

Appendix 6.2-E

Afton Tailings Impoundment - Seismic Hazard and Seismic Stability Assessment

AJAX PROJECT

**Environmental Assessment Certificate Application / Environmental Impact Statement
for a Comprehensive Study**

Afton Operating Corporation

AFTON TAILINGS IMPOUNDMENT

Seismic Hazard and Seismic Stability Assessment

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EXECUTIVE SUMMARY

A review of the seismic hazard and seismic stability of the closed Afton tailings impoundment has been completed by Klohn Crippen Berger Ltd. (KCB). A site-specific seismic hazard assessment was carried out to establish seismic ground motion parameters. Both probabilistic and deterministic seismic hazard assessments were performed. The probabilistic assessment gave higher values for the peak ground acceleration (PGA) than the deterministic assessment, so the results of the probabilistic assessment were used in the seismic stability analysis. It was conservatively assumed that the glacial till foundation of the Afton tailings impoundment belongs to the reference NEHRP site class C (very dense soil and soft rock) / D (stiff soil) boundary.

Ground motion parameters of 0.34 g PGA and M7.3 earthquake magnitude corresponding to the 10,000 year return period were used to perform a seismic stability analysis of the tailings impoundment. An analysis of the available Standard Penetration Test (SPT) data indicated that liquefaction is not a concern in the foundation soils. A limit equilibrium stability analysis was performed for the maximum section of the West Dam, representing the critical section for stability. The factors of safety calculated in the analysis for static, pseudo-static, and post-liquefaction stability met or exceeded the minimum targets. Seismic deformation of the West Dam was estimated using the Hynes-Griffin and Franklin (1984) method. The computed deformation (mean plus one standard deviation) was 11 cm, well within acceptable limits.

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1. INTRODUCTION

This report presents a review of the seismic hazard and seismic stability of the closed Afton tailings impoundment. Section 2 describes the site-specific seismic hazard assessment that was performed, and presents the seismic ground motion parameters determined from this assessment. These ground motion parameters were used in a seismic stability assessment, described in Section 3. Conclusions and recommendations from the seismic hazard and stability assessments are given in Section 4.

2. SEISMIC HAZARD ASSESSMENT

2.1 General

A site-specific seismic hazard assessment was carried out to establish seismic ground motion parameters for the Afton mine site located in Kamloops, BC. The representative site coordinates for the West Dam at the mine site were taken as 50.65°N and 120.54°W. Figure 2.1 shows the location of the West Dam at the Afton mine site. The East Dam is located about 1 km east of the West Dam.

Probabilistic seismic hazard assessment was conducted using the Cornell-McGuire method embodied in the computer program Ez-Frisk (Risk Engineering Inc., 2009); both aleatory and epistemic uncertainties in the ground motion estimates were considered. Three alternative seismic source zone models consisting of areal sources were developed and used in the probabilistic seismic hazard assessment. An earthquake catalogue consisting of historical earthquake data up to April 2010 was used in the development of areal source zone model and model parameters. Selected ground motion prediction equations were used in the assessment to predict ground motions due to shallow crustal earthquakes and deep intraplate (or in-slab) earthquakes. The epistemic uncertainties in the model, model parameters and attenuation equations were treated following a logic tree approach. In addition, a deterministic seismic hazard analysis was also conducted to assess the impact of the earthquakes that may occur on known and unknown or blind faults around the project site.

2.2 Review of GSC Seismic Hazard Model

The 2005 National Building Code of Canada (NBCC) was based on a countrywide seismic hazard model developed by the Geological Survey of Canada (GSC) in the mid 1990s (Adams and Halchuck, 2003). The GSC model incorporated some of the advances in the understanding of the seismicity and reflected the current state of practice for

seismic hazard assessment as of about 1995. The model is regional and it was intended to provide ground motions for the 2005 NBCC, which uses a probability level of 2% in 50 years (or 2,475 year return period). Extrapolation of ground motions to lower probability levels, such as for a 10,000 year return period, is not recommended by the GSC as their model is not intended to provide motions at such low return period level (Halchuck, 2007). The GSC warns against using its model to establish Maximum Credible Earthquake (MCE) parameters for very high or extreme consequence category dams. Furthermore, there is significant new information on seismicity and ground motions since the GSC model development in the mid 1990s. Therefore, a site specific seismic hazard assessment was carried out to establish the ground motion parameters for the Afton dam sites.

2.3 Tectonic Setting, Earthquakes and Faults

Figure 2.2 shows the tectonic setting of the region surrounding the Afton site. The active tectonics in the offshore region to the west of British Columbia is dominated by the complex interactions of plate boundaries consisting of various plates including Pacific plate, Juan de Fuca plate, Explorer plate and North American plate. Complex tectonic movements are ongoing along the plate boundaries including oceanic floor spreading, transform faulting and subduction (Riddihough, 1982; Adams and Clague, 1993).

British Columbia is located within the North American plate, along the northeast margin of the Pacific plate and consists entirely of continental crust. Inland from plate boundary, the continental mass is largely mountainous with a complex geology and tectonics. Large scale faulting with historical movements of up to several hundreds of kilometres is common. However, most on-land faults within BC are not recognized as being active at present. Most of the Pacific Ocean basin is underlain by the Pacific plate, which is the largest tectonic plate and is almost entirely composed of mafic oceanic crust. About 90% of the earthquakes in the world occur in a band of seismicity that follows the tectonically

active margins. However, the rate of seismicity at the Pacific Northwest is somewhat lower than the seismicity in other portions of the Pacific Plate.

The Juan de Fuca, Explorer and Gorda (off northern California) plates comprise the Cascadia subduction zone along the active plate margin with the North American plate (see Figure 2.2). These plates are subducting beneath the North American plate although the Explorer plate is progressively slowing down as it is jammed against the North American Plate and becoming partially coupled to the north moving Pacific plate (Riddihough, 1977). The Cascadia subduction zone extends from the northern terminus of San Andreas Fault off Northern California to the southern end of the Queen Charlotte Fault off the north end of the Vancouver Island. The three dimensional geometry of the subduction zone is now relatively well defined. Figure 2.2 shows the depth contour of the Cascadia Subduction zone (Flück et al., 1997). The Cascadia Subduction zone follows a distinctive curve in plain view from a north-south strike off the coast of Washington and Oregon to a nearly north-westerly strike off Vancouver Island with a change in strike of about 30 degrees.

North of the subduction zone, the Pacific Plate moves relative to the North America Plate. Along the west coast, this motion is largely accommodated by dextral transcurrent motion along the Queen Charlotte and Fairweather Faults.

The seismic hazard for regions in and around the Subduction zone comes from three sources: (1) Crustal seismicity in the North American plate; (2) Great earthquakes of the Cascadia subduction zone on the interface between the North American and subducting Juan de Fuca plate; and (3) Deep earthquakes within the subducting slab ("in-slab" earthquakes). It is, however, the deep in-slab earthquakes that dominate the hazard in this region as their rates are approximately five times higher than the rates of shallow crustal earthquakes and their shaking levels are also greater for the same size (Adams and

Halchuck, 2002). Well known historical earthquakes in the Georgia Straight/Puget Sound region (M6.9 in 1949, M6.7 in 1965 and M6.8 in 2001) are in-slab earthquakes. The last great subduction interface-earthquake occurred approximately 310 year ago. These great earthquakes can have magnitude as large as M9 and have approximate mean recurrence rate of 600 years (Adams, 1990).

At the Afton site, the seismic hazard can come from the same three sources, i.e. shallow crustal earthquakes, great subduction interface earthquake and deep in-slab earthquake. However, the subducting Juan de Fuca plate is located too far away from the Afton site to cause any significant hazard due to either the great interface earthquake or deep in-slab earthquakes. There have been no deep historical earthquakes with depths greater than 20 km within about 225 km of the Afton site. The hazard at the Afton site is dominated by shallow crustal earthquakes only. The seismic activity north of the Afton site has also been low in an area confined by the Queen Charlotte Fault to the west and the North Rocky Mountain Trench to the east. The seismic activity generally decreases in the easterly direction towards Afton site from the active subduction zone. However, relatively more historical earthquakes have occurred in the Southern Rocky Mountain area located east of the Afton site.

Major faults onshore within BC generally trend in north-westerly direction consistent with the overall regional tectonic pattern but many faults also strike across this trend. Most reports describe the major fault movements in BC as having occurred several tens of million years and there has never been a published case of Quaternary surface rupture associated with an earthquake in BC, although surface traces of many faults cannot closely be followed due to erosion, deposition and short history of historical seismicity. Figures 2.3a and 2.3b shows the major regional and local faults, respectively, surrounding the Afton site. None of the faults shown in Figure 2.3a and 2.3b are considered active except for the Hell Creek fault (#9 in Figure 2.3a). This fault has a mapped length of

4.5 km and a projected length of 10 km and is considered a splay of Yalakom Fault. Roddick and Hutchison (1973) identified this fault as possibly active. However, this fault is located more than 125 km from Afton site.

2.4 Seismicity

2.4.1 Earthquake Catalogue

The instrumental catalogue for Western Canada, containing data from January 27, 1700 to December 31, 2007, was obtained from Pacific Geoscience Center (PGC) (Cassidy, 2008). Earthquake data from January 1, 2008 to April 30, 2010 were downloaded from the online database maintained by Natural Resources Canada (NRCan, 2009a) and added to the PGC's earthquake catalogue.

The original magnitude type in PGC catalogue was generally M_L , especially in the interior of British Columbia, sometimes m_b for offshore events, and often M_s for the older larger events. For more recent events (since about 1995), M_w has been computed directly for earthquakes larger than about M_4 . PGC has used the conversions listed below, which were also used by USGS in their development of seismic hazard maps for the United States (Petersen et al., 2008), to convert m_b , M_s and M_L type earthquakes magnitudes into M_w (Cassidy, 2008).

- $M = 1.46 * m_b - 2.42$ for $m_b > 5.3$, and $M = m_b$ otherwise.
- $M = 0.75 * (M_s + 1.93)$ for $M_s < 5.8$, $M = 1.50 * (M_s - 2.60)$ for $M_s > 7.8$, and $M = M_s$ otherwise.
- $M = 1.67 * (M_L - 2.60)$ for $M_L > 6.5$, and $M = M_L$ otherwise.

The recent earthquake data downloaded from online database maintained by NRC Canada were either in M_w or M_L . The M_L data were converted to M_w using the above relationship.

It is noted that the magnitude conversion was not done in the GSC hazard calculations to establish parameters for the 2005 NBCC (Adams and Atkinson, 2003).

2.4.2 Historical Earthquakes

Figure 2.4 shows the historical earthquakes with magnitude $M \geq 3$ within about 600 km of the Afton site. Figure 2.5 shows the historical earthquakes with magnitude $M \geq 2.5$ within about 100 km of Afton. Only the earthquakes that occurred north of 47° latitude are shown in Figures 2.4 and 2.5. Table 2.1 lists the historical earthquakes within 300 km of the Afton site.

The largest earthquakes that occurred within 50 km, 100 km and 200 km are $M_{3.2}$, $M_{4.5}$, and $M_{5.5}$, respectively, and they occurred approximately 7 km, 76 km and 189 km from the Afton site, respectively, in 1970, 1936 and 1926, respectively. As can be seen from Figure 2.5 and Table 2.1, the seismic activity within 100 km of the Afton site with magnitude greater than M_3 in the last century was relatively low.

Table 2.1 List of Historical Earthquakes

Year	Month	Day	Hour	Min	Sec	Longitude (deg)	Latitude (deg)	Depth (km)	Moment Magnitude, M_w	Epicentral Distance (km)
Earthquakes within 0-50 km with magnitude $M_w \geq 2$										
1970	11	16	2	41	47.0	-120.6	50.6	18	3.2	7
1984	11	16	7	4	39.0	-120.6	50.4	18	3.0	32
1994	1	4	0	24	5.5	-120.1	50.4	10	3.0	41
1981	2	26	14	28	58.0	-121.0	50.8	18	2.7	38
1994	7	28	0	1	38.6	-121.1	50.5	5	2.6	44

Table 2.1 List of Historical Earthquakes (cont'd)

Year	Month	Day	Hour	Min	Sec	Latitude (deg)	Longitude (deg)	Depth (km)	Moment Magnitude, Mw	Epicentral Distance (km)
Earthquakes within 50-100 km with magnitude Mw ≥ 3										
1936	3	28	9	15	0.0	-119.5	50.5		4.5	76
2002	8	17	16	6	27.2	-120.3	50.0	10	4.4	79
1993	9	19	11	24	4.1	-120.4	50.2	10	3.7	55
2003	8	20	8	33	4.1	-120.3	50.0	10	3.7	79
1989	4	9	10	33	32.8	-119.5	50.5	5	3.5	78
1956	10	4	7	30	27.0	-119.7	50.8		3.3	62
1979	4	22	2	40	24.0	-119.4	50.6	18	3.0	84
Earthquakes within 100-200 km with magnitude Mw ≥ 4										
1926	9	17	23	14	40.0	-123.0	50.0		5.5	189
1926	9	22	21	9	50.5	-121.9	50.2		5.0	107
1962	8	28	19	19	59.0	-121.9	51.7		4.3	151
Earthquakes within 200-300 km with magnitude Mw ≥ 5										
1918	2	4	20	37	43.2	-118.4	52.3		6.0	237
1909	1	11	23	49		-122.8	48.7		6.0	270
1976	5	16	8	35	15.1	-123.4	48.8	60.0	6.0	287
1915	8	18	14	5		-121.4	48.5		5.6	247
1942	1	31	6	49	11.3	-123.6	51.2		5.5	223
1864	10	29	14	55		-123.3	48.8		5.5	286
1920	1	24	7	10	0.0	-123.0	48.6		5.5	287
1926	12	4	13	55		-123.0	48.5		5.0	296

2.4.3 Subduction Zone and In-slab and Interface Earthquakes

Figure 2.6a shows the plate boundaries in the Cascadia Subduction zone and the location of Afton mine site. It also shows the historical earthquakes with $M \geq 3$ within a grid of 49°N - 51°N 110°W - 130°W . Figure 2.6b shows the same earthquakes on a cross section through the Afton site and the approximate interface of subducting Juan de Fuca plate along the 49°N latitude. The interface of subducting Juan de Fuca plate at the shortest distances from the Afton site is also shown in Figure 2.6b, and was estimated from the depth contours based on Flück et al. (1997).

