurrence within a 50-year design horizon). Modern guidelines therefore include a deterministic MCE—alongside low-probability probabilistic levels—to avoid underestimation of long-period demand in subduction settings ([2016](#ref-icold2016); [2021](#ref-cda2021); [2020](#ref-gistm2020)).

### MCER and ELastic Design Spectra (ASCE)

Seismic demand for all Risk Categories I–IV was derived in accordance with **ASCE/SEI 7-22**—as adopted by **IBC 2021**—using the site-specific uniform-hazard spectrum (UHS) established in the preceding chapter. Chapter 11 of ASCE 7 permits replacement of mapped spectral accelerations by values obtained from a probabilistic hazard analysis coupled with site-response modelling, provided the procedures of §11.4.7 are satisfied ([American Society of Civil Engineers 2022, sec. 11.4.7](#ref-ASCE722)). Because the UHS already captures local soil effects, the standard site coefficients were taken as ; thus

where and are the 5 %-damped spectral accelerations at and , respectively. These values represent the horizontal **Maximum Considered Earthquake Response Spectrum** (MCER), consistent with the risk-targeted collapse performance objective of ASCE 7-22 ([American Society of Civil Engineers 2022, sec. 1.2](#ref-ASCE722)). Consistent with §11.4.5, the **design-level spectral accelerations** were obtained by scaling the MCER ordinates by the factor ([American Society of Civil Engineers 2022](#ref-ASCE722) Eq. 11.4-3):

The characteristic periods are

For a damping ratio of 5 %, the horizontal **elastic design spectrum** is defined piece-wise following Equations (11.4-5)–(11.4-7):

where is the long-period transition defined in §11.4.6. The resulting spectrum applies to ordinary, essential, and hazardous facilities; structural design actions are adjusted by the importance factor prescribed in Table 1.5-2 of ASCE 7-22 ([American Society of Civil Engineers 2022, sec. 1.5.2](#ref-ASCE722)). The final MCER and design spectra, anchored to the site-specific hazard and incorporating the requirements of ASCE 7-22 and IBC 2021, provide the basis for subsequent seismic demand assessments.

## Design Criteria for Seabee Gold Mine

### Operational Stage/OBE

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Table 5.1: Design Ground Motions for operation, closure and post-closure, defined in terms of peak ground accelerations (PGA) [cm/s2] for diffrent consequence levels (**Class**) according to different standards. Spectral ordinates were obtained for AEP ranging from 1/475 to 1/9975s [1/yr] and site conditions with Vs30 ranging from 180 to 1500 [m/s].   | **Standard** | **Class** | **Stage** | **180** | **270** | **360** | **560** | **760** | **1250** | **1500** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | CDA | Low | Operation | 25 | 20 | 18 | 15 | 13 | 13 | 14 | | CDA | Significant | Operation | 25 | 20 | 18 | 15 | 13 | 13 | 14 | | CDA | High | Operation | 49 | 38 | 34 | 29 | 25 | 26 | 27 | | CDA | Very High | Operation | 79 | 62 | 54 | 46 | 41 | 43 | 43 | | CDA | Extreme | Operation | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Low | Operation | 14 | 11 | 10 | 9 | 7 | 8 | 8 | | GISTM | Significant | Operation | 25 | 20 | 18 | 15 | 13 | 13 | 14 | | GISTM | High | Operation | 49 | 38 | 34 | 29 | 25 | 26 | 27 | | GISTM | Very High | Operation | 79 | 62 | 54 | 46 | 41 | 43 | 43 | | GISTM | Extreme | Operation | 122 | 97 | 85 | 72 | 64 | 67 | 69 | |

### Closure Stage/SEE

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Table 5.2: Design Ground Motions for operation, closure and post-closure, defined in terms of peak ground accelerations (PGA) [cm/s2] for diffrent consequence levels (**Class**) according to different standards. Spectral ordinates were obtained for AEP ranging from 1/475 to 1/9975s [1/yr] and site conditions with Vs30 ranging from 180 to 1500 [m/s].   | **Standard** | **Class** | **Stage** | **180** | **270** | **360** | **560** | **760** | **1250** | **1500** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | CDA | Low | Closure | 25 | 20 | 18 | 15 | 13 | 13 | 14 | | CDA | Significant | Closure | 49 | 38 | 34 | 29 | 25 | 26 | 27 | | CDA | High | Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | CDA | Very High | Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | CDA | Extreme | Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Low | Closure | 14 | 11 | 10 | 9 | 7 | 8 | 8 | | GISTM | Significant | Closure | 25 | 20 | 18 | 15 | 13 | 13 | 14 | | GISTM | High | Closure | 49 | 38 | 34 | 29 | 25 | 26 | 27 | | GISTM | Very High | Closure | 79 | 62 | 54 | 46 | 41 | 43 | 43 | | GISTM | Extreme | Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | |

### Post-Closure Stage/PCE

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Table 5.3: Design Ground Motions for operation, closure and post-closure, defined in terms of peak ground accelerations (PGA) [cm/s2] for diffrent consequence levels (**Class**) according to different standards. Spectral ordinates were obtained for AEP ranging from 1/475 to 1/9975s [1/yr] and site conditions with Vs30 ranging from 180 to 1500 [m/s].   | **Standard** | **Class** | **Stage** | **180** | **270** | **360** | **560** | **760** | **1250** | **1500** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | CDA | Low | Post-Closure | 25 | 20 | 18 | 15 | 13 | 13 | 14 | | CDA | Significant | Post-Closure | 49 | 38 | 34 | 29 | 25 | 26 | 27 | | CDA | High | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | CDA | Very High | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | CDA | Extreme | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Low | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Significant | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | High | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Very High | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | | GISTM | Extreme | Post-Closure | 122 | 97 | 85 | 72 | 64 | 67 | 69 | |

## Summary

*Seismic design criteria for tailings storage facilities were established according to the performance-based frameworks defined in GISTM (*[*2020*](#ref-gistm2020)*), CDA (*[*2021*](#ref-cda2021)*), and ANCOLD (*[*2019*](#ref-ancold2019)*). These standards specify consequence-based design earthquake levels and performance objectives for each facility and lifecycle phase, determined by population at risk, potential loss of life, and the magnitude of environmental and socio-economic impact. For each consequence class and operational phase (construction, operation, closure, and post-closure), ground-motion criteria were identified in accordance with the applicable standard. Resulting design spectra are provided for each consequence class and service level (OBE, SEE, and PCE).*

*The classification of consequence categories, population thresholds, and the design ground-motion criteria assigned to each class are detailed in Appendix* [*D.2*](#sec-consequence_appendix)*, with tabulated requirements in Tables* [*D.1*](#tbl-gistm-seismic)*,* [*D.2*](#tbl-cda-seismic)*, and* [*D.3*](#tbl-ancold-seismic)*. Technical requirements and design considerations for each lifecycle phase are described in Appendix* [*D.1*](#sec-lifecycle_appendix)*. The resulting design spectra for each consequence class and service level are summarized in Sections* [*5.2.1*](#sec-criteria_results_obe)*,* [*5.2.2*](#sec-criteria_results_sse)*, and* [*5.2.3*](#sec-criteria_results_pce)*.*

# Slope Response

## Methodology

### Fundamental Periods of Slopes

The fundamental period of the embankment can be expressed in terms of the shear-wave velocity at the base , which can be estimated from the shear stiffness at the base and the density of the material using the relation . The expression [6.1](#eq-TS) provides an estimate of different fundamental periods where, is the th-root of the characteristic equation (eigenvalues) that depend on the homogeneity ratio and the truncation ratio .

The truncation ratio can be defined in terms of the height , the berm width and the average slope in the form of [eq. 6.2](#eq-LAMBDA), where is the base width of the embankment. The truncation ratio is a measure of the influence of the berm width on the fundamental period of the embankment.

For a given height , the fundamental period of the embankment increases with the truncation ratio . The homogeneity ratio reflects the law of variation of the shear modulus , where is the (maximum) shear moduli at the base . The following table summarizes the fundamentals periods, truncation ratios, berm widths and material properties obtained for the critical slopes of the Seabee Gold Mine project.

The shear moduli of an arbitrary layer at depth can be estimated in terms of the void ratio and the octahedral stresses from [eq. 6.3](#eq-SM), where is a reference pressure, , , and are material-dependent constants and is the void ratio of the material, and is the octahedral stress at depth ([Ishihara 1997](#ref-Ishihara1997)).