As shown in Figure 2.6a, the Afton site is located near the northern end of the subducting Juan de Fuca plate but far away from the plate boundary. The approximate shortest distances to plate boundary at depths 30 km, 50 km, 70 km and 100 km are 350 km, 285 km, 230 km and 160 km, respectively. As highlighted in Figure 2.6b, there have been no deep historical earthquakes (with depths >30 km) in the last century within about 200 km from the Afton site. All the deep earthquakes, which could be considered as intraplate or in-slab earthquakes, occurred more than 200 km from the Afton site. Therefore, it is unlikely that deep in-slab earthquakes will have much influence on the seismic hazard at the Afton site. Figure 2.6 also shows that the subduction interface is located at more than 400 km from the Afton site. Thus, the mega subduction interface earthquake is unlikely to have great influence on the seismic hazard at Afton site.

2.5 Probabilistic Seismic Hazard Assessment

2.5.1 Methodology

The quantification of the probabilistic seismic hazard at the site is estimated using the well-known Cornell-McGuire approach. The steps in a probabilistic seismic hazard assessment (PSHA), as shown in Figure 2.7, consist of defining seismic sources, either areal or linear faults; definition of the earthquake frequency within each source zone; definition of the attenuation of ground shaking relationship for earthquakes in the area, and, finally, numerical summation of the contributions of all earthquake magnitudes at all distances from the site from each source. The computer program EZ-Frisk (Risk Engineering Inc., 2009) is used to perform the calculations in the last step.

2.5.2 Input Parameters

The input parameters typically required to calculate the earthquake ground motions using the Cornell-McGuire method for areal source zones include:

- The source zone coordinates;
- β (slope of the magnitude-recurrence relation);
- The focal depth (or range of depths);
- The minimum magnitude to consider in the hazard calculations;
- The maximum magnitude that is expected to be possible within the source zone;
- The number of annual earthquakes exceeding the minimum magnitude (activity rate); and
- Ground-motion relations giving site amplitude as a function of magnitude, distance, shear wave velocity in the top 30 m, etc.

For fault sources, typically, either the activity rate or slip rate is used. In addition, they can also be modeled using the rate at its characteristic magnitude.

2.5.3 Uncertainties and Treatment of Uncertainty

Two types of uncertainties associated with the seismic hazard were considered in the analyses, namely the aleatory uncertainty and the epistemic uncertainty. The aleatory uncertainty or the random uncertainty is due to the physical variability of the earthquake processes such as the randomness of the location of the earthquakes and the scatter in the earthquake ground motions. This uncertainty is readily incorporated within the Cornell-McGuire analysis framework by integrating over the statistical distribution in the ground motion relations and by considering the randomness in earthquake location.

The epistemic uncertainty or the professional uncertainty is due to incomplete understanding the physical models governing the earthquake occurrence and ground motion generation, i.e. selection and characterization of sources zones, ground motion

relations, etc. The epistemic uncertainty was considered in the analyses following a logic tree approach.

In the logic tree, uncertainties such as the different choices for source zones, attenuation relations, source zone model parameters such as the maximum magnitude, depth, etc., are weighed subjectively. Each calculated hazard result (i.e. probability of exceedance of a specified motion) is multiplied by the branch probability. The weighted branch probabilities are summed to obtain the mean hazard results, or may be ordered to calculate, for example, 50th (median), 16th and 84th percentiles; thus, for a specific ground motion parameter value (e.g. PGA), the mean frequency of exceedance can be calculated along with the median value and other confidence limits. The uncertainties in the source zone model and the ground motion relations are the key parameters that significantly affect the seismic hazard estimates.

2.5.4 Seismic Source Zone Models

Three alternative source zone models were developed for the Afton project based on the tectonic setting, geological structures and the historical seismicity to estimate the seismic hazard for the Afton site. Figures 2.8, 2.9 and 2.10 show the three alternative source zone models. In the development of the models, the following factors were considered:

- The Afton site is located far away from major plate boundaries located near the coast involving the subducting Juan de Fuca plate, explorer plate and North American plate, which are seismically active areas. The historical earthquake data in Figure 2.8 shows that the seismic activity generally decreases in the easterly direction from the plate boundaries to the southern rocky mountain trench where relatively more activity was observed;
- As shown in Figure 2.6a, the subducting Juan de Fuca plate is located far away from the Afton site that it is unlikely that any deep in-slab earthquake on the subducting Juan de Fuca plate will have any significant influence on the seismic hazard at Afton site. As illustrated in Figure 2.6b,

there have been no deep earthquakes with depth greater than 20 km within about 225 km from the Afton site. The seismic hazard at Afton site is dominated by shallow crustal earthquakes;

- Subduction interface mega earthquakes with magnitude up to M9, which may occur at shallow depths at the Juan de Fuca and North American plate boundary, is located more than 400 km from the Afton site. Therefore, it is unlikely this mega interface earthquake will have any significant influence on the seismic hazard at the Afton site;
- The seismically active areas associated with the Queen Charlotte fault are located offshore and also too far away from Afton site to influence hazard;
- Figure 2.3a shows the known regional faults around the Afton site. None of the faults located in the vicinity of the Afton site have been identified as active. BC Hydro indicates that the Hell Creek fault could be considered as active. However, this fault is located more than 75 km from the Afton site. The other known on-land active faults (Seattle Fault, Southern Whidbey Island Fault, Devils Mountain fault, etc.) are located south of the US-Canadian border, far away from the Afton site; and
- There is a known seismically low active area located north of the Afton site, which is bounded by the Queen Charlotte fault to the west and Northern Rocky Mountain trench to the east. This zone contains the Coast Range-Coast Shear Zone, which is considered inactive.

Figure I.1 in Appendix I shows the GSC-H model for the Western Canada and the location of Afton site. The GSC-H source zone model was used as the base model for the source zone Model 1 shown in Figure 2.8, in which the entire zone east of the more active subduction zone is captured in a new zone named SEBC12. Most of the regional faults are also included in this SEBC12 zone. In this model, the GSC-H model zones within the subduction zones were retained without modification to their boundaries. They include zones NJFF, JDF, SCM, CAS, GEO and PUG. Among them, GEO and PUG zones represent deep in-slab earthquakes that occur on the subducting Juan de Fuca plate, and all the other zones represent shallow crustal earthquakes. Although the GEO zone is contained within the SCM zone, they represent earthquakes arising from two different

sources: deep in-slab and shallow crustal, respectively. Similarly the zones PUG and CAS also represent earthquakes from deep and shallow sources, respectively.

Model 1 assumes that the seismic activity observed east of the more active subduction zone is possible anywhere within the SEBC12 zone and this model has the effect of spreading the pockets of activity over a much broader area. It also assumes that the shallow and deep seismic activity associated with the subduction zone would not extend to the Afton site and limited to the GSC-H model zones located west of the SEBC12 zone. In Model 1, a new zone NBC' was introduced to capture the low seismic activity northwest of the Afton site and the boundaries of GSC model zones NRMT and SFT were modified to create the new zones NRMT' and SFT'.

Figure 2.9 shows alternative Model 2, which is similar to Model 1, except that the SEBC12 zone in Model 1 is subdivided into SEBC1 and SEBC2 zones as shown in Figure 2.9. In this model, the Afton site is located within the SEBC1 zone, which captures seismic activity within about 100 km band located parallel to the subducting Juan de Fuca plate and North American plate boundaries.

Figure I.2 in Appendix I shows the GSC R model and the approximate location of Afton site. The GSC R model was used as a base model in the Model 3 shown in Figure 2.10. In Model 3, all the GSC-R model zones, except CAS and SBC zones, were retained without modification to their boundaries.

The Afton dam sites are located at the boundary between the more active CAS' zone and the less active SBC' zone in Model 3. Both CAS' and SBC' zones represent the shallow crustal earthquakes. Deep in-slab seismic earthquakes on the subducting Juan de Fuca plate are represented in Model 3 by the GSP zone shown in Figure 2.10, which is isolated from the shallow zone CAS' by depth.

In Model 3, seismically more active subduction zone was not excluded from the zone containing the Afton site as in Models 1 and 2. Instead, the seismic activity associated with the subduction zone was spread across the CAS' zone. This aids in the consideration of uncertainty in tectonic zonation and seismicity rates.

It is expected that Models 1 and 2 will provide a better estimate of seismic hazard at Afton dam sites which exclude the seismicity within the subduction zone. In Model 3, the CAS' zone includes seismicity in the subduction zone. Thus, in the probabilistic seismic hazard assessment, forty percent weight was assigned to each of Models 1 and 2, and twenty percent weight was assigned to Model 3.

2.5.5 Magnitude Recurrence Relationships

The earthquake recurrence within each source zone was assumed to follow the well-known Gutenberg-Richter relationship:

$$\text{Log [N(>M)]} = a - bM \quad (1)$$

where $N(M)$ = the number of events per year greater than M
 a = activity rate at M_{\min}
 b = slope of the relationship.

The Gutenberg–Richter parameters for each source zone are determined by plotting the logarithm of the number of events per year against earthquake magnitude. These relationships are dependent on the reliability of the earthquake records in an area.

The magnitude completeness of earthquake records varies significantly with time. The magnitude and location of historical earthquakes prior to the deployment of seismic monitoring stations across Western Canada were estimated based on shaking intensity reports and such information were available only for large magnitude earthquakes. The instrumental earthquake recording in Western Canada began in 1899 with the deployment

of seismograph in Victoria, BC. Since then, more instruments were deployed. The early instruments were low gain instruments and could detect only large magnitude earthquakes and recently low period large gain instruments were deployed which could detect small magnitude earthquake. Figure 2.11 shows current seismic monitoring stations in the Western Canada, which are part of Canadian National Seismographic Network (CNSN).

Magnitude recurrence relationships for the proposed source zones were developed using the PGC earthquake catalogue updated to April 30, 2010. Duplicates were removed from earthquake catalogues. Aftershocks were not removed from the earthquake catalogue for events in the region under consideration. This is consistent with the procedure used by the GSC to develop the seismic hazard maps for 2005 NBCC (Adams and Halchuck, 2003). The inclusion of aftershocks violates the assumption of Poisson independence; however, large aftershocks contribute to the seismic hazard. It is often difficult to decide if earthquakes have occurred as mainshock-aftershock sequences, or as swarms with many events of similar magnitude. In general, the effect on the magnitude-recurrence relations of including aftershocks is a small change in the recurrence slope (Basham et al., 1982).

Prior to calculating the magnitude-recurrence relationships for each of the source zones, earthquake data were removed from the database for periods where the data were incomplete within a given magnitude range. This prevents bias in the computation of per annum activity rates at each magnitude level. Table 2.2 shows the magnitude completeness as a function of time used for all source zones. Magnitude completeness year of 1970 for M3 means that the earthquake catalogue contains all the earthquakes greater than or equal to M3 that occurred since 1970.

The magnitude completeness data shown in Table 2.2 were based on the magnitude completeness data provided for GSC's H and R models (Adams and Halchuck, 2003). The completeness years shown in Table 2.2 for the SEBC12, CAS' and SBC' zones were checked by plotting the number of events in the PGC catalogue for every five years.

Table 2.2 Magnitude Completeness Years

Year	Magnitude
5.8	1899
5.3	1917
4.8	1940
4.3	1960
3.8	1965
3.0	1970
2.5	1985

Earthquakes for events within the subduction zone contains both shallow crustal earthquakes and deep in-slab earthquakes. The GEO and PUG zones in Models 1 and 2 and GSP zone in Model 3 represent the deep in-slab earthquakes, and all the other zones in Models 1 to 3 represent shallow crustal earthquakes. Therefore, the earthquake catalogue data were separated into shallow and deep categories, and the data with depths greater than 30 km were used to develop magnitude recurrence relationships for the deep zones PUG, GEO and GSP. These deep earthquake data were not used in the development of magnitude recurrence relationships for the shallow zones.

Exponential curve fitting was generally used for the source zones to develop the M-R relationships, and they were truncated at the maximum magnitude values selected for the zones.

Figure 2.12 shows the data and the M-R relationships for the shallow source zones SEBC12, SEBC1, SEBC2 and SBC'. Figure 2.13 shows the M-R relationships for the

shallow source zones CAS', CAS and SCM. Figure 2.14 shows the M-R relationships for the deep zones, GEO, PUG and GSP. Note that, for the source zones SEBC12, SEBC1 and CAS', where the project site is located, the M-R relationships were conservatively adjusted to follow the trend corresponding to lower magnitude ($M < 4.5$) earthquakes rather than full magnitude range up to the maximum magnitude. This adjustment resulted in much flatter slope (b) for the M-R relationship.

Table 2.3 lists the magnitude recurrence parameters used for all the source zones.

Table 2.3 Magnitude Recurrence Relationship Parameters Used for the Source Zones

Models	Source Zone	Shallow/Deep	β	b	N_5
Model 1	SEBC12	Shallow	1.620	0.70	0.1225
Model 2	SEBC1	Shallow	1.290	0.56	0.0954
Model 2	SEBC2	Shallow	1.959	0.85	0.0421
Model 3	CAS'	Shallow	1.460	0.63	0.2725
Model 3	SBC'	Shallow	2.350	1.02	0.0213
Models 1 and 2	NBC'	Shallow	1.578	0.69	0.0108
Models 1 and 2	NRMT'	Shallow	1.410	0.61	0.0126
Models 1 and 2	SFT'	Shallow	2.884	1.25	0.0127
Models 1 and 2	SCM	Shallow	1.851	0.80	0.0293
Models 1 and 2	CAS	Shallow	1.759	0.76	0.1638
Models 1 and 2	GEO	Deep	2.548	1.11	0.0024
Models 1 and 2	PUG	Deep	1.418	0.62	0.1395
Model 3	GSP	Deep	1.452	0.63	0.1345

Note: $\beta = b * \ln 10$ and N_5 is the activity rate expressed as number of events per year with magnitude greater than or equal to M5.

For source zones other than those listed in Table 2.3, the best estimate M-R parameters of the GSC model were used (Adams and Halchuck, 2003). These zones are located far away from the Afton sites and do not have any significant influence on the seismic hazard.

2.5.6 Maximum and Minimum Magnitudes

It is generally accepted that the low magnitude earthquakes below a certain threshold value are incapable of causing damage to engineered structures, and thus should not be considered in the seismic hazard assessment. This threshold magnitude is typically taken to be in the range of M4.5 to M5.0. The GSC selected a minimum magnitude of M4.75 for Western Canada in the development of hazard maps for 2005NBCC (Adams and Halchuck, 2003) and USGS selected M5.0 for Western United States in their 2008 update of the US Hazard Maps (Petersen et al., 2008).

The duration of the earthquake, or significant number of cycles in an earthquake motion, depends on the earthquake magnitude, and this parameter is strongly correlated to the liquefaction triggering potential of soils and displacements and strains in earth structures. Arango (1996) compiled data on earthquake magnitudes and distances, which have caused soil liquefaction, and which indicate that liquefaction has not been noted for earthquakes smaller than M5.2, even when the earthquakes have occurred at very short distances. In the current assessment, a minimum magnitude of M5.0 was selected.

The magnitude-recurrence relationships used to characterize the earthquake activity in each source zone were truncated at a pre-selected maximum magnitude. This maximum magnitude is considered to be the possible maximum magnitude of earthquake which may occur within the zone. In the current assessment, similar to the GSC's approach for the 2005 NBCC, three possible values called the best, upper bound and lower bound estimates were considered and they were assigned different weights. The maximum magnitude values for the Model 1, 2 and 3 source zones are listed in Table 2.4. A focal depth of 5 km was used conservatively for all the shallow zones and 30 km was used for the deep zones listed in Table 2.4.

Note that studies by Johnson et al. (1994) and Fenton et al (2006) concluded that M7.0 is the approximate maximum magnitude that occurs in unrifted stable continental cratons throughout the world, and would therefore be the lowest maximum magnitude that would be appropriate to include in seismic hazard models for any continental source zone. Adams and Atkinson (2003) suggested that the current best estimates of maximum moment magnitude are M7.0 for the stable continental shield, and M7.0 - M7.8 for zones of weakness within the continent. Ebel and Kafka (1991) noted that because earthquake activity in the region cannot be identified with specific faults and geologic features, geologic arguments cannot be used to further constrain the maximum magnitude that can be expected in a region.

Table 2.4 Minimum and Maximum Magnitudes and Depths used for the Source Zones

Models	Source Zone	Historical Maximum Magnitude, Mw	Minimum Magnitude Mw	Maximum Magnitude, Mw			Depth (km)
				Best	Lower	Upper	
Model 1	SEBC12	6.8	5	7.3	7.0	7.8	5
Model 2	SEBC1	6.8	5	7.3	7.0	7.8	5
Model 2	SEBC2	6.0	5	7.0	6.8	7.5	5
Model 3	CAS'	7.3	5	7.8	7.8	7.8	5
Model 3	SBC'	5.5	5	7.0	6.8	7.5	5
Models 1 and 2	NBC'	5.0	5	7.0	6.8	7.5	5
Models 1 and 2	NRMT'	5.5	5	7.0	6.8	7.5	5
Models 1 and 2	SFT'	4.7	5	7.0	6.8	7.5	5
Models 1 and 2	SCM	5.5	5	7.0	6.8	7.5	5
Models 1 and 2	CAS	6.4	5	7.5	7.3	7.8	5
Models 1 and 2	GEO	4.0	5	7.0	6.8	7.5	30
Models 1 and 2	PUG	6.8	5	7.3	7.0	7.8	30
Model 3	GSP	6.8	5	7.3	7.0	7.8	30

In the analyses, 60% of the weight was assigned to the best estimate magnitude, and 20% of the weight was assigned to each of the lower bound and upper bound estimates of magnitudes.

2.5.7 Ground Motion Prediction Equations

2.5.7.1 General

Ground motion prediction equations (GMPEs) describe the peak ground acceleration (PGA) or the spectral acceleration at a given period in terms of the earthquake magnitude and distance from the source. The GMPEs are typically developed by curve fitting recorded ground motion data to an assumed or derived functional form or by theoretical modelling. The theoretical model results are normally calibrated using recorded strong motion data.

2.5.7.2 Uncertainties in Ground Motions

In probabilistic seismic hazard analyses, the ground motion variability, which is usually described by a lognormal distribution, has significant impact on the computed hazard. The standard deviation of the log normal distribution is typically used to characterize the aleatory uncertainty or the random uncertainty in the ground motions. This random uncertainty is directly incorporated into hazard calculations by integrating over the entire distribution of the ground motion about the mean values. However, sometimes, the lognormal distribution is truncated at a specified number of standard deviations. For example, United States Geological Survey (USGS) truncated their ground motion distribution at three standard deviations when it developed the 2008 hazard maps for the Western United States (Petersen et al. 2008).

The value of the standard deviation and the number of standard deviations at which the distribution is truncated have significant control on the ground motions at lower probability level, such as the 10,000 year return period (Abrahamson, 2000, Bommer et al., 2004). Larger values of standard deviation and untruncated distribution result in larger computed ground motion, especially at low probabilities.

In this study, the standard deviation values recommended by the respective authors of the GMPEs were used and the ground motion distributions were not truncated. Note that Abrahamson (2006), who studied the truncation of lognormal distribution of ground motion relations, concluded that using an untruncated lognormal distribution in probabilistic seismic hazard analyses is appropriate for ground motion values that are below the physical limits of the underlying rock or soils.

The epistemic uncertainties in the GMPEs are one of the significant contributors to overall uncertainty in a seismic hazard analysis and are assessed by considering alternative sets of GMPEs. In this study, a suite of four GMPEs were used.

2.5.7.3 Ground Motion Prediction Equations (GMPEs)

The seismic hazard at the Afton project site primarily comes from relatively shallow crustal earthquakes. For these shallow crustal earthquakes, the four GMPEs listed below were used in the analyses:

- Abrahamson and Silva (2008);
- Boore and Atkinson (2008) ;
- Chiou and Youngs (2008); and
- Campbell and Bozorgnia (2008).

The above GMPEs were developed by a group of leading experts as part of the New Generation Attenuation (NGA) Model Project sponsored by Earthquake Engineering Research Institute (EERI) and based on strong motion database containing strong motion records from 173 earthquakes worldwide. It was accepted that all the NGA equations do a good job in capturing the accepted median relations (Abrahamson et al., 2008). These

NGA equations were used by USGS to develop their recent seismic hazard maps for the Western United States (Peterson et al., 2008).

The four GMPEs selected are defined for the horizontal component of the ground motion and use moment magnitude (M_w) as the magnitude variable. Boore and Atkinson (2008) uses the closest horizontal distance to surface projection of the rupture, R_{jb} , and the remaining three equations use the closest distance to the rupture, R_{rup} as the primary distance variable.

Note that the Boore, Joyner and Fumal (1997) relationship was used to develop seismic hazard maps for Western Canada by GSC in their 2005 NBCC maps (Adams and Halchuk, 2003). This relationship was validated against ground motion data in California, as of the early 1990s. A much larger strong-motion dataset (about five times) was used in the regression analyses of the NGA equations, than the dataset used in the validation of the Boore, Joyner and Fumal (1997).

Considering the uncertainties in the attenuation, equal weights (25%) were assigned for each of the four NGA attenuation equations in the analyses.

For the deep in-slab or intraplate earthquakes, the following three equations were used and they were assigned equal weights (33%) in the probabilistic hazard assessment.

- Atkinson and Boore (2003);
- Youngs et al. (1997); and
- Zhao et al. (2006).

The three equations listed above were also used by USGS in their development of the recent seismic hazard maps for United States (USGS, 2008). GSC used only the Youngs

et al. (1997) equation for the deep intraplate earthquakes in the development of their 2005 NBCC seismic hazard maps.

In the probabilistic seismic hazard assessment, the hazard due to the shallow crustal and deep in-slab earthquakes were summed up to estimate the total hazards at the site.

2.5.7.4 Reference Site Condition

The reference site condition for the analyses was taken as NEHRP Site Class B (Rock) and Site Class C (Soft Rock and Very dense Soil) boundary (NEHRP, 2003), which is defined as having a shear wave velocity of 760 m/s in the top 30 m; thus the ground motions were computed for this site condition. Unless otherwise noted, the ground motion results presented in this report correspond to this NEHRP Site Class B/C boundary condition. Note that the reference site condition used in the development of 2005 NBCC seismic hazard maps is Site Class C (Soft Rock or Very dense Soil), which is defined as having shear wave velocity in the range between 360 m/s and 760 m/s.

2.5.8 Logic Tree

Figure 2.15 shows the logic tree used in the analysis to handle the epistemic uncertainty and the corresponding weightings. The key uncertainties, which can significantly affect the seismic hazard estimate at the project site, namely the uncertainties in the source zone models and the GMPEs, were considered in the analysis. In addition, the uncertainty in the maximum magnitude was also considered. Table 2.5 summarizes the weights assigned for each branch in the logic tree.

Table 2.5 Logic Tree Branches and the Corresponding Weights

Parameter	Branch	Weight
Source Zone Models	Model 1	0.4
	Model 2	0.4
	Model 3	0.2
Maximum Magnitude	Best Estimate	0.6
	Upper Bound	0.2
	Lower Bound	0.2
Ground Motion Prediction Equation (GMPE) – Shallow Crustal Earthquakes	Abrahamson and Silva (2008) -AS08	0.25
	Boore and Atkinson (2008)- BA08	0.25
	Chiou and Youngs (2008) - CY08	0.25
	Campbell and Bozorgnia (2008) - CB08	0.25
Ground Motion Prediction Equation (GMPE) – Deep In-slab Earthquakes	Atkinson and Boore (2003)	0.33
	Youngs et al. (1997)	0.33
	Zhao et al. (2006)	0.33

2.5.9 Analyses and Results

Seismic hazard analyses were conducted for each combination of parameters shown in the logic tree using the computer program EZ-Frisk (Risk Engineering Inc., 2009), and the results were processed to obtain the mean, median, 16th and 84th percentile PGAs and spectral accelerations. Source zones within about 600 km radius from the site were included in the analyses.

Figure 2.16 shows the probability of annual exceedance versus PGA for the Afton site for the three alternative source models, each with the best estimate magnitude and for the four ground motion prediction equations. Figure 2.17 shows the mean, median, 16th and 84th percentile PGA hazard curves for the Afton site. Table 2.6 lists the mean, median, and 84th percentile PGAs for 2475, 5000 and 10000 year return periods for the Afton site. Note that the results correspond to NEHRP B/C site condition.

Table 2.6 Mean, Median and 84th Percentile PGAs for Afton Site for Various Return Periods

Return Period (yrs)	Peak Ground Acceleration, PGA (g)		
	Mean	Median	84 th Percentile
2,475	0.163	0.161	0.190
5,000	0.222	0.212	0.255
10,000	0.293	0.272	0.334

Figure 2.18 shows the mean, median, 16th and 84th percentile Uniform Hazard Response Spectra (UHRS) (with 5% damping) for the 10,000 year return period for the Afton site. Figure 2.19 shows the mean UHRS for various return periods for the Afton site. Table 2.7 lists the spectral ordinates corresponding to 10,000 year return period mean UHRS for the Afton site.

Table 2.7 PGAs and Spectral Accelerations for 10,000 Year Return Period for the Afton Site

Period (sec)	Spectral Acceleration (g) (5% damped)
PGA	0.293
0.1	0.619
0.2	0.708
0.5	0.383
1.0	0.188
2.0	0.079
3.0	0.046
4.0	0.031
5.0	0.025

Figures 2.20, 2.21 and 2.22 show the source zone contributions to the hazard for Alternative Models 1, 2 and 3, respectively. The results are for the mean values from the four GMPEs and for the best estimate maximum magnitude parameter.