The parameters of the model are obtained from laboratory tests on undisturbed samples of the material. The following table from Ishihara ([Ishihara 1997](#ref-Ishihara1997)) reports the model parameters of the shear stiffness estimation model together with the range of void ratios in which the correlations were calibrated.

### Newmark-based Displacement Model

Seismic-induced permanent displacements of slopes are quantified using the Newmark sliding-block approach, which conceptualizes the sliding mass as a rigid or flexible block with a critical yield acceleration . When the ground acceleration exceeds , the mass experiences incremental downslope movement, and the total displacement accumulates over the course of shaking. The response is characterized probabilistically using empirical or semi-empirical relationships that link to ground-motion parameters and slope properties. All models used in this framework are formulated in the following general form:

where denotes the set of ground-motion intensity measures specific to each model (e.g., , at a reference period, Arias Intensity , peak ground velocity ), is the fundamental period of the sliding mass, and is the earthquake moment magnitude when included by the model. The deterministic term is the model-predicted mean, expressed as a function of these parameters, and is the associated standard deviation in natural-log space. The stochastic term is a standard normal variate, accounting for record-to-record variability not captured by the regression.

Ground-motion intensity measures are obtained from the site hazard results. If the hazard is derived from a general GMPE or site-specific response analysis, spectral accelerations are evaluated at periods required by each model, typically at with in the range 1.3 1.5. Where necessary, logarithmic interpolation is used to obtain the spectral ordinates at non-tabulated periods. The yield acceleration is established by geotechnical analysis and expressed as a fraction of gravity. The selection of is determined by the seismic scenario or logic-tree branch.

Specific functional forms for and, where applicable, for variable , are documented in Appendix [E.1](#sec-newmark_appendix). For example, Ambraseys & Menu (1988) and Yegian et al. (1991) base on the dimensionless ratio , with polynomial terms in . Flexible-block and duration-sensitive formulations (([Bray and Travasarou 2007](#ref-BrayTravasarou2007); [Bray and Macedo 2017](#ref-BrayMacedo2017), [2019](#ref-BrayMacedo2019))) include dependencies on , period, and magnitude. Jibson ([2007](#ref-Jibson2007)) and Saygili & Rathje ([2008](#ref-SaygiliRathje2008)) introduce and to capture duration and velocity effects. In all cases, the models are cast in lognormal space for probabilistic simulation.

All predictive models summarized in Appendix [E.1](#sec-newmark_appendix) were implemented for the displacement calculations. Rigid and flexible-block models, subduction and shallow-crustal event formulations, and both classical and duration-sensitive approaches were applied in accordance with the requirements of the project and the logic-tree weighting scheme. For each scenario, permanent displacements were computed across the entire set of models, with input parameters defined as described above. No predictive models were excluded. The functional forms, input requirements, and statistical parameters for each model are documented in the referenced appendix.

### Uncertainty Model

The uncertainty in input ground-motion parameters is represented by empirical quantile distributions obtained from the probabilistic seismic hazard analysis (PSHA) or site-specific response calculations. For each oscillator period or amplitude measure required by the displacement models, a set of fractiles (e.g., 2%, 5%, 16%, 50%, 84%, 95%, 98%) and the mean are available from the hazard computation. These fractiles reflect both epistemic and aleatory uncertainty from source, path, and site effects. For each realization in the displacement simulation, the required intensity measure is sampled from its empirical quantile function. This is achieved by constructing a monotonic, piecewise function , where is a uniform random variable in . For spectral ordinates not explicitly available in the hazard output, logarithmic interpolation in period is used to obtain the required value. Where the site hazard is derived from reference-rock conditions (), period-dependent amplification factors are applied, with amplification model uncertainty propagated as a lognormal variable in accordance with the methodology outlined in the site response chapter.

Each empirical Newmark displacement model provides a log-standard deviation , determined from regression residuals of observed or simulated events. For all models is treated as a constant for all scenarios. For each realization, a standard normal deviate is drawn, and the predicted displacement is computed as using the input parameters from the sampled scenario. Model-to-model epistemic uncertainty is retained by assigning logic-tree weights before by aggregating results over all models.

The total distribution of predicted displacements is obtained by convolving the empirical variability in input ground-motion parameters with the conditional lognormal scatter from the site response models and Newmark displacement models. The simulation proceeds by drawing, for each scenario and model, a realization of the intensity measures from their empirical quantile functions, computing the deterministic mean displacement from the model’s regression, and applying the model’s standard deviation . This process is repeated for each model and each realization, with model weights applied as appropriate. Quantiles of interest (e.g., median, 84th, and 95th percentile displacements) are computed directly from the assembled ensemble. Appendix [E.1](#sec-newmark_appendix) contains explicit equations, coefficients, and references for all ensemble models, as well as guidance on selecting and weighting models according to local or regional seismotectonic context.

### Performance-Based Seismic Coefficients

The performance-based seismic coefficient is established as the minimum horizontal acceleration required to satisfy a prescribed displacement performance objective under the combined effects of ground motion variability, model dispersion, and site-specific uncertainty. This framework links the allowable permanent displacement threshold directly to the statistical distribution of predicted displacements for the full set of seismic hazard and material scenarios considered in the project. Let denote the random variable representing the Newmark-type displacement, incorporating all aleatory and epistemic uncertainties as characterized in the preceding sections. The allowable displacement is specified as a design criterion for permanent ground deformations under seismic loading. For a given yield acceleration , the exceedance probability for the displacement threshold is given by

where is the distribution of predicted displacements conditional on the selected value of . The distribution reflects both the variability in ground motion parameters (such as , , or other relevant intensity measures required by the selected predictive model) and the model-specific uncertainty in displacement prediction. The performance-based seismic coefficient for a target exceedance probability is defined by the implicit relationship , or, equivalently, as the infimum of for which the probability of exceeding the allowable displacement is controlled at the specified level, under the combined statistical effects of hazard, amplification, and displacement model variability.

For any given , the coefficient represents the lowest value of for which the probability of exceeding the allowable displacement is controlled at the specified level, under the combined statistical effects of hazard, amplification, and displacement model variability. Displacement values below a numerical threshold are floored at m to maintain well-defined logarithmic operations within the displacement models. Expression [6.4](#eq-kmax) links the seismic coefficient to the site-specific hazard, the displacement-based design objective, and the full statistical characterization of both ground motion and material uncertainty.

## Slope Design Parameters for Seabee Gold Mine

### Fundamental Periods

| **IDm** | **USCS** | **TSF Zone** |
| --- | --- | --- |
| U1 | ML MH | Fine Tailings (Slimes, Pond) |
| U2 | SP SM | Coarse Tailings Beach / Upstream Shell |
| U3 | SP SM ML MH | Transitional Zone (Beach Margin / Intermediate Tailings) |
| U4 | GP GW | Rockfill Buttress / Downstream Shell |
| U5 | GP GW SP SM | Downstream Shell with Granular Filter or Transition Layer |
| U6 | GP GW ML MH | Downstream Shell with Fines-Enriched Transition / Filter |
| U7 | CL CH | Core Zone (Impervious Core / Cutoff Trench) |
| U8 | CL CH ML MH | Core Zone with Silty-Clay Transition (Core Margin / Fine Fill) |

### Seismic Coefficients

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Table 6.1: Performance-based seismic coefficients [cm/s2] for different geometry scenarios (S8 to S75) and service levels (AEP 1/475 to 1/9975). Sesmic coefficients were calibrated for residual displacements 10 [mm] and averaged through different material scenarios (U1 to U8)   | **TR** | **Da** | **p** | **S8** | **S15** | **S22** | **S30** | **S38** | **S45** | **S52** | **S60** | **S68** | **S75** | | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | | 475 | 10 | mean | 3 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | | 975 | 10 | mean | 6 | 6 | 6 | 7 | 7 | 7 | 7 | 8 | 8 | 8 | | 1,975 | 10 | mean | 10 | 11 | 11 | 12 | 12 | 13 | 13 | 13 | 14 | 14 | | 2,475 | 10 | mean | 12 | 13 | 14 | 14 | 15 | 15 | 15 | 16 | 16 | 17 | | 4,975 | 10 | mean | 21 | 22 | 23 | 24 | 24 | 25 | 25 | 26 | 27 | 27 | | 9,975 | 10 | mean | 34 | 36 | 37 | 37 | 38 | 39 | 40 | 42 | 42 | 43 | |

## Summary

Slope performance is evaluated probabilistically by integrating site‑specific ground‑motion hazard, slope dynamic characterization, and a suite of Newmark‑type empirical and semi‑empirical displacement models. Shear stiffness, shear‑wave velocities, and fundamental periods are obtained from numerical models spanning representative geometries and material profiles for tailings and waste‑rock facilities. Permanent displacements are computed per scenario using a weighted ensemble of rigid‑ and flexible‑block Newmark models. Aleatory and epistemic uncertainties from ground motion, site amplification, and Newmark models are propagated via Monte Carlo simulation. Performance‑based seismic coefficients are established for different objectives by identifying the minimum yield acceleration that limits the probability of exceeding specified displacement thresholds.