Figure 2.23 shows the effect of maximum magnitude parameter on the PGA predicted by Model 2 and the Boore and Atkinson (2008) GMPE.

The key results from the analyses are summarized below:

- Figure 2.16 shows that the ground motions are sensitive to the source zone models and the range of PGAs predicted by the three models with the four GMPEs illustrates the uncertainty in the source zone model and GMPEs. In general, the 10,000 year return period PGAs predicted by Models 2 and 3 were similar and Model 1 were lower. The ranges of PGAs predicted by Models 1, 2 and 3 vary approximately between 0.20 g to 0.26 g, 0.27 g to 0.36 g, and 0.27 g to 0.36 g, respectively;
- As shown in Figures 2.20, 2.21 and 2.22, the 10,000 year return period PGA at the Afton site was dominated by contribution from the source zones SEBC12, SEBC1 and CAS⁷ in Models 1, 2 and 3, respectively; and
- As shown in Figure 2.23, the effect of maximum magnitude was not significant for the PGA at the Afton.

Consideration of earthquake size and distance is less straightforward in a probabilistic seismic hazard analysis than in a deterministic analysis. This is because in probabilistic analysis, the total hazard at a particular site is the result of possible occurrences of many different earthquakes of varying sizes and distances from the site. The hazard analysis results must be examined to identify the major contributors to the hazard at the ground motion level of interest. This is achieved through de-aggregation of the probabilistic seismic hazard. The representative earthquake scenario defined by dominant magnitude-distance may vary, depending not only on the probability level under consideration, but also on the period. For example, the dominant earthquake scenario for PGA is not necessarily the same as that for a one second period motion. A rigid and a flexible structure may be controlled by different earthquakes. Thus, there are different magnitude-distance combinations applicable to different ground motions. Selection of ground

motion time histories for dynamic deformation analyses and liquefaction assessment also requires a design earthquake be defined in terms of earthquake magnitude and distance. Results from de-aggregation analysis can be used to define such design earthquake scenarios.

Figures 2.24, 2.25 and 2.26 show the magnitude-distance de-aggregation analysis results for Models 1, 2 and 3, respectively, from analyses with the best estimate magnitude parameters. The results for PGA and one second period spectral acceleration are shown in these figures. The mean magnitude and mean distance corresponding to the 10,000 year PGA and 1 second spectral period are listed in Table 2.8.

Table 2.8 De-Aggregation Analyses Results for the 10,000 year Return Period

Source Zone Model	Maximum Magnitude Estimate	Parameter	Peak Ground Acceleration, PGA (g)	Spectral Acceleration at 1 sec, Sa (g)
Model 1	Best	Acceleration (g)	0.24	0.15
		Mean Magnitude	6.1	6.6
		Mean Distance (km)	16	41
Model 2	Best	Acceleration (g)	0.32	0.20
		Mean Magnitude	6.2	6.6
		Mean Distance (km)	14	26
Model 3	Best	Acceleration (g)	0.32	0.22
		Mean Magnitude	6.3	7.0
		Mean Distance (km)	15	41

Note that selection of the appropriate magnitude for the design earthquake will depend on several factors, including: (1) whether the magnitude is to be used in liquefaction assessment following a deterministic approach or probabilistic approach, or in the estimate of seismic displacements or deformation; (2) the natural period or dominant modal periods of the structures and how they will vary during the design earthquake; and (3) whether the dominant source contribution to the hazard is unimodal or bi-modal.

2.6 Deterministic Seismic Hazard Assessment

2.6.1 General

A deterministic seismic hazard assessment was conducted to assess the PGAs due to earthquakes associated with the rupture of known and unknown or blind faults in the vicinity of the project site.

A deterministic seismic hazard assessment typically involves the following four steps:

- identification and characterization of earthquake sources capable of producing significant ground motion at the site. Earthquakes at these sources are called the scenario earthquakes;
- determination of source-to-site distances, usually the shortest distance to the source;
- selection of controlling earthquake usually expressed in terms of earthquake magnitude and distance; and,
- estimation of ground motion due to the controlling earthquake using attenuation equation appropriate for the source and site.

Figure 2.27 schematically shows the four steps involved in a typical deterministic seismic hazard analysis.

2.6.2 Scenario Earthquakes

The seismic hazard at the Afton site can originate from either shallow crustal earthquakes on unknown or uncharacterized faults or from the deep in-slab earthquakes that may occur on the subducting Juan de Fuca plate. Thus, in the deterministic seismic hazard assessment, the following potential earthquake scenarios were considered:

- Scenario 1: Shallow crustal earthquakes at unknown or uncharacterized faults; and
- Scenario 2: Deep intraplate or in-slab type subduction earthquakes.

2.6.3 Ground Motion Prediction Equations (GMPEs)

The same suites of GMPEs that were used in the probabilistic analyses, were used in the deterministic analyses for both the shallow crustal and deep in-slab earthquakes, and the ground motions were estimated for the same reference Site Class conditions (i.e. NEHRP Site Class B and C boundary condition with $V_s=760$ m/s).

2.6.3.1 Scenario 1 Earthquakes – Shallow Crustal Earthquakes on Unknown and Uncharacterized Crustal Faults

Historical earthquake records from the PGC catalogue show that the maximum magnitudes of earthquakes, which occurred within 100 km, 200 km and 300 km from the Afton site are M4.5, M5.5 and M6.0, respectively. The seismic source zones containing Afton site in Models 1 to 3 were assigned maximum magnitudes of M7.8 in the probabilistic seismic hazard assessment. Thus, a quasi-probabilistic approach proposed by USBR (Committee on the Safety Criteria for Dams, 1985) was used to calculate the epicentral distance for random events in the vicinity of the project site with magnitude of M5.0, M6.5 and M7.8 and for a return period of 10,000 years. The magnitude recurrence parameter for the CAS' zone in Model 3 was used in this procedure to estimate the distance for the M5, M6.5 and M7.8 events. (Note that the zonal parameters for the CAS' zone in Model 3 predicted the shortest distances compared to the parameters for the SEBC12 and SEBC1 zones in Models 1 and 2, respectively. Therefore, the ground motion estimates using the parameters for CAS's zone will be conservative). The estimated distances for the M5, M6.5 and M7.8 events are 5 km, 13 km and 35 km, respectively.

Table 2.9 lists the calculated average PGAs from the median and median + σ estimates using the four GMPEs for the three earthquakes with magnitudes M5.0, M6.5 and M7.8.

Table 2.9 Peak Ground Accelerations (PGAs) from Deterministic Seismic Hazard Analyses for the Random Floating Earthquakes

Earthquake Magnitude, Mw	Distance (km)	Average ¹ Peak Ground Acceleration, PGA (g)	
		Median	Median+ σ
5.0	5	0.14	0.27
6.5	15	0.15	0.26
7.8	39	0.13	0.22

Note: ¹Average using the four GMPEs: Abrahamson and Silva (2008), Boore and Atkinson (2008), Chiou and Youngs (2008) and Campbell and Bozorgnia (2008)

2.6.3.2 Scenario 2 Earthquakes – Deep In-Slab Subduction Earthquake

In this scenario, the deep in-slab earthquakes that may occur on the subducting Juan de Fuca Plate were considered. Two deep earthquakes with magnitude M7.5, which is the likely maximum magnitude, at depths of 50 km and 100 km were considered in this scenario. The approximate shortest distances to these two deep earthquakes on the subducting Juan de Fuca plate are 285 km and 170 km, respectively.

Table 2.10 lists the calculated average PGAs from the median and median + σ estimates using the three GMPEs for the M7.5 earthquakes at depths of 50 km and 100 km, respectively.

Table 2.10 Peak Ground Accelerations (PGAs) from Deterministic Seismic Hazard Analyses for the Deep In-Slab Earthquakes

Earthquake Magnitude, Mw	Depth (km)	Distance (km)	Average ¹ Peak Ground Acceleration, PGA (g)	
			Median	Median+ σ
7.5	50	295	0.01	0.02
7.5	100	180	0.05	0.10

Note: ¹Average using the three GMPEs: Atkinson and Boore (2003), Youngs et al. (1997) and Zhao et al. (2006).

2.6.4 Conclusions from Deterministic Seismic Hazard Assessment

In the deterministic seismic hazard assessment, shallow crustal earthquakes on known faults and uncharacterized or unknown faults in the vicinity of the project site were considered. In addition, two deep in-slab earthquakes were also considered. The PGA estimates at the Afton site from the deterministic assessment were less than the 10,000 year return period PGA of 0.29 g estimated from the probabilistic assessment.

2.6.5 Local Faults Around the Site

Figure 2.3b shows the local faults in the vicinity of the Afton site. The information available to-date in the literature indicates that none of the local faults in the vicinity of the Afton site could be considered as active.

Two key faults near the site are the Coldwater Fault and the Cherry Creek Fault. The Coldwater fault is a steep, brittle normal fault associated with tertiary extension (Moore 1988), (Ewing 1980). The Coldwater fault system is a north-northeasterly trending Eocene (56 to 34 million years ago) structure with vertical offsets on a kilometre scale (Thorkelson, 1988). The Cherry Creek Fault is a normal fault that is associated with the Eocene extension. The Cherry Creek units are late Triassic to early Jurassic but the faulting occurred in the tertiary period (Kwong, 1987). The Cherry creek fault cross cuts the Coldwater fault and is therefore younger than the Coldwater fault.

Note that, as a part of this seismic hazard assessment, no field investigations such as field mapping or trenching were carried out and no remote sensing data were collected and reviewed. Such studies may be warranted in the future to identify any potentially active faults in the vicinity of the Afton site and to assess their impact on the recommended ground motions.

2.7 Ground Motions for the Reference Site Class Conditions

The PGA and spectral accelerations corresponding to 10,000 and 2,475 year return periods Uniform Hazard Response Spectra (UHRS-5% damped) for the Afton site are listed in Table 2.11. The ground motions listed in Table 2.11 are applicable for the reference site class B/C boundary condition (with shear wave velocity of 760 m/s) assumed in the analyses.

Table 2.11 UHRS for the Afton Dam Sites for the Reference NEHRP Site Class B/C Boundary Condition

Period (sec)	Spectral Acceleration (g) (5% Damped)	
	10,000 year Return Period	2,475 year Return Period
PGA	0.29	0.16
0.1	0.62	0.33
0.2	0.71	0.39
0.5	0.38	0.22
1.0	0.19	0.11
2.0	0.08	0.05
3.0	0.05	0.03
4.0	0.03	0.02
5.0	0.03	0.01

2.8 Recommended Ground Motions for Afton Dam Sites Corresponding to Site Class C/D Boundary Condition

Table 2.12 presents the NEHRP (2003) site class definitions for site class categories A to D. The probabilistic seismic hazard analyses were repeated to derive ground motion parameters for site class C (very dense soil) and D (stiff Soil) boundary with shear wave velocity of 360 m/s. In the seismic stability assessment of the dams at the Afton site, the foundation glacial till was conservatively taken as belonging to the site class C/D boundary category.

Table 2.12 NEHRP (2003) Site Class Definitions for Site Class Categories A to D

Site Class	Site Class Name and General Description	Site Class Definition
A	Hard rock	Hard rock with measured shear wave velocity, $V_s > 1500$ m/s
B	Rock	Rock with $760 \text{ m/s} < V_s \leq 1500 \text{ m/s}$
C	Very dense soil and soft rock	Very dense soil and soft rock with $360 \text{ m/s} < V_s \leq 760 \text{ m/s}$ or with either $N > 50$ or $S_u > 100 \text{ kPa}$
D	Stiff soil	Stiff soil with $180 \text{ m/s} < V_s \leq 360 \text{ m/s}$ or with either $15 \leq N \leq 50$ or $50 \text{ kPa} \leq S_u \leq 100 \text{ kPa}$

Note: V_s , N and S_u are average shear wave velocity, average standard penetration resistance and average undrained shear strength

The probabilistic seismic hazard analyses for the site class C/D boundary condition with shear wave velocity of 360 m/s were conducted to derive mean UHRS corresponding to 10,000 year and 2475 year return periods. Figure 2.28 shows the comparison of UHRS corresponding to site class B/C boundary with shear wave velocity of 760 m/s and site class C/D boundary with shear wave velocity of 360 m/s. As expected, the ground motions amplified and the amplification at the long period was greater than the amplification at short period and PGA.

The recommended PGA and spectral accelerations corresponding to 10,000 and 2,475 year return periods Uniform Hazard Response Spectra (UHRS-5% damped) for the Afton site are listed in Table 2.13. The recommended motions are applicable for site class C/D boundary with shear wave velocity of 360 m/s.

The representative earthquake magnitude for both 10,000 and 2,475 year return period recommended for the Afton site is $M_w 7.3$, which can be used in the liquefaction assessment and simplified deformation assessment of the dams, for example using Newmark (1965) sliding block approach.

Table 2.13 UHRS for the Afton Dam Sites for the NEHRP Site Class C/D Boundary Condition

Period (sec)	Spectral Acceleration (g) (5% Damped)	
	10,000 year Return Period	2,475 year Return Period
PGA	0.34	0.20
0.1	0.65	0.38
0.2	0.80	0.47
0.5	0.58	0.34
1.0	0.32	0.19
2.0	0.14	0.08
3.0	0.08	0.05
4.0	0.06	0.03
5.0	0.05	0.03

Note that the analyses presented in this report were performed for the West Dam site coordinates at the Afton site. The East Dam is located approximately 1 km east of West Dam. The difference over the distance of 1 km is expected to be negligible.

3. SEISMIC STABILITY ASSESSMENT

Stability of the Afton tailings dams was reviewed based on the new seismic ground motion parameters discussed in Section 2. This included an assessment of the liquefaction potential of the foundation soils, static and pseudo-static stability analysis using the limit-equilibrium method (Morgenstern-Price), and an estimation of earthquake induced permanent displacements using the Hynes-Griffin and Franklin (1984) method. This assessment is an update to that presented in "Closure Tailings Dam Stability Review" prepared by Klohn-Crippen Consultants Ltd. (KC) in 1998.

3.1 Liquefaction Assessment of Foundation Soils

Foundation soils at the East and West Dams generally consist of dense, well-graded silt-sand-gravel glacial till materials of varying depth overlying bedrock (KC 1998). Organic soils up to 2 ft thick were present in most areas prior to construction, and soft silt up to 15 ft deep was present in areas close to Hughes Lake. Construction records (Afton Operating Corporation, 1982 to 1996) indicate that organic and soft soils were removed from the foundation in accordance with the construction specifications (Klohn Leonoff (KL) 1979b).

Site investigations in the 1970s included test pits and drill holes in the foundations of both the East and West Dams (Ripley, Klohn & Leonoff 1973, KL 1976, KL 1978, KL 1979a). Standard Penetration Tests (SPT) were performed in 12 drill holes in the foundation of the West Dam in 1977 (KL 1978). The locations of these drill holes are shown on Klohn Leonoff drawing D-1836-12, included in Appendix II. This SPT data was used to perform a liquefaction assessment using the NCEER procedure described in Youd et al. 2001.

The results of the liquefaction assessment are shown in Figures 3.1 to 3.12. SPT blow counts recorded in the drill logs were corrected for field procedures and overburden stress, and converted to a clean sand equivalent $(N_1)_{60-cs}$. These values are plotted versus depth for all 12 drill holes. Soils with $(N_1)_{60-cs}$ values greater than 30 are too dense to liquefy (Youd et al. 2001) regardless of the applied ground motion. For $(N_1)_{60-cs}$ values less than 30, the cutoff for liquefiable soils (corresponding to a factor of safety against liquefaction of 1.0) is determined from parameters including depth below ground surface, total and effective overburden stress, earthquake magnitude, and peak ground acceleration. For the purposes of this calculation it was conservatively assumed that each drill hole represents a location outside of the dam footprint (i.e. no fill above ground surface), and that the long term water table is at the ground surface. An earthquake magnitude of M7.3 and a peak ground acceleration of 0.34 g were used, corresponding to the 10,000 year return period (see Section 2.8).

The results of the liquefaction assessment can be summarized as follows:

- 2 SPTs represent materials removed during construction;
- 28 SPTs indicated soils too dense to liquefy ($(N_1)_{60-cs} > 30$); and
- 1 SPT (from DH-2013, 18.6 m depth) indicated soil liquefiable under the applied ground motions and the assumed ground conditions.

The procedure used to calculate liquefaction resistance was developed for granular soils. A review of the one SPT indicating liquefiable soil shows that the material is primarily silt with some sand and some fine gravel. The blow count recorded in the field is 30, corresponding to a very stiff consistency. A very stiff, fine-grained soil would not develop significant positive pore water pressure under seismic loading. Therefore it is reasonable to conclude that liquefaction is not a concern for this material.

3.2 Stability Analysis

A cross-section at the maximum height of the West Dam was selected for stability analysis. The East Dam is buttressed by a waste rock dump on the downstream side, and has a lower earthquake design ground motion than the West Dam due to the different consequence of dam failure (BGC 2009). Therefore, the West Dam section is the critical stability section for the entire impoundment, and only this section was analyzed.

The stability analysis was performed using the limit equilibrium method (Morgenstern-Price) with the Slope/W component of the GeoStudio 2007 computer program. The section was analyzed for static, pseudo-static and post-liquefaction cases. A seismic coefficient of 0.17 g was adopted for the pseudo-static analysis, equivalent to 0.5 times the peak ground acceleration as recommended by Hynes-Griffin and Franklin (1984). For the post-liquefaction analysis, full liquefaction of the tailings was assumed.

The material properties used in the analysis are given in Table 3.1. These properties are almost the same as those used in the previous stability review (KC 1998), with a couple of exceptions:

- The friction angle of the tailings in the static and pseudo-static analyses was increased from 15° to 30°, which represents a more realistic strength for tailings.
- For the post-liquefaction analysis, a strength of $S_u / \sigma_v' = 0.15$ was assumed for the tailings.

Table 3.1 Summary of Material Properties Used in Stability Analysis

Material Type	Unit Weight (pcf)	Effective Friction Angle, ϕ' (degrees)	Post-Liquefaction Strength
Tailings	110	30	$S_u / \sigma_v' = 0.15$
Compacted Till	143 saturated, 140 unsaturated	32	-
Foundation Till	143	30	-
Filters	125	32	-
Rockfill	125	37	-

Note: σ_v' = effective vertical stress prior to earthquake event

For the stability analysis it was assumed that the phreatic surface in the dam corresponds to the conditions described in KC 1998, namely, a low phreatic surface near the base of the rockfill. Specifically, it was assumed that the phreatic surface drops quickly within the upstream till facing and continues downstream at 6 ft above the foundation. Recent readings in piezometers 2A and 3B confirm that the phreatic surface is near the base of the dam, as shown in Table 3.2.

Table 3.2 West Dam Piezometer Readings – March 24, 2011

Piezometer #	Depth to Water (ft)	Approximate Collar Elev. ¹ (ft)	Approximate Water Elev. (ft)	Approximate Foundation Elev. ² (ft)
2A	48.9	2148	2099	2119
3B	33.8	2251	2217	2211

- Notes: 1. Collar elevation estimated by adding 3 ft (assumed stickup height) to ground surface elevation (estimated from plan drawing of tailings area, Figure 5 in 1991 Tailings Dam Construction Report, Afton Operating Corporation)
2. Elevation of dam fill/foundation interface taken from drill logs 5201 and 5302 as reported in KL 1978.

The pond level in the impoundment was taken to be at El. 2298 ft to match the most recent survey in January 2010. The elevation of the tailings was assumed to be 3.7 m (12 ft) below the crest of the dam, as in the stability section analyzed in KC 1998.

The stability analysis section, critical slip surfaces, material properties, and calculated factors of safety are shown in Figure 3.13. The results of the stability analysis are summarized in Table 3.3.

Table 3.3 Stability Analysis Results - Calculated Factors of Safety

Failure Surface		Factor of Safety		
		Static	Pseudo-static (0.17 g)	Post-liquefaction (Tailings fully liquefied)
Downstream	A	1.61	1.07	-
	B	1.86	1.19	-
	C	2.36	1.35	2.32
Upstream	D	5.21	1.83	3.96

The static factors of safety meet the minimum target of 1.5 given in the Canadian Dam Association Dam Safety Guidelines (CDA 2007). The pseudo-static factors of safety meet the minimum target of 1.0 recommended by Hynes-Griffin and Franklin (1984). The post-liquefaction factors of safety are well above the minimum target of 1.0 required to prevent flow failure.

3.3 Seismic Deformation

Seismic deformation of the West Dam was estimated using the Hynes-Griffin and Franklin (1984) method. Pseudo-static analysis was applied to Section C of the stability model described above to calculate a yield acceleration (the pseudo-static acceleration at which the calculated factor of safety is 1.0). Section C was chosen because significant deformation of this section could potentially result in release of pond water and tailings. Hynes-Griffin and Franklin provide families of curves which can be used to estimate deformation based on the ratio of the yield acceleration to the peak ground acceleration. Curves are provided to calculate mean, mean plus 1 standard deviation and an upper bound. Generally the Hynes-Griffin and Franklin method is considered to overestimate deformations, as is appropriate for a screening tool. The results of the deformation

analysis are given in Table 3.4. Note that the upper bound deformation is considered statistically unlikely; the mean plus 1 standard deviation is a more realistic estimate.

Table 3.4 Seismic Deformation Estimate

Yield Acceleration	Deformation (cm) (PGA = 0.34 g)		
	Mean	Mean + 1 Std. Dev.	Upper Bound
0.28 g	< 10	11	67

For comparison, the height of freeboard is about 5.2 m, the thickness of the fine filter zone is about 2 m, and the upstream till facing is minimum 9 m thick. Therefore, the estimated seismic deformation of the dam is well within acceptable limits.

4. CONCLUSIONS

A seismic hazard assessment for the Afton tailings impoundment has been completed. The seismic ground motions derived from this assessment were used to complete a stability assessment that reached the following conclusions:

- The foundation soils were found to be unsusceptible to liquefaction;
- The dams meet factor of safety targets for slope stability under static and seismic loading; and
- The predicted seismic deformation is within acceptable limits.

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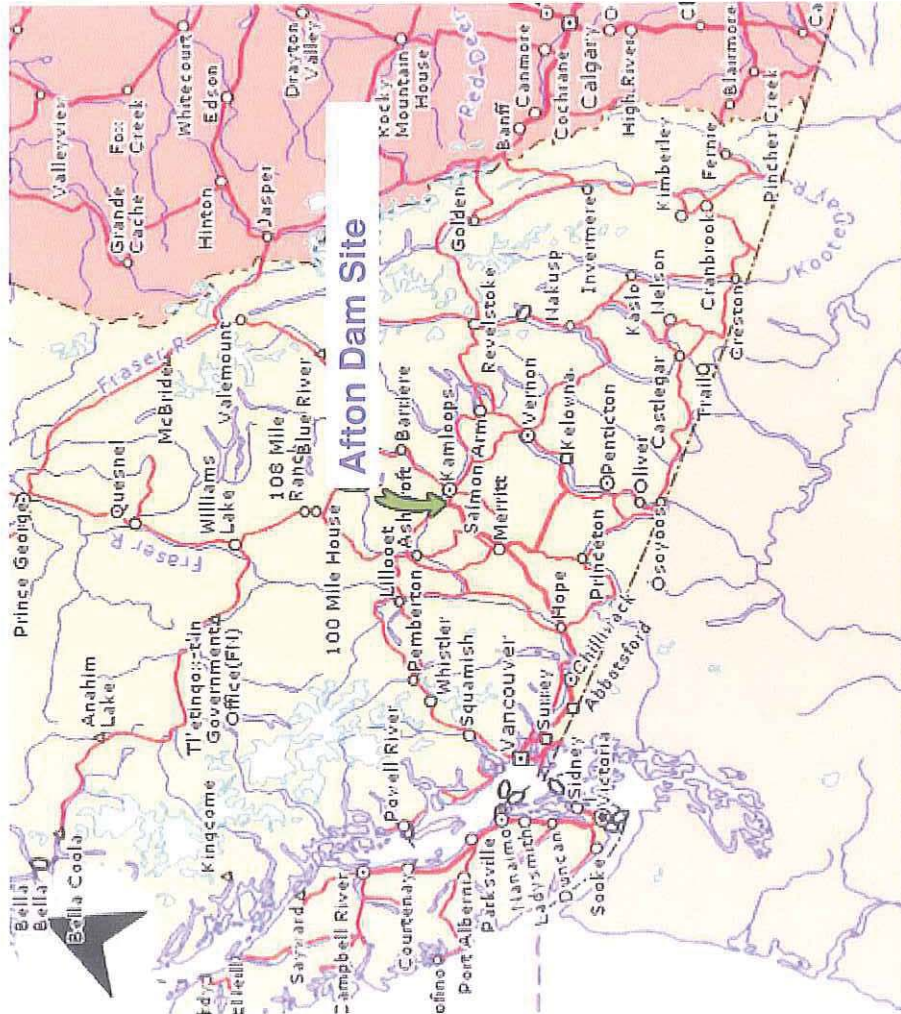
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FIGURES