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1. Hazard Model
   1. Appendix: Numerical Implementation in OpenQuake

The probabilistic seismic hazard analysis is performed with the OpenQuake engine. The input model follows the NRML structure: a source-model logic tree and a ground-motion logic tree describe epistemic alternatives and weights; realizations are built from their Cartesian product, with realization weights given by the product of branch weights ([GEM Foundation 2023, sec. 2.2.1](#ref-OpenQuakeManual); [Pagani et al. 2014](#ref-Pagani2014)).

Calculation settings are defined in the job configuration, including mesh spacing for source discretization, magnitude-frequency bin width, maximum integration distance, truncation level for the Gaussian distribution of , investigation time, intensity-measure types, and the grid of intensity-measure levels ([“Classical PSHA Configuration” 2024](#ref-OpenQuakeConfig)).

For each realization, OpenQuake constructs ruptures by sampling source geometry and magnitude bins. Rupture dimensions follow the configured magnitude–area or magnitude–length scaling relationship and aspect-ratio assumptions; a common option is the Wells–Coppersmith family of relations ([Wells and Coppersmith 1994](#ref-Wells1994)).

Ground-motion exceedance probabilities for each magnitude–distance scenario are computed from the selected GSIM branch using its median and dispersion and the required distance metrics (for example, and ) evaluated from the rupture surfaces, consistent with the hazard library interface ([Pagani et al. 2014](#ref-Pagani2014); [“Openquake.hazardlib.gsim — GSIM Interface and Distance Requirements” 2019](#ref-OpenQuakeGSIM)).

Hazard curves are produced on the user-specified grid of intensity-measure levels for each oscillator period. When epistemic uncertainties are present, OpenQuake stores per-realization curves and statistical aggregates. In OpenQuake, the mean at a given intensity level is the weighted arithmetic mean of branchwise probabilities of exceedance, and quantiles are weighted quantiles of those probabilities; statistics are thus taken in PoE space at fixed IMLs ([Weatherill et al. 2024](#ref-Weatherill2024); [GEM Foundation 2023, sec. 2.3.3.1](#ref-OpenQuakeManual)).

Uniform-hazard spectra are obtained by interpolating hazard curves at a fixed exceedance probability across periods, yielding spectral ordinates consistent with the chosen statistic (mean or quantile) ([*Hazard Calculators: Uniform Hazard Spectra* 2018](#ref-OpenQuakeHazardCalcs)).

Outputs and metadata are available through the standard export mechanisms. The documentation details per-site CSV outputs with poe-<IML> columns, the list of realizations and weights, and datastore/HDF5 contents for reproducibility ([“Outputs — Classical PSHA” 2025](#ref-OpenQuakeOutputs)).

The 2020 National Building Code of Canada (NBCC 2020) provides the authoritative national framework for seismic hazard parameters in Canadian engineering design. Seismic hazard values are derived from the Sixth Generation National Seismic Hazard Model for Canada ([Adams et al. 2020](#ref-Adams2020); [2020](#ref-Halchuk2020)), which applies a probabilistic seismic hazard analysis (PSHA) methodology consistent with international standards ([Geological Survey of Canada 2020](#ref-GSC2020)).

Official NBCC hazard values are distributed via the Natural Resources Canada online hazard tool ([Natural Resources Canada 2021](#ref-NaturalResourcesCanada2021c)). The tool accepts site location (latitude/longitude or address) and site condition (either NBCC site class or time-averaged shear-wave velocity, ). Output parameters include mean ground motion values for the principal annual exceedance probabilities (AEP): 0.002 ( years), 0.001 ( years), and 0.0004 ( years), corresponding to ordinary, important, and post-disaster structures, respectively ([National Research Council of Canada 2020b](#ref-NRC2020); [2020](#ref-Adams2020)). For each AEP, spectral accelerations are provided at peak ground acceleration (PGA), and structural periods ranging from 0.05 to 5 sec., assuming 5 % critical damping.

Site-specific PSHA results are compared directly with NBCC 2020 outputs for the matched location and condition. [[7]](#footnote-7) The NBCC tool enables interpolation at arbitrary locations and supports “mean” fractile output for code-compliant spectral values. [[8]](#footnote-8) The NBCC hazard tool is accessible at: <https://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php>. Agreement between the PSHA and NBCC 2020 values confirms consistency with the national standard model and supports use of the results for code-based engineering design. Small differences observed, specially in soft soils, are attributed to the site response model employed to complement GMPE models with limited support for different shear-wave velocities.

* 1. Appendix: Western Canada (WCAN) - Source Model

The Western Canada source model delineates four principal tectonic regimes controlling regional seismicity. The **Subduction Interface** regime comprises the Cascadia megathrust, modeled with three alternative down-dip fault depths (23 km, 27 km, and 31 km), alongside the Explorer interface, Alaska coastal, and Beaufort-Mackenzie convergence zones, characterized by interplate thrust and low-angle subduction mechanisms with magnitudes ranging from to .

The **Subduction Intraslab** regime includes the Juan de Fuca plate, Puget Sound, and Alaska inland deep zones, modeled as areal sources at depths of 30 km and 55 km, exhibiting intraslab normal faulting mechanisms with magnitudes from to .

The **Active Shallow Crust** regime encompasses significant fault systems (e.g., Denali, Fairweather, Queen Charlotte, Devils Mountain, Winona) and distributed area sources such as Brooks Peninsula, Glacier Bay, Mackenzie Mountains, and Rocky Mountain Fold/Thrust Belts. This regime is marked by strike-slip, crustal thrust, and oblique faulting mechanisms, with magnitudes from to for faults and up to 8.0 for selected area sources.

The **Stable Shallow Crust** regime covers regions including the interior craton, Foothills, Williston Basin, and cratonic cores, typified by low-activity seismicity characteristic of stable continental areas, with magnitudes ranging from to .

The source inventory comprises area sources defined by polygons discretized at 10–20 km intervals, employing truncated incremental Gutenberg–Richter (G–R) distributions with a bin width of . Simple fault sources, such as Denali and Fairweather, feature explicit trace geometries, dips, rakes, seismogenic depths, and recurrence models using either G–R ( or ) or near-characteristic () approaches. The Cascadia megathrust is represented as a complex fault source, incorporating epistemic uncertainty in slab geometry through alternative landward down-dip depths.

Recurrence parameterization employs truncated incremental Gutenberg–Richter recurrence for both area and fault sources, assigning completeness thresholds and maximum magnitudes specific to each region and source type. Major faults include branches for characteristic and hybrid alternatives, applying scaling relations such as WC1994 ([1994](#ref-Wells1994)) for most crustal sources, GSCCascadia, GSCEISB, and GSCOffshoreThrusts for subduction interfaces and thrusts, and CEUS2011 for the stable craton.