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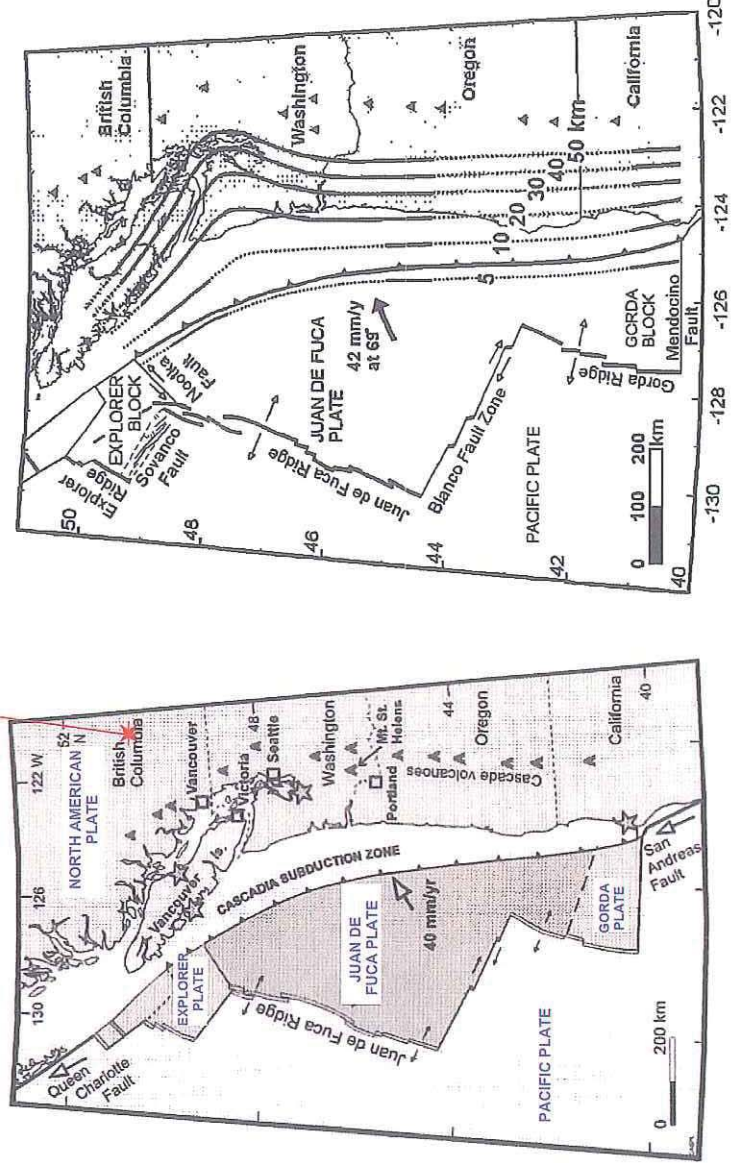
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PROJECT: AFTON TAILINGS IMPOUNDMENT SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
 TITLE: LOCATION MAP
 PROJECT NO: M05713A01
 FIGURE NO: 2.1


 Klohn Crippen Berger
AFTON OPERATING CORPORATION

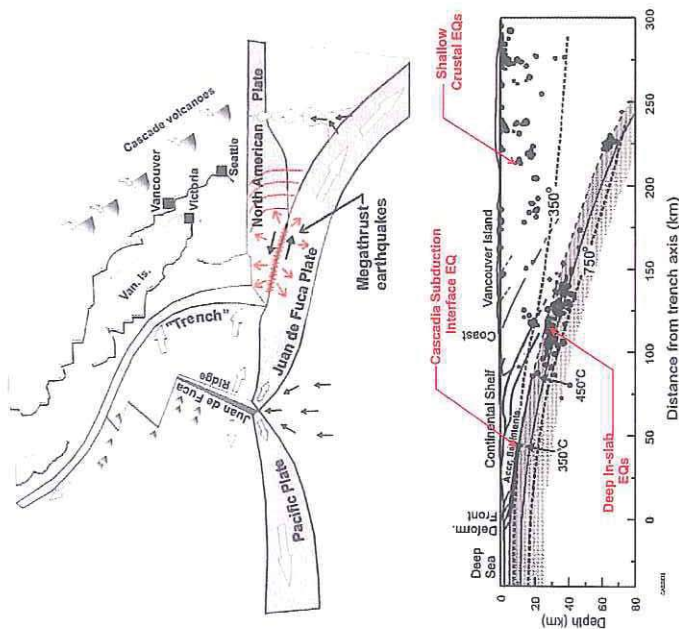
Note:
 Map taken from Atlas Canada

Approximate Location of Afton Site




2.2b) Plate Tectonic Configuration of Pacific Northwest and the location of HVC Site

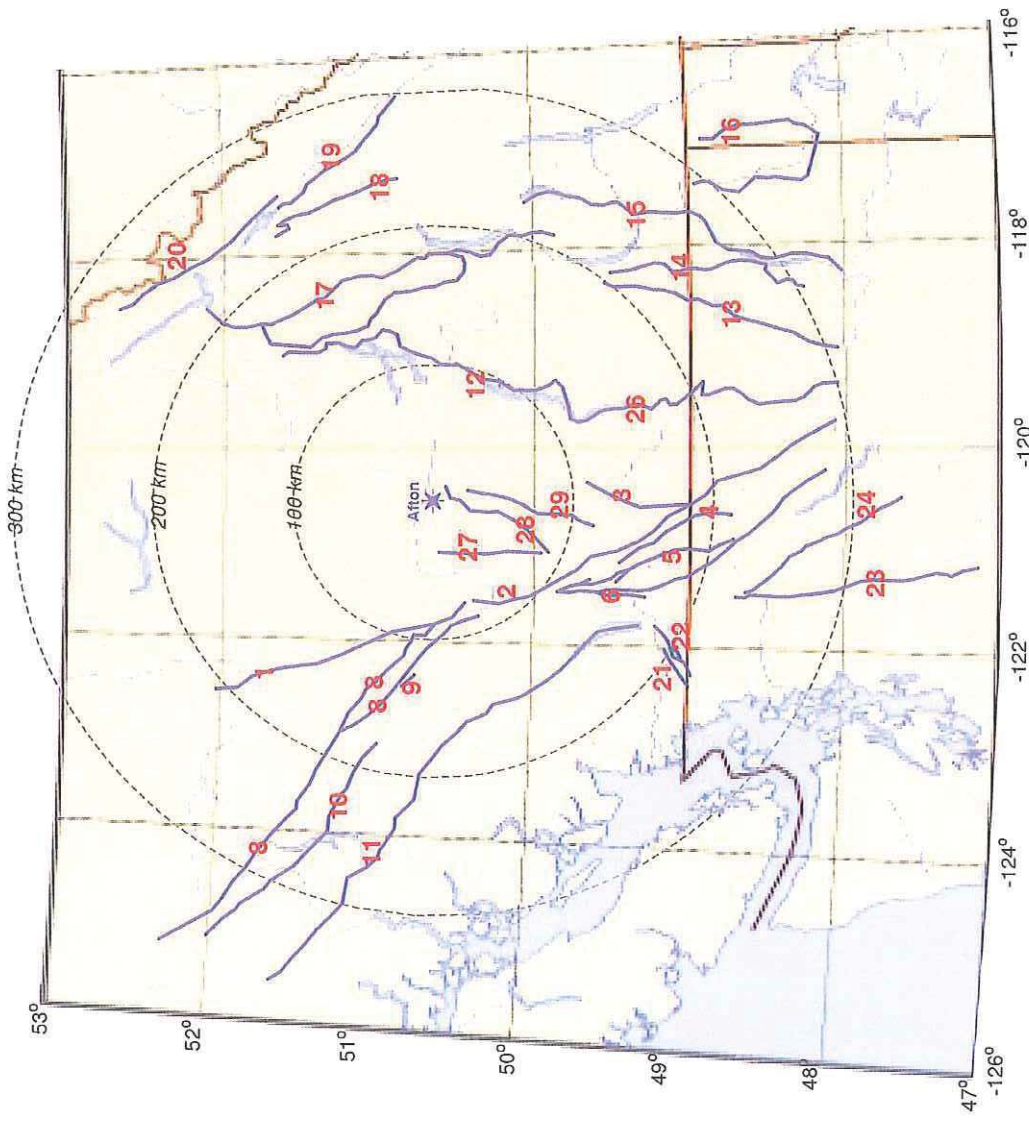
2.2c) Tectonic Configuration and Contours of Cascadia Subduction Zone



2.2a) Cascadia Subduction Zone and Sources of Earthquakes

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	PROJECT NO.: M09713A01 FIGURE NO.: 2.2



1. Fraser River Fault
2. Pasayten Fault
3. Boundary Fault
4. Chuwanten Fault
5. Hozameen Fault
6. Hope Yale Fault
7. Ross Lake Fault
8. Yalakom Fault System
9. Hell Creek fault
10. Tchaikazan Fault
11. Harrison Fault
12. Eagle River Fault
13. Granby Fault
14. Kettle River Fault
15. Slocan Lake Fault
16. Newport Fault
17. Columbia River Extension
18. Espalonde Fault
19. Purcell Thrust
20. Chatter Creek
21. Sumas Mtn Fault
22. Vedder Fault
23. Straight Creek Fault
24. Entjat Fault
25. Leavenworth Fault2
26. Okanagan Valley Fault
27. Lornex Fault
28. Coldwater Fault
29. Quilchena Fault

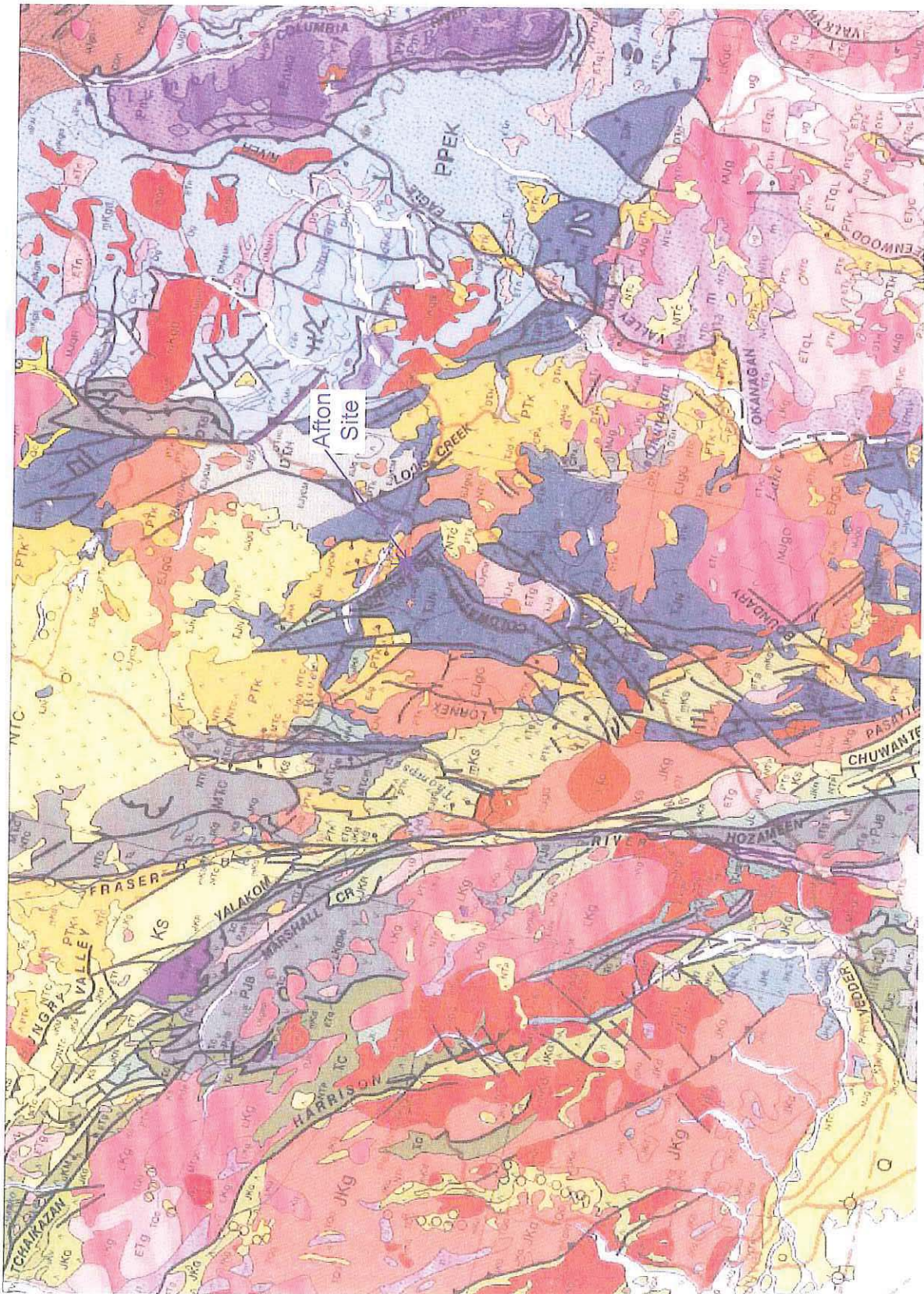
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Notes:
 1. Only the regional faults surrounding Afton site are shown.
 2. Fault traces are approximate
 3. Background map taken from NR Canada and is approximate


 Klohn Crippen Berger
 CONSULTANTS

AFTON OPERATING CORPORATION

PROJECT	AFTON TAILINGS IMPOUNDMENT
TITLE	SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
PROJECT NO.	M09713A01
FIGURE NO.	2.3a