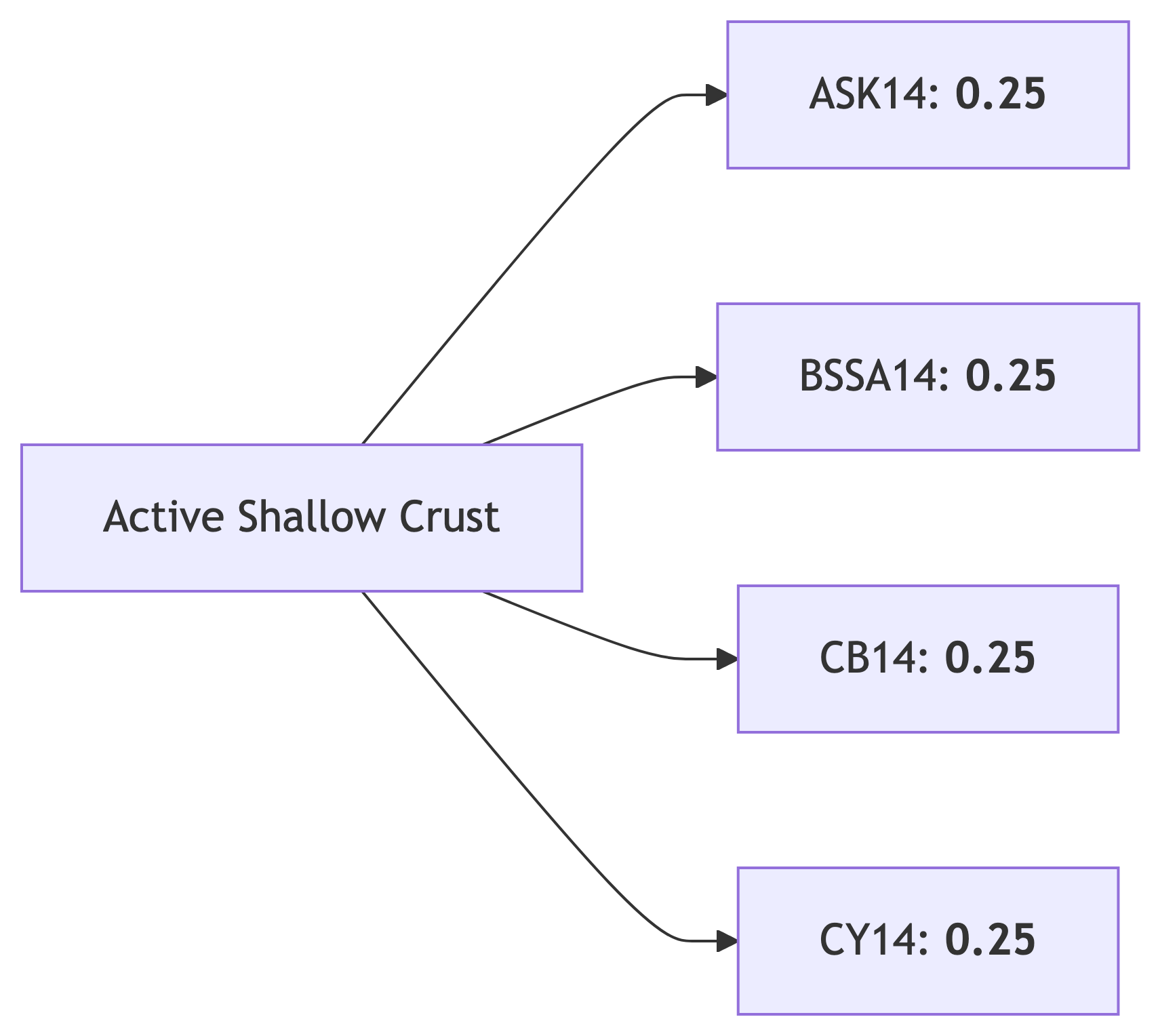
Activity rates for area sources commonly use , with alternative logic-tree branches assigning or where appropriate. Fault sources default to , with additional branches at and for significant faults. Nodal plane and hypocentral depth distributions reflect seismotectonic uncertainties within each regime.

Sources are categorized by tectonic region type (Subduction Interface, Subduction Intraslab, Active Shallow Crust, Stable Shallow Crust), adopting independence assumptions (src\_interdep="indep", rup\_interdep="indep"). Magnitude–frequency discretization applies incremental MFDs with defined magnitude ranges and bin width .

* 1. Appendix: Western Canada (WCAN) – GMPE Logic Tree

The ground motion logic tree for Western Canada is defined by the regional tectonic environment, encompassing active shallow crust, subduction interface, and subduction intraslab regimes. Each source type is mapped to a suite of ground motion models (GMPEs), with epistemic uncertainty quantified through logic tree branch weights. All assignments are fully consistent with the Canadian National Seismic Hazard Model (CanadaSHM6) ([Adams et al. 2022](#ref-Adams2022); [Goulet et al. 2021](#ref-Goulet2021)) and comply with contemporary international best practices for continental active and subduction settings.

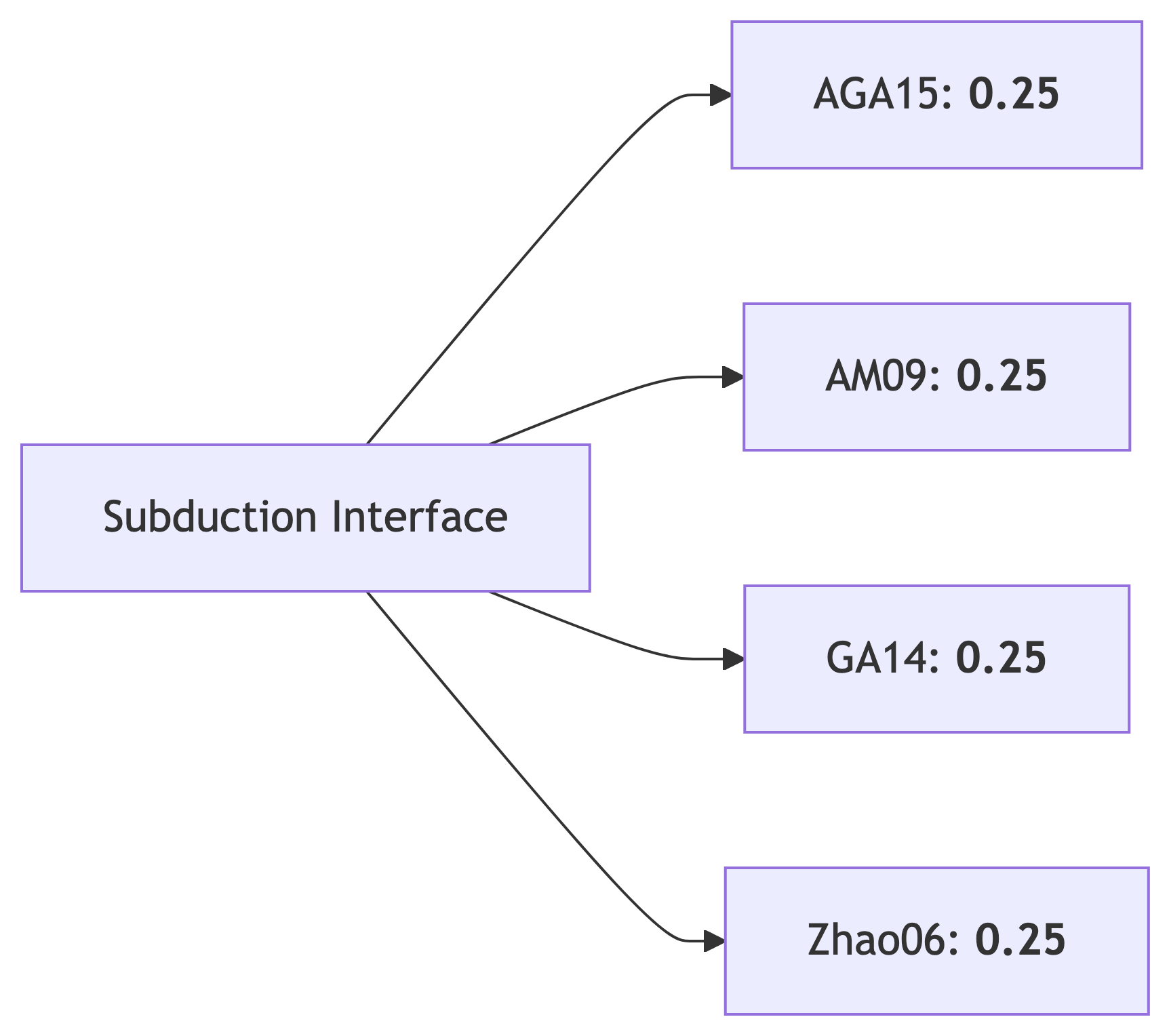
Active shallow crustal earthquakes in Western Canada are assigned to the active crustal regime, modeled by four NGA-West2 GMPEs with equal weights of 0.25 each: Abrahamson, Silva & Kamai ([2014](#ref-Abrahamson2014)), Boore, Stewart, Seyhan & Atkinson ([2014](#ref-BooreEtAl2014)), Campbell & Bozorgnia ([2014](#ref-Campbell2014)), and Chiou & Youngs ([2014](#ref-ChiouYoungs2014)). Each branch receives equal weight to ensure balanced representation of epistemic uncertainty.



Where:

* ASK14: Abrahamson, Silva & Kamai ([2014](#ref-Abrahamson2014))
* BSSA14: Boore, Stewart, Seyhan & Atkinson ([2014](#ref-BooreEtAl2014))
* CB14: Campbell & Bozorgnia ([2014](#ref-Campbell2014))
* CY14: Chiou & Youngs ([2014](#ref-ChiouYoungs2014))

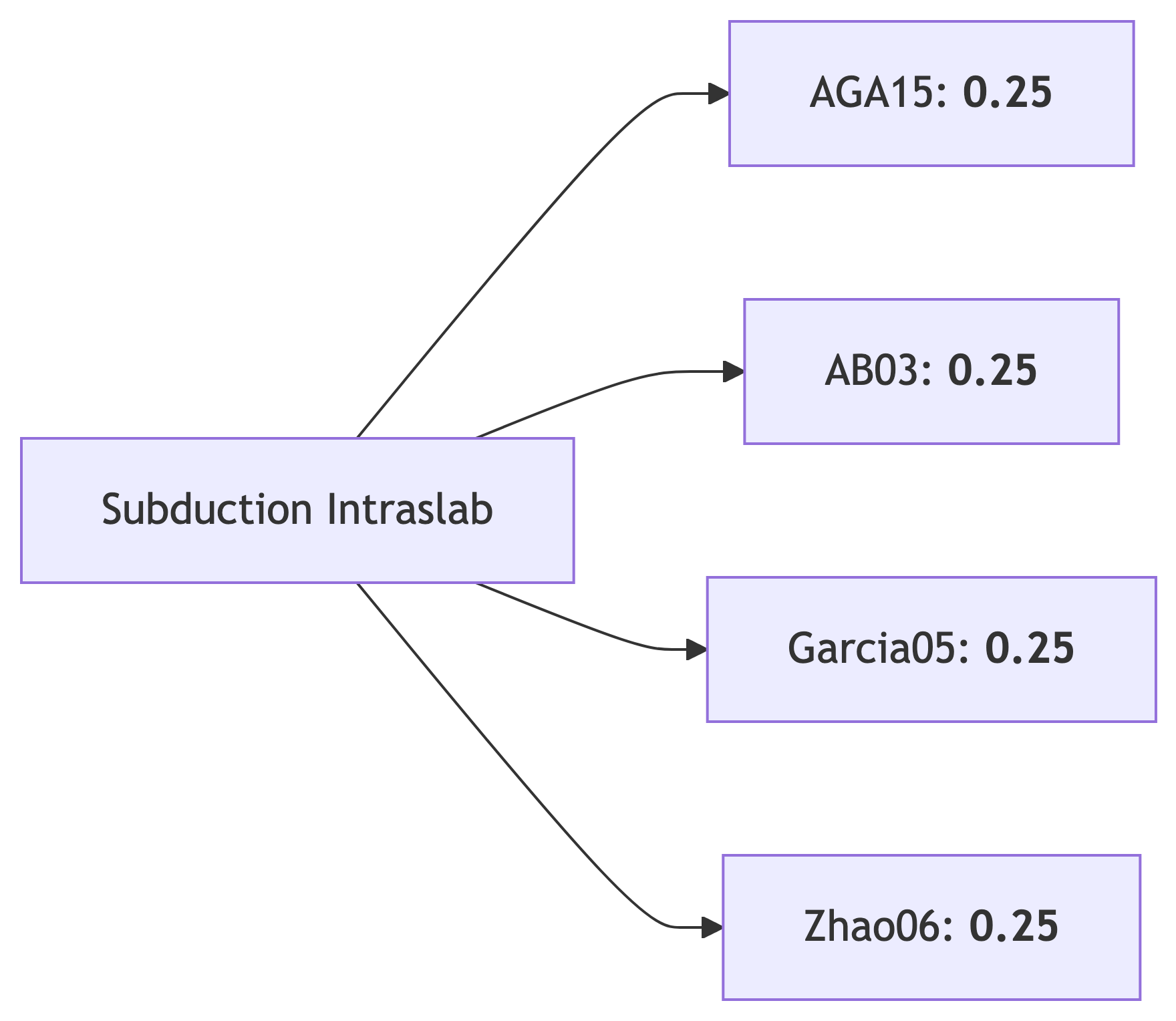
The subduction interface regime (Cascadia megathrust) utilizes a four-GMPE logic tree with equally weighted branches at 0.25: Abrahamson, Gregor & Addo ([2016](#ref-Abrahamson2015)), Atkinson & Macias ([2009](#ref-Atkinson2009)), Ghofrani & Atkinson ([2014](#ref-Ghofrani2014)), and Zhao et al. ([2006](#ref-Zhao2006)). These models collectively represent epistemic uncertainty in interface ground motions, with regional calibration specific to the Cascadia margin.



Where:

* AGA15: Abrahamson, Gregor & Addo ([2016](#ref-Abrahamson2015))
* AM09: Atkinson & Macias ([2009](#ref-Atkinson2009))
* GA14: Ghofrani & Atkinson ([2014](#ref-Ghofrani2014))
* Zhao06: Zhao et al. ([2006](#ref-Zhao2006))

The subduction intraslab regime (intermediate-depth events within the Juan de Fuca and related slabs) is similarly modeled with four equally weighted GMPE branches at 0.25 for both 30 km and 55 km depth classes: Abrahamson, Gregor & Addo ([2016](#ref-Abrahamson2015)), Atkinson & Boore ([2003](#ref-Atkinson2003)), Garcia et al. ([2005](#ref-Garcia2005)), and Zhao et al. ([2006](#ref-Zhao2006)).



Where:

* AGA15: Abrahamson, Gregor & Addo ([2016](#ref-Abrahamson2015))
* AB03: Atkinson & Boore ([2003](#ref-Atkinson2003))
* Garcia05: Garcia et al. ([2005](#ref-Garcia2005))
* Zhao06: Zhao et al. ([2006](#ref-Zhao2006))

All relevant seismic sources are assigned to active or subduction tectonic settings. The logic tree structure, GMPE selection, and weighting scheme are implemented exactly as specified in the externally referenced logic tree XML file. Model assignments, reference periods, and epistemic weights are inherited directly, ensuring no reinterpretation or reweighting.

1. Site Classification
   1. Appendix: Shear-Wave Velocity

The **Vs30** parameter is widely measured in the field using methods such as borehole logging, seismic cone penetration tests, and surface wave techniques. Introduced into engineering practice in the early 1990s, it provides a simple, robust metric to characterize site conditions for earthquake ground motions. **Vs30** is defined as the time-averaged shear-wave velocity in the upper 30 meters of a site profile. In practical terms, it represents the overall stiffness of the near-surface soils and rock. Mathematically, Vs30 is computed as the harmonic average of shear-wave velocity () over the top 30 m:

where and are the thickness and shear-wave velocity of the -th layer in the upper 30 m. This formula gives the equivalent uniform velocity that yields the same travel time through 30 m of material as the actual layered profile.

Vs30 has become the primary quantitative predictor of site effects in modern ground-motion models. Virtually all recent ground motion prediction equations (GMPEs) incorporate Vs30 as a key input parameter to adjust predicted shaking for site conditions. Empirical GMPEs (e.g., NGA-West and NGA-East) often use a functional form in which the median ground motion is scaled according to or a similar proxy, with separate terms for linear scaling and nonlinear saturation at low velocities. Large datasets have validated the use of Vs30 in GMPEs, showing consistent correlations between Vs30 and site amplification across many locations ([Borcherdt 2012](#ref-Borcherdt2012)).

In parallel, site-specific analyses (such as 1D site response simulations) rely on Vs30 as an initial classification of the site profile, although they incorporate the full shear-wave velocity profile with depth. While Vs30 does not capture all facets of site response—such as basin depth or resonance effects—it serves as a practical first-order proxy.