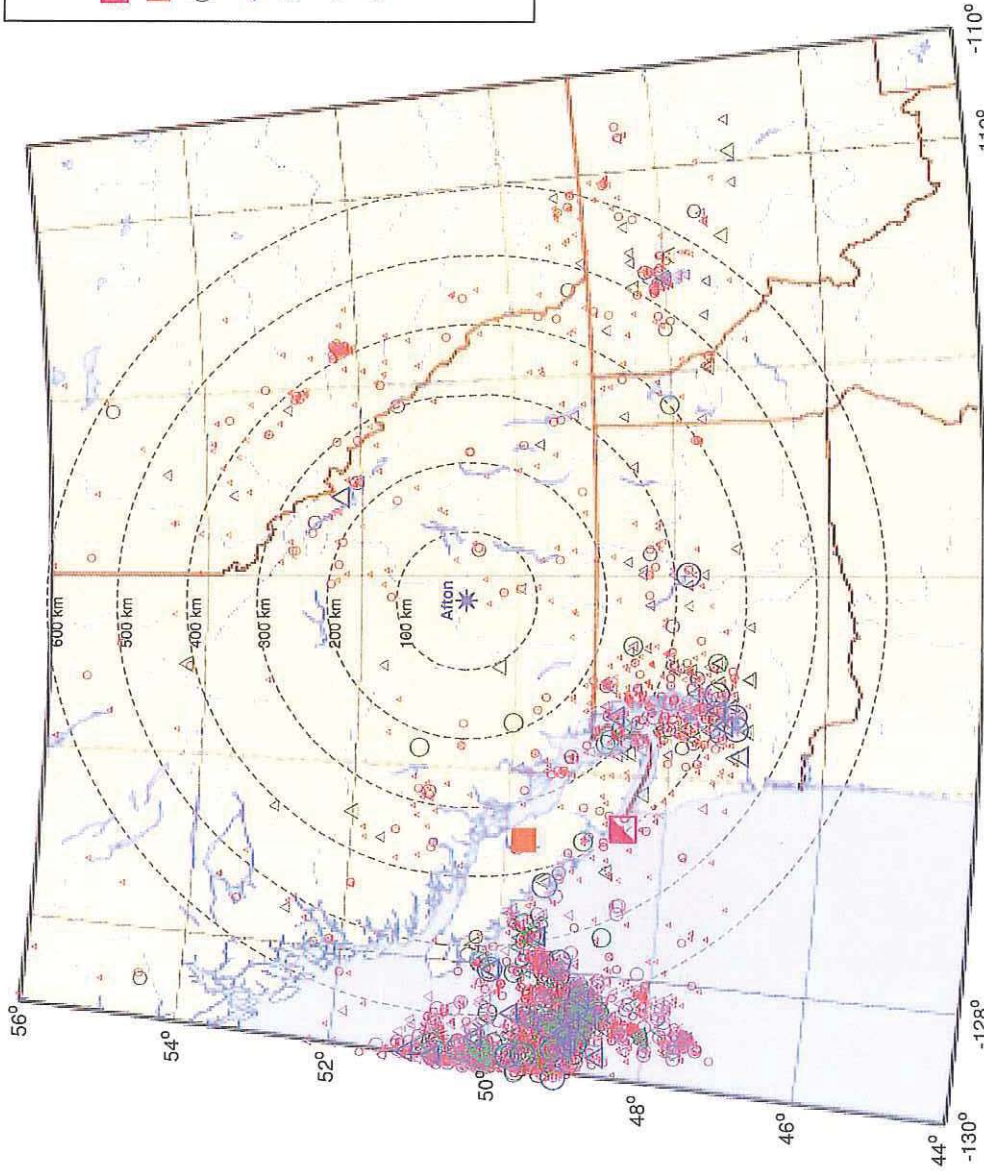
PROJECT: AFTON TAILINGS IMPOUNDMENT
 SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
 TITLE: TECTONIC ASSEMBLAGE MAP SHOWING LOCAL FAULTS AROUND AFTON SITE
 PROJECT NO: M09713A01
 FIGURE NO: 2.3b


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Note:
 1. Dark lines show faults
 2. Afton mine site is approximate
 2. Tectonic assemblage map obtained from Geological Survey of Canada (Wheeler and McFeely, 1991)

Magnitude Range			
			M9
			M7.3
			M6.5-M6.9
			M6.0-M6.4
			M5.5-M5.9
			M5.0-M5.4
			M4.5-M4.9
			M4.0-M4.4
			M3.5-M3.9
			M3.0-M3.4



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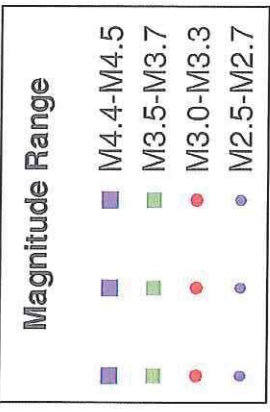
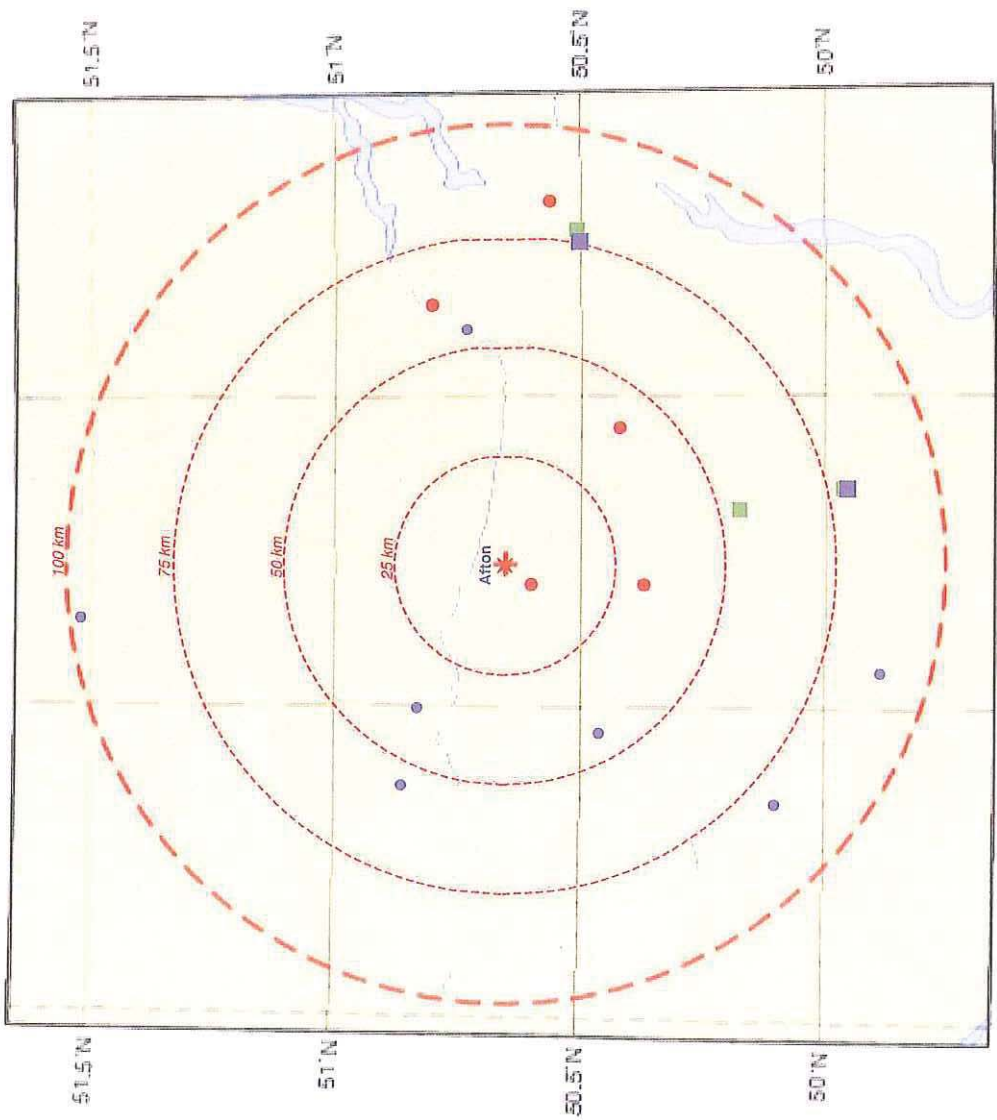


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PROJECT	AFTON TAILINGS IMPOUNDMENT
TITLE	SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
PROJECT NO.	M09713A01
FIGURE NO.	2.4

Note:
1. Map taken from NR Canada and is approximate



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AFTON OPERATING CORPORATION

PROJECT	AFTON TAILINGS IMPOUNDMENT SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
TITLE	EARTHQUAKES WITH MAGNITUDE, M > 2.5
PROJECT No	M09713A01
FIGURE No	2.5

Note:
1. Map taken from NR Canada and is approximate

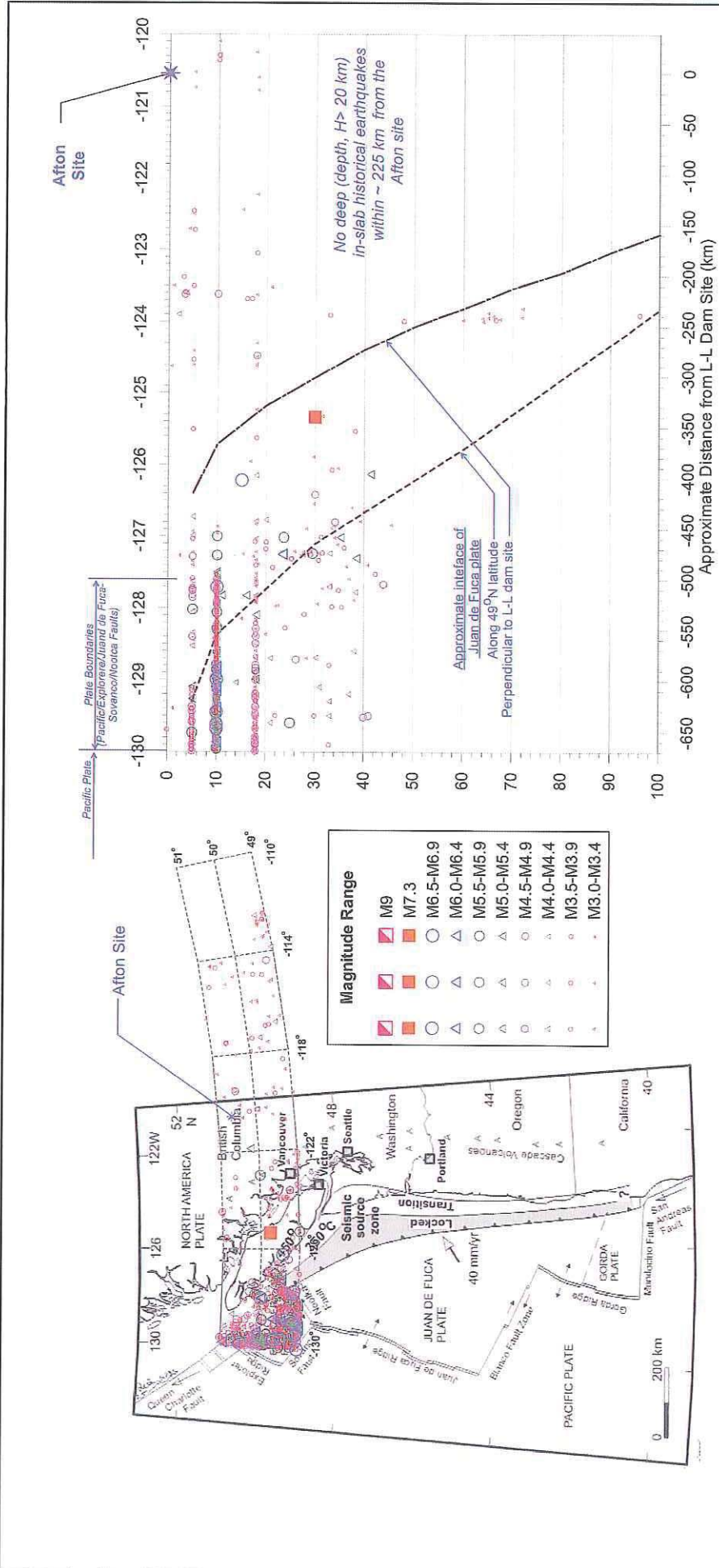


Figure 2.6a - Plan View

- Notes:
1. Earthquakes with magnitude $M > 3$ from 1700 to April 30, 2010 are shown.
 2. Epicentre data taken from PGC Catalog.
 3. Distances from project site are approximate.
 4. Map taken from Earthquakes Canada website.
 5. Interface between subducting Juan de Fuca and North American plate estimated from Fluck et al. (1997)