* 1. Appendix: NEHRP Site Classes

The NEHRP provisions ([2015](#ref-BSSC2015)) classify sites into categories (Site Class A through E, and special Class F) based on Vs30 ranges. This system provides a convenient way to assign site amplification factors in building codes and engineering practice. The standard classes and their Vs30 criteria are approximately:

* **Site Class A:** Hard rock, m/s (very high velocity material).
* **Site Class B:** Rock, 760–1500 m/s (typical competent rock; 760 m/s is the B/C boundary).
* **Site Class C:** Very dense soil or soft rock, 360–760 m/s.
* **Site Class D:** Stiff soil, 180–360 m/s.
* **Site Class E:** Soft soil, 180 m/s (includes clay-rich soils prone to high amplification).
* **Site Class F:** Soils requiring site-specific evaluation (e.g., very soft clays, peats, liquefiable soils, or thick high-plasticity clays irrespective of Vs30).

Classes A–E are defined by the Vs30 thresholds above. Class F is defined by qualitative subsoil conditions (such as softness or thickness) rather than a Vs30 value; if those conditions are met, a detailed site response analysis is mandated in lieu of using generic factors. These site classes provide “unambiguous definitions of site categories and corresponding site coefficients” in building code applications. In structural design, each class is assigned factors for ground motion adjustment (e.g. , in NEHRP/ASCE 7) that modify the standard response spectrum to account for local site amplification.

* 1. Appendix: AS1726 Guidelines

The Australian Standard AS 1726-2017 ([2017](#ref-AS1726-2017)) provides guidelines for conducting geotechnical investigations and includes a site classification system that categorises sites into classes based on their soil properties, such as shear wave velocity parameter , depth to bedrock, and soil type. The Australian Standard AS 1170.4 “Earthquake Actions on Structures” uses a similar site classification system based on the stiffness of the ground, but it does not explicitly use the shear wave velocity parameter to classify the ground. The categories range from A (rock) to E (soft soil), with a special category (S) for sites requiring site-specific investigations.

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Table B.1: Site Classification. Source: Australian Standard AS-1726   | **SC** | **Description** | **Vs30** | | --- | --- | --- | | A | Strong Rocks | >1500 [m/s] | | B | Rocks | >360 [m/s] | | C | Shallow soil site | >150 [m/s] | | D | Deep or soft soil site | >150 [m/s] | | E | Very soft soil site | <=150 [m/s] | |

* 1. Appendix: ASCE/SEI 7-22 Guidelines

The ASCE/SEI 7-22 Standard ([2022](#ref-ASCE722)) provides guidelines for conducting geotechnical investigations and includes a site classification system that categorises sites into classes based on their soil properties, such as shear wave velocity, depth to bedrock, and soil type. According to ASCE 7-22 ([American Society of Civil Engineers 2022](#ref-ASCE722)), the site soil shall be classified based on the average shear wave velocity parameter, , which is derived from the measured shear wave velocity profile from the ground surface to a depth of 30 m.

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| Table B.2: Site Classification. Source: American Society of Civil Engineers ASCE/SEI 7-22 (2022)   | **SC** | **Description** | **Vs30** | | --- | --- | --- | | A | Hard rock | >1500 [m/s] | | B | Medium hard rock | >910-1500 [m/s] | | BC | Soft rock | >640-910 [m/s] | | C | Very dense soil or hard clay | >440-640 [m/s] | | CD | Dense sand of very stiff clay | >300-440 [m/s] | | D | Medium dense sand or stiff clay | >210-300 [m/s] | | DE | Loose sand or medium stiff clay | >150-210 [m/s] | | E | Very loose sand or soft clay | <=150 [m/s] | | F | Soils requiring site response analysis (ASCE/SEI 7-22 21.1) | \*See ASCE 7 section 20.2.1 | |

* 1. Appendix: NBC-2020 Guidelines

The National Building Code of Canada ([2020a](#ref-NBC2020)) provides guidelines for site characterization and seismic site response. Similar to ASCE 7-22, it uses an average shear wave velocity in the upper 30 m (commonly denoted ), standard penetration test (SPT) blow count, and/or undrained shear strength () to classify sites into categories A through F. Where site-specific investigations are warranted (e.g., liquefiable soils, highly sensitive clays, peat), the code requires more detailed analysis.

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| Table B.3: Site Classification. Source: National Building Code of Canada NBC (2020)   | **SC** | **Description** | **Vs30** | | --- | --- | --- | | A | Hard rocks | >1500 [m/s] | | B | Rocks | >760-1500 [m/s] | | C | Very dense soils or soft rocks | >360-760 [m/s] | | D | Stiff soils | >180-360 [m/s] | | E | Soft soils | >140-360 [m/s] | | F | Other soils | <=140 [m/s] | |

1. Site Response
   1. Appendix: Ergodic Site Amplification Model

The site response methodology employs an **ergodic site amplification model** that yields regionally representative, non-site-specific amplification factors. The framework was developed for probabilistic seismic hazard analysis (PSHA) in Central and Eastern North America (CENA), based on companion studies addressing linear and nonlinear amplification components (see Stewart *et al.* [2020](#ref-Stewart2020); Hashash *et al.* [2020](#ref-Hashash2020)). The base linear amplification model is established by Stewart *et al.* ([2020](#ref-Stewart2020)), while Hashash *et al.* ([2020](#ref-Hashash2020)) quantifies the nonlinear modification for large-strain effects. In ergodic PSHA applications, ground motion on soil is derived by scaling a reference-rock ground motion with a site amplification factor. The **total amplification factor** can be decomposed in logarithmic form as:

where denotes the time-averaged shear-wave velocity in the upper 30 m, and is the oscillator period. The three right-hand terms represent: (1) the reference condition adjustment , (2) a linear site term that incorporates small-strain behavior as a function of , and (3) a nonlinear factor dependent on and reference-rock to account for soil nonlinearity. In non-logarithmic space, .

The model components are the following:

**Vs30 Scaling Term ():** The empirical linear amplification term quantifies site stiffness relative to the m/s reference. The function is period-dependent, approaches unity at m/s, and tapers as increases to 2000–3000 m/s. Distinct site profiles, including sharp impedance contrasts and gradual velocity gradients, are incorporated.

**Reference Condition Adjustment ():** bridges the difference between the 760 m/s reference and a hard-rock condition (e.g., m/s) adopted by some ground motion models. generally exceeds 1.0 and is period-dependent. For cases where the base motion is already referenced to 760 m/s, ; otherwise, the factor converts 3000 m/s motions to 760 m/s before applying .

**Nonlinear Amplification Term ():** The reduction factor for soil nonlinearity reflects strain-dependent behavior. At low , . As increases, decreases, quantifying the de-amplification associated with nonlinear soil response. is defined as a function of , , and , and is separately calibrated for both 760 m/s and 3000 m/s reference conditions.

All model components include quantified epistemic uncertainty for integration in PSHA calculations. The use of m/s and 3000 m/s as reference conditions reflects both engineering convention and regional geology:

* **760 m/s (NEHRP B/C Boundary):** Serves as the baseline for U.S. building codes and seismic design, corresponding to NEHRP site class B/C. Most strong-motion recordings fall within the 200–800 m/s range, and ground motion models typically adopt this as the reference class. Hazard or ground motion at 760 m/s is used for code-based design adjustments.
* **3000 m/s (Hard Rock):** Central and Eastern North America features near-surface crystalline bedrock with up to 3000 m/s. NGA-East models and other CENA ground motion models frequently reference this condition, particularly in national-scale mapping. Stewart *et al.* ([2020](#ref-Stewart2020)) defines procedures to translate hazard estimates from 3000 m/s to 760 m/s, maintaining compatibility with empirical data and code-based site classes. Models are well-calibrated for between 200 m/s and 1500–2000 m/s; extrapolation beyond this range is managed with tapering adjustments ([2020](#ref-Stewart2020)).

### Uncertainty in Site Amplification Model

Application of site amplification in PSHA requires the explicit treatment of variability and uncertainty. Let denote the reference rock ground motion (with its variance), and denote the amplification factor (with its own variance). The distribution of the resulting site ground motion, , exhibits higher variability than alone, unless is deterministic. Amplification factors are represented as random variables, with conditional median and a log-normal error term:

where is standard normal. Assuming independence, total aleatory variability is , where is the log-standard deviation of the GMPE for reference rock, and is that of the amplification factor.

Treatment of as aleatory or epistemic varies by implementation. In conventional and national-scale PSHA, site amplification variability is generally treated as **aleatory**—incorporated directly into total sigma (“sigma inflation”) or through **numerical convolution** of hazard curves with the amplification factor distribution. This propagates into the hazard curve. In alternative frameworks, such as NGA-East, a partially non-ergodic approach separates **site-to-site** from **record-to-record** variability. In non-ergodic, site-specific analyses, the mean amplification for a site is fixed per model branch, and is not included in the aleatory sigma. Instead, different amplification models/parameters are evaluated via a logic tree, and the uncertainty in is addressed through model weighting rather than sigma inflation. This prevents double-counting of site variability in well-characterized sites and shifts the primary uncertainty to model selection.

1. Design Criteria
   1. Appendix: Lifecycle Considerations

### Construction phase

Tailings dams are frequently constructed in stages, with embankments raised incrementally. Seismic stability during construction is a critical aspect, as the dam may be at partial height or incomplete strength. Design requirements stipulate that temporary factors of safety under seismic loading must be satisfied at all construction stages. Even with a low probability of major earthquake occurrence during the relatively brief construction phase, the potential consequences of a moderate event on an incomplete dam can be severe. Accordingly, initial and intermediate dam stages are analyzed for at least Operating Basis Earthquake (OBE)-level events; in high seismicity regions, Safety Evaluation Earthquake (SEE) criteria may also be applied. Temporary material properties and geometries are used in analyses, with design iterations as the dam evolves. Standards (e.g., CDA mining dams bulletin, 2014) recommend explicit consideration of seismic hazards at every project phase, supported by independent review at critical milestones. Construction methods and schedules are adjusted as needed to maintain seismic safety, including limiting impounded water volumes or avoiding vulnerable states during periods of heightened seismic risk.

### Operation (Active Life)

During operations, when the facility contains large volumes of tailings and water, the risk profile is highest. OBE and SEE criteria, assigned by consequence classification, govern seismic design for this phase. The dam must safely withstand the OBE without operational interruption, and must not release tailings under the SEE. Water management is integral—adequate freeboard must be maintained to accommodate seismic settlement, wave run-up, or ground deformations. For high or extreme consequence dams, design includes provision for concurrent storm and seismic loading (e.g., maintaining a 0.1% AEP storm pond without overtopping during seismic events, in line with ANCOLD and international practice). Operational seismic readiness is maintained through regular surveillance (crack/deformation monitoring) and emergency response planning. Engineering features—such as buttresses, drainage to control pore pressures, and control of loose, saturated zones—can enhance seismic performance. If an OBE is exceeded, post-event inspection and prompt repairs are required to restore the safety margin.

### Closure and Post-Closure

After the cessation of operations, TSFs are expected to remain physically stable for the indefinite future, in the absence of active management. Design for closure and post-closure phases is based on the principle that the probability of a rare seismic event approaches certainty over an infinite time horizon. Best practice is to require that all TSFs at closure are verified to withstand the Maximum Credible Earthquake (MCE) or a 1/10,000-year event, irrespective of their original consequence class. This approach is mandated by GISTM (2020) and ANCOLD, reflecting the expectation of permanent containment. Closure design is typically more stringent than operational design for lower consequence facilities. Rehabilitation measures (flattened slopes, reinforcements, erosion protection) may be implemented to achieve maintenance-free performance. Permanent spillways are designed for probable maximum floods, eliminating reliance on operational water management. Post-closure stability analyses must explicitly account for the degradation of temporary features. Components not expected to persist (e.g., geomembranes) are excluded from the long-term stability evaluation; the residual embankment must independently resist seismic loading. Long-term factors of safety are typically higher, and materials (cemented tailings, stabilizing berms) may be specified to enhance seismic resilience.

* 1. Appendix: Consequence Considerations

### GISTM Guidelines

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| Table D.1: GISTM Seismic Design Criteria by Consequence Class. Source: GISTM ([2020](#ref-gistm2020)) . The Global Industry Standard on Tailings Management (GISTM) defines five consequence classes for tailings storage facilities (TSFs), each based on **Population at Risk (PAR)** and the severity of environmental and socio-economic impacts. For all classes, the Maximum Credible Earthquake (MCE) governs if it results in more severe ground motions than the probabilistic criterion. Post‑closure criteria are uniform across all classes. GISTM does not use OBE/SEE terminology or dual‑level design earthquakes.   | **Class** | **PAR\*** | **Consequences** | **Operation/Closure†** | **Passive Closure‡** | | --- | --- | --- | --- | --- | | Low | None | No expected loss of life; minimal off‑site impacts. | 1/200 | 1/10,000 | | Significant | 1–10 | Minor but reportable off‑site impacts. | 1/1,000 | 1/10,000 | | High | 10–100 | Major localized impacts, potential for multiple fatalities. | 1/2,500 | 1/10,000 | | Very High | 100–1,000 | Substantial regional impacts, likely multiple fatalities. | 1/5,000 | 1/10,000 | | Extreme | >1,000 | Catastrophic, long‑term, widespread impacts, >100 likely fatalities. | 1/10,000 | 1/10,000 | | \*PAR = Population at Risk (off‑site persons) | | | | | | †Probabilistic annual‑exceedance criteria during operations and active closure; MCE governs if larger. | | | | | | ‡Probabilistic annual‑exceedance criteria during passive closure. | | | | | |

### CDA Guidelines

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| Table D.2: CDA Seismic Design Criteria by Consequence Category (CDA Mining Dams Technical Bulletin ([2021](#ref-cda2021)) . The Canadian Dam Association (CDA) defines five consequence categories, based on **Population at Risk (PAR)** and the scale of environmental and economic impact. The post-closure criterion (1/10,000 AEP or MCE) must be achieved before abandonment for all facilities. Lower classes may temporarily use a risk-based closure/transition approach if justified. MCE governs if it produces higher demand than the probabilistic AEP. SEE = Safety Evaluation Earthquake; PAR = Population at Risk. Closure is considered the final, perpetual phase of a tailings facility   | **Class** | **PAR∆** | **Consequences** | **Operation†** | **Closure‡** | | --- | --- | --- | --- | --- | | Low | 0 | Minimal off‑site or environmental impact | 1/100 | 1/1,000 | | Significant | 1–10 | Limited off‑site impact, no expected fatalities | 1/1,000 | 1/2,475 | | High | 11–100 | Potential for up to 100 fatalities, major local environmental/economic damage | 1/2,475 | 1/10,000\* | | Very High | 101–1,000 | Likely multiple fatalities, substantial regional impacts | 1/5,000\* | 1/10,000\* | | Extreme | >1,000 | Catastrophic, long‑term, and widespread impacts, >100 likely fatalities | 1/10,000\* | 1/10,000\* | | ∆PAR = Population at Risk (off‑site persons) | | | | | | †Earthquake AEP targets – Construction / Operation phase (CDA Table 3‑3, 2021) | | | | | | ‡Earthquake AEP targets – Closure & Passive‑Care phase (CDA Table 4‑2, 2021) | | | | | | \*SEE / MCE level: use the larger of the listed AEP or the site‑specific Maximum Credible Earthquake (CDA Tables 3‑3 & 4‑2, notes 2–3). | | | | | |

### ANCOLD Guidelines

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| Table D.3: ANCOLD Seismic Design Criteria by Consequence Category. Source: ANCOLD Guidelines on Tailings Dams ([2019](#ref-ancold2019)). The Australian National Committee on Large Dams (ANCOLD) defines six consequence categories, based on **Population at Risk (PAR)** and expected loss of life, with three gradations for High: High C, High B, and High A. The post-closure criterion (MCE or 1/10,000 AEP) must be achieved before abandonment for all facilities. OBE and SEE are assigned by consequence category. Prior to abandonment, all facilities must be reassessed under Extreme-class criteria (MCE or 1/10,000 AEP, 84th percentile ground motion). ANCOLD does not publish seismic criteria by lifecycle phase, but mandates upgrades prior to abandonment as necessary.   | **Class** | **PAR** | **Consequences** | **OBE** | **SEE** | | --- | --- | --- | --- | --- | | Low | 0 | No expected loss of life; negligible off‑site impact. | 1/475–1/1,000 | 1/1,000 | | Significant | 1–10 | Few persons at risk; minor off‑site or environmental impact. | 1/475–1/1,000 | 1/1,000 | | High C | 11–100 | Possible loss of life; moderate environmental/economic impact. | 1/475–1/1,000 | 1/2,000 | | High B | 101–1,000 | Likely multiple fatalities; major environmental/economic loss. | 1/475–1/1,000 | 1/5,000 | | High A | >1,000 (up to a few thousand) | Likely multiple fatalities; regional/catastrophic impact. | 1/475–1/1,000 | 1/10,000 | | Extreme | > several thousand | Very high loss of life expected; catastrophic, long‑term impacts. | 1/475–1/1,000 | MCE or 1/10,000 (85th percentile) | |

1. Slope Response
   1. Appendix: Newmark-based Sliding-Block Models

A suite of predictive models have been developed to estimate earthquake-induced permanent displacements in slopes and embankments using the sliding-block framework originally proposed by Newmark. These models range from fully empirical formulations based on observed seismic records to semi-empirical approaches that incorporate site-specific dynamic parameters and statistical regression. Each method implements a distinct combination of input parameters—such as yield acceleration, ground motion intensity measures, and spectral ordinates—to quantify the relationship between seismic demand and slope performance. The following sections present the principal displacement models currently recognized in the technical literature, with explicit definitions of parameters, functional forms, and associated uncertainties for engineering application. The general notation for all models is the following:

* : permanent displacement [cm].
* : yield-acceleration ratio .
* : peak ground acceleration [g]; : peak ground velocity [cm s].
* : fundamental period of the sliding mass [s]; is evaluated at a model-specific period.
* All dispersions are one‐standard-deviation values in natural-log space unless stated otherwise.

### Bray & Macedo ([2017](#ref-BrayMacedo2017))

Flexible-Block (compliant) Model. Subduction earthquakes

with

* : yield-acceleration ratio (dimensionless).
* : spectral acceleration at the fundamental period [g].
* : fundamental period of the sliding mass [s].
* : moment magnitude.

### Bray & Macedo ([2019](#ref-BrayMacedo2019))

Flexible-Block (compliant) Model. Shallow-crustal earthquakes. Same functional form as above, with coefficients

* : peak ground velocity [cm s]; if not supplied it is estimated from .
* : spectral acceleration at [g].
* Remaining symbols as in the 2017 model.

### Bray & Travasarou ([2007](#ref-BrayTravasarou2007))

Flexible-Block (compliant) Model.

* : spectral acceleration at [g].

### Jibson ([2007](#ref-Jibson2007))

Rigid-Block Model.

.

* : Arias intensity [m s]; estimated from if not provided.
* expresses slide strength relative to intensity.

### Saygili & Rathje ([2008](#ref-SaygiliRathje2008))

Rigid-Block Model. Multi-parameter model. Requires and

(increases as the slope becomes “weaker”)

### Ambraseys & Menu ([1988](#ref-AmbraseysMenu1988)) – rigid sliding block

Rigid-Block Model.

### Yegian et al. ([1991](#ref-Yegian1991))

Rigid-Block Model.

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* 1. Appendix: Shear stiffness parameters for gravels, sands and fines

The shear moduli of an arbitrary layer at depth can be estimated in terms of the void ratio and the octahedral stresses from [eq. 6.3](#eq-SM), where is a reference pressure, , , and are material-dependent constants and is the void ratio of the material, and is the octahedral stress at depth ([Ishihara 1997](#ref-Ishihara1997)).

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| Table E.1: Shear stiffness parameters for gravels, sands and fines. Source: Ishihara ([1997](#ref-Ishihara1997))   | **Material** | **Author** | **A** | **Ce** | **n** | **emin** | **emax** | | --- | --- | --- | --- | --- | --- | --- | | Round-grain Ottawa sand | Hardin & Richart (1963) | 7.00 | 2.174 | 0.50 | 0.30 | 0.8 | | Angular-grain crushed sand | Hardin & Richart (1963) | 3.27 | 2.973 | 0.50 | 0.60 | 1.3 | | Several sands | Shibata-Soelarno (1975) | 42.00 | 0.670 | 0.50 | 0.60 | 0.9 | | Several sands | Iwasaki et al.(1978) | 9.00 | 2.174 | 0.38 | 0.60 | 0.9 | | Toyoura sand | Kokusho (1980) | 8.40 | 2.173 | 0.50 | 0.60 | 0.8 | | Several sands | Yu-Richart (1984) | 7.00 | 2.173 | 0.50 | 0.60 | 0.9 | | Ticino sands | Lo Presti et.al (1993) | 7.10 | 2.270 | 0.43 | 0.60 | 0.9 | | Several undisturbed NC Clays | Hardin & Black (1968) | 3.27 | 2.973 | 0.50 | 0.50 | 1.7 | | Reconstituted NC Kaoline | Marcuson & Wahls (1972) | 4.50 | 2.973 | 0.50 | 1.10 | 1.3 | | Reconstituted NC Bentonite | Marcuson & Wahls (1972) | 0.45 | 4.400 | 0.50 | 1.60 | 2.5 | | Clay remolded IP=0-50% | Zen-Umehara (1978) | 2.00 | 2.973 | 0.50 | 1.60 | 2.5 | | Clay remolded IP=0-50% | Zen-Umehara (1978) | 3.00 | 2.973 | 0.50 | 1.60 | 2.5 | | Clay remolded IP=0-50% | Zen-Umehara (1978) | 4.00 | 2.973 | 0.50 | 1.60 | 2.5 | | Undisturbed NC Clay | Kokusho et al.(1982) | 0.14 | 7.320 | 0.60 | 1.70 | 3.8 | | Balast | Prange (1981) | 7.23 | 2.970 | 0.38 | 0.25 | 0.7 | | Crushed Rock | Kokusho-Esashi (1981) | 13.00 | 2.170 | 0.55 | 0.25 | 0.7 | | Rounded Gravel | Kokusho-Esashi (1981) | 8.40 | 2.170 | 0.60 | 0.25 | 0.7 | | Gravel | Tanaka et al.(1987) | 3.08 | 2.170 | 0.60 | 0.25 | 0.7 | | Gravel | Goto et al.(1987) | 1.22 | 2.170 | 0.85 | 0.25 | 0.7 | | Gravel | Nishio et al.(1985) | 9.36 | 2.170 | 0.44 | 0.25 | 0.7 | |

1. is a random variable representing the residual (in log units) for an individual event [↑](#footnote-ref-1)
2. For small (hazard levels of practical interest are usually low probability), (since ). [↑](#footnote-ref-2)
3. A common case is years (an approximate lifespan of structures); for example, if per year, then , i.e. a 10% probability in 50 years. [↑](#footnote-ref-3)
4. The NBCC 2020 reference condition is (NEHRP B/C boundary), consistent with GMPE industry practice and adopted in the NBCC since 2015 ([Halchuk et al. 2020](#ref-Halchuk2020)). Previous Canadian codes used a “firm ground” condition near (Site Class C), but the current model allows for specification across a broad range, including hard rock (), supporting velocity-based site adjustment ([David M. Boore et al. 2014](#ref-BooreEtAl2014)). [↑](#footnote-ref-4)
5. NBCC spectral accelerations represent uniform-hazard values at each period and do not account for multi-period joint probability or nonlinear site effects unless input parameters or code-based site factors are appropriately matched. The engineering design process may require additional site amplification factors (, ) or site-specific response analyses, as specified in the code commentary ([National Research Council of Canada 2020b](#ref-NRC2020)). [↑](#footnote-ref-5)
6. In practice, this distance could be zero if the site lies within the source zone. To avoid singularities in ground-motion predictions, small minimum distance are imposed in Openquake model [↑](#footnote-ref-6)
7. The NBCC 2020 reference condition is (NEHRP B/C boundary), consistent with GMPE industry practice and adopted in the NBCC since 2015 ([Halchuk et al. 2020](#ref-Halchuk2020)). Previous Canadian codes used a “firm ground” condition near (Site Class C), but the current model allows for specification across a broad range, including hard rock (), supporting velocity-based site adjustment ([David M. Boore et al. 2014](#ref-BooreEtAl2014)). [↑](#footnote-ref-7)
8. NBCC spectral accelerations represent uniform-hazard values at each period and do not account for multi-period joint probability or nonlinear site effects unless input parameters or code-based site factors are appropriately matched. The engineering design process may require additional site amplification factors (, ) or site-specific response analyses, as specified in the code commentary ([National Research Council of Canada 2020b](#ref-NRC2020)). [↑](#footnote-ref-8)