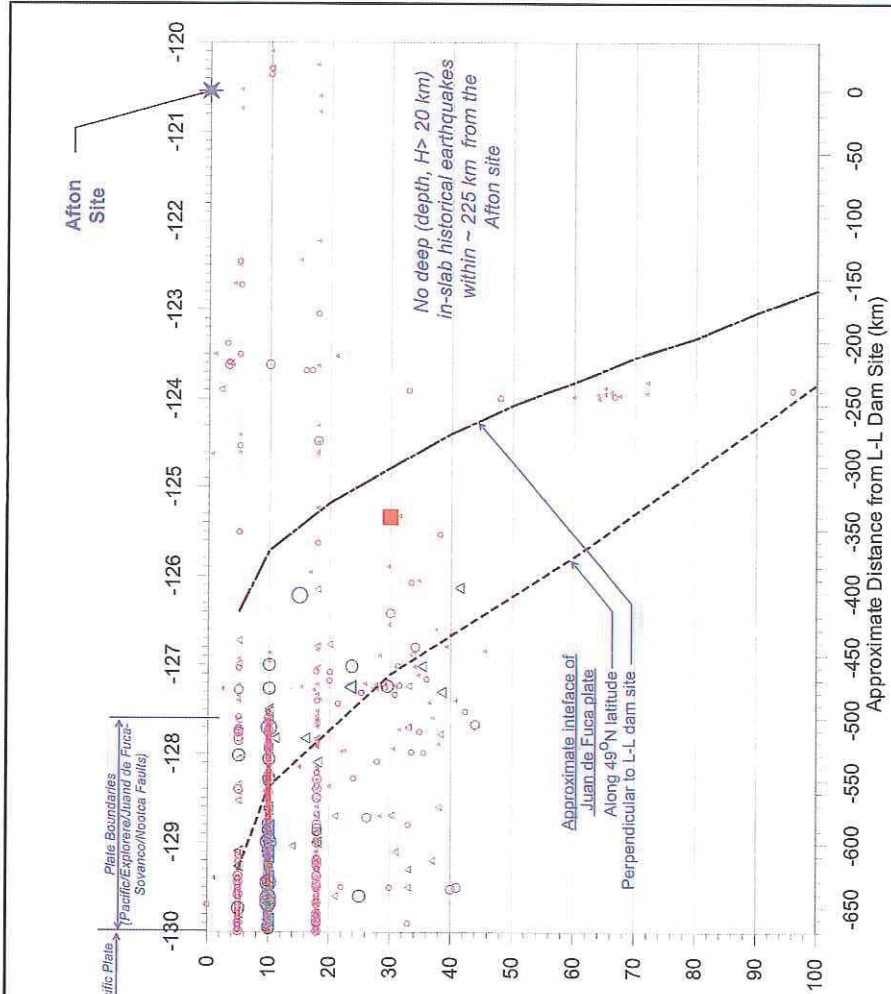
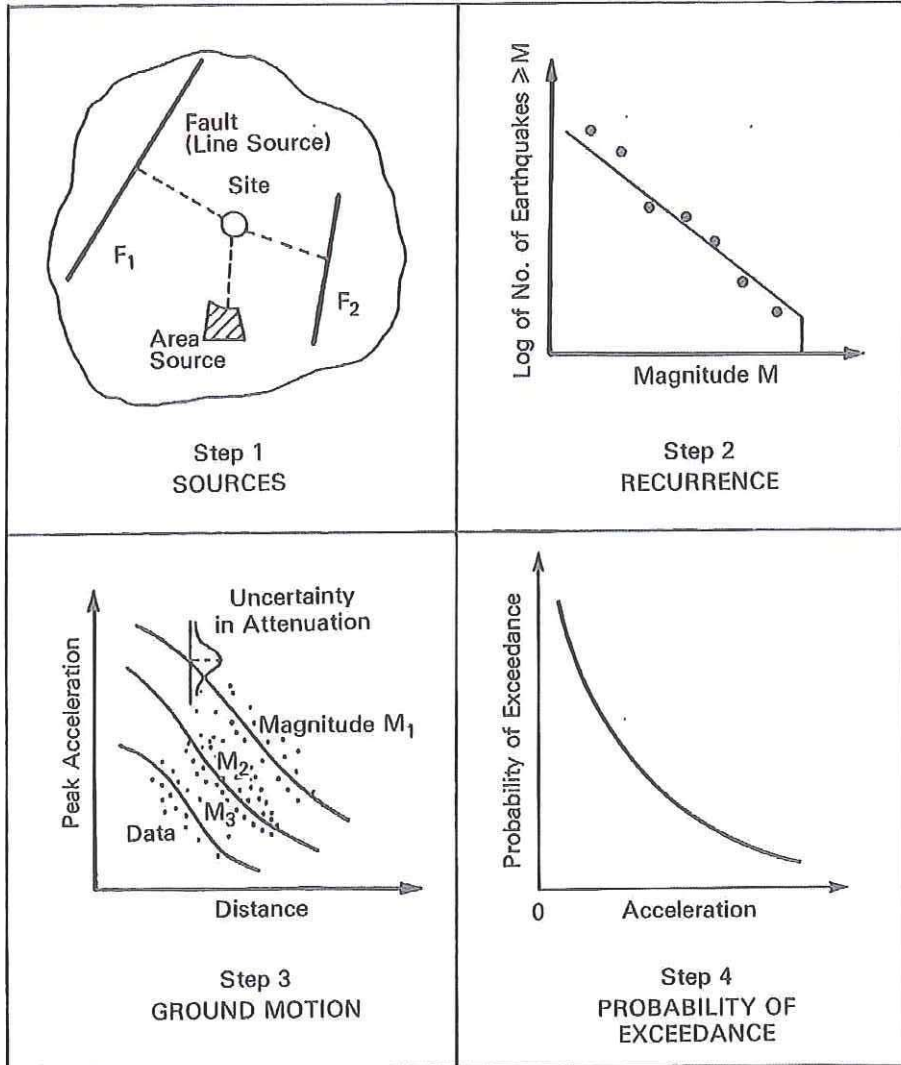


Figure 2.6b - Cross Section

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	<p>PROJECT NO: M09713A01</p> <p>FIGURE NO: 2.6</p>	



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PROJECT

AFTON TAILINGS IMPOUNDMENT
SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

TITLE

STEPS IN PROBABILISTIC SEISMIC HAZARD ANALYSIS

CLIENT:

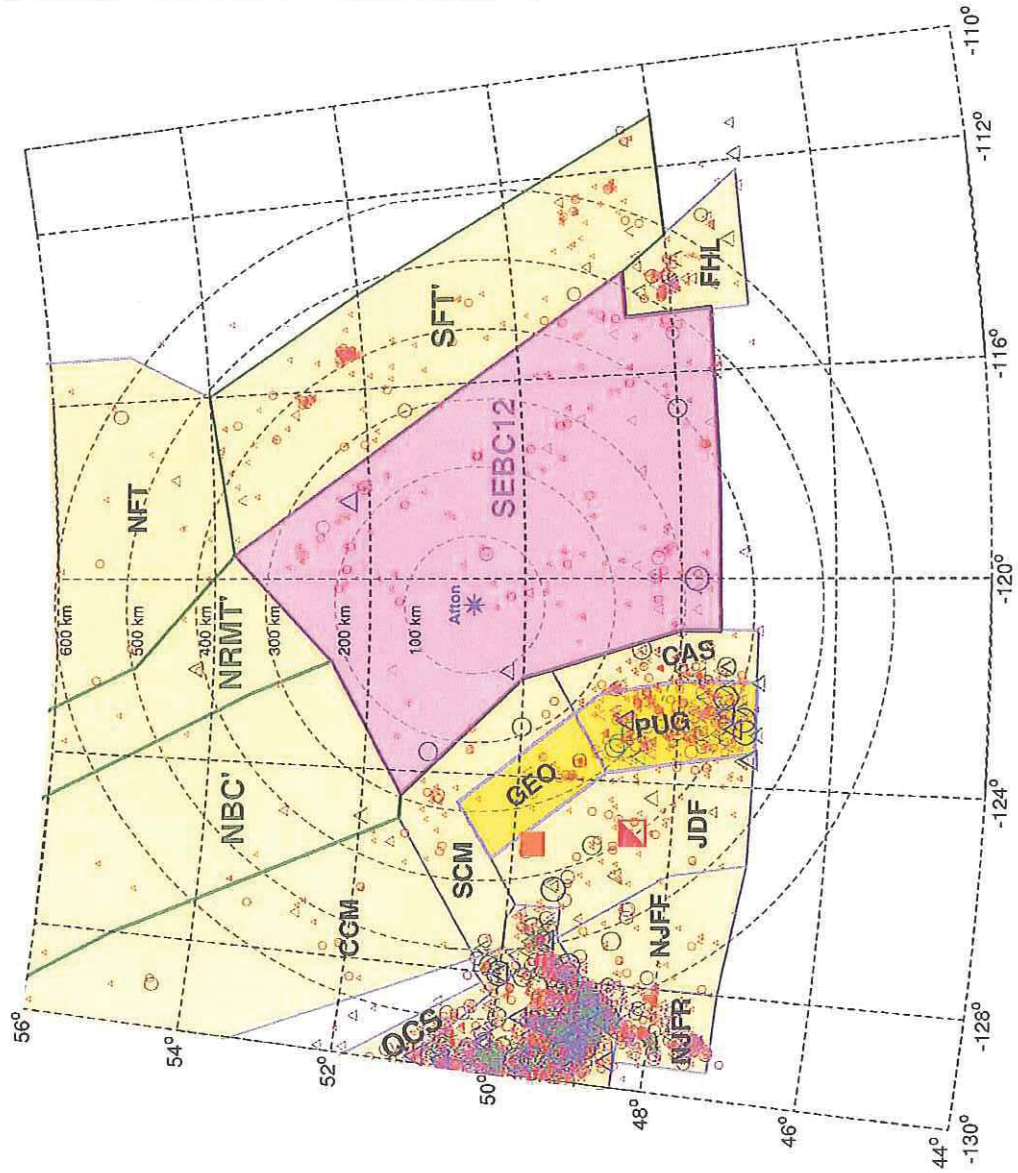
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PROJECT No.

M09713A01

FIGURE No.

2.7



ALTERNATIVE MODEL 1

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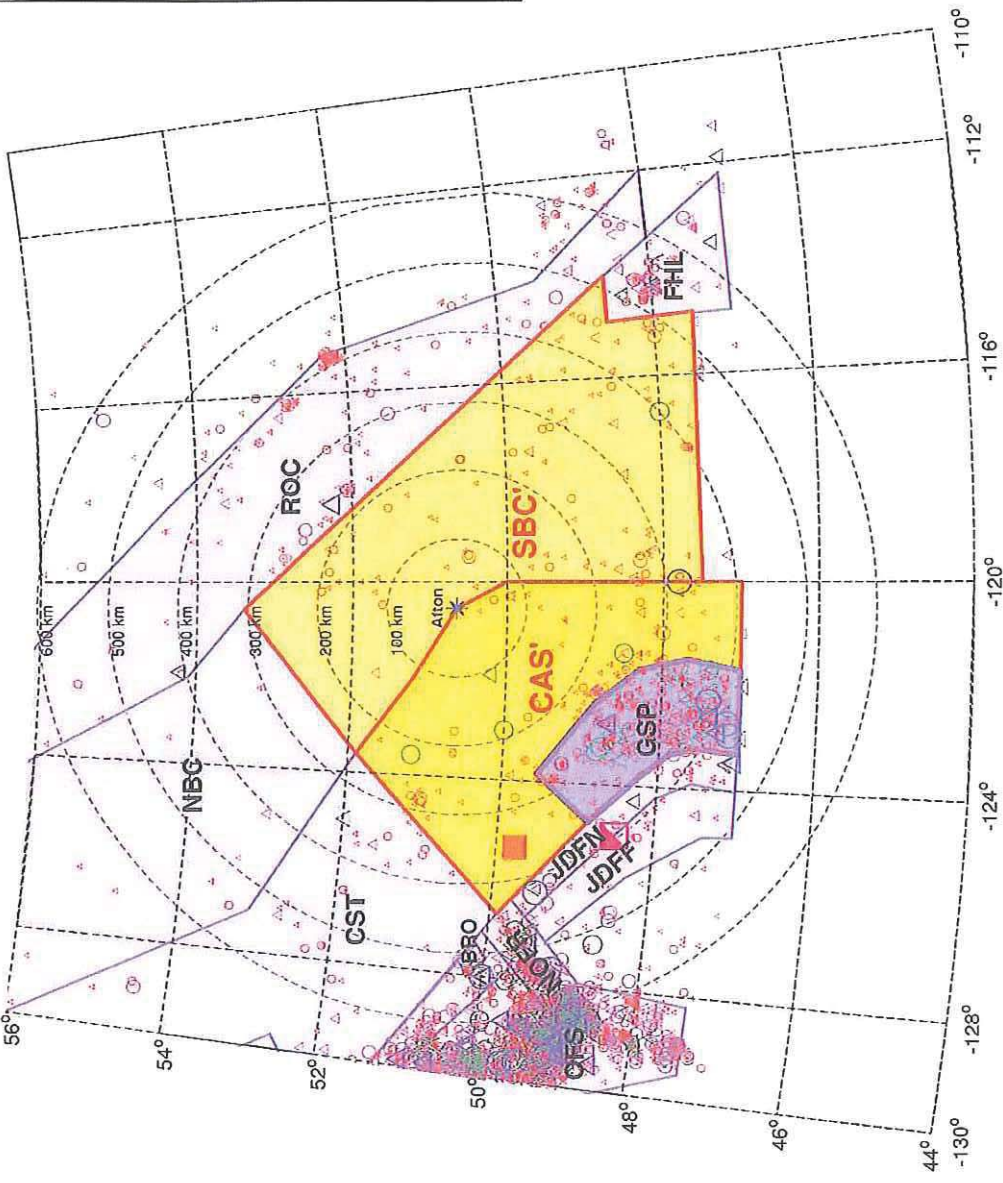
PROJECT: AFTON TAILINGS IMPOUNDMENT SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

TITLE: ALTERNATIVE SOURCE ZONE MODEL 1

FIGURE NO: M09713A01

FIGURE NO: 2.8

CLIENT: AFTON OPERATING CORPORATION



Magnitude Range	
▣	M9
▣	M7.3
○	M6.5-M6.9
△	M6.0-M6.4
○	M5.5-M5.9
△	M5.0-M5.4
○	M4.5-M4.9
△	M4.0-M4.4
○	M3.5-M3.9
△	M3.0-M3.4

Model 3 New Zone Boundary
 Unmodified GSC-R Model Zone Boundary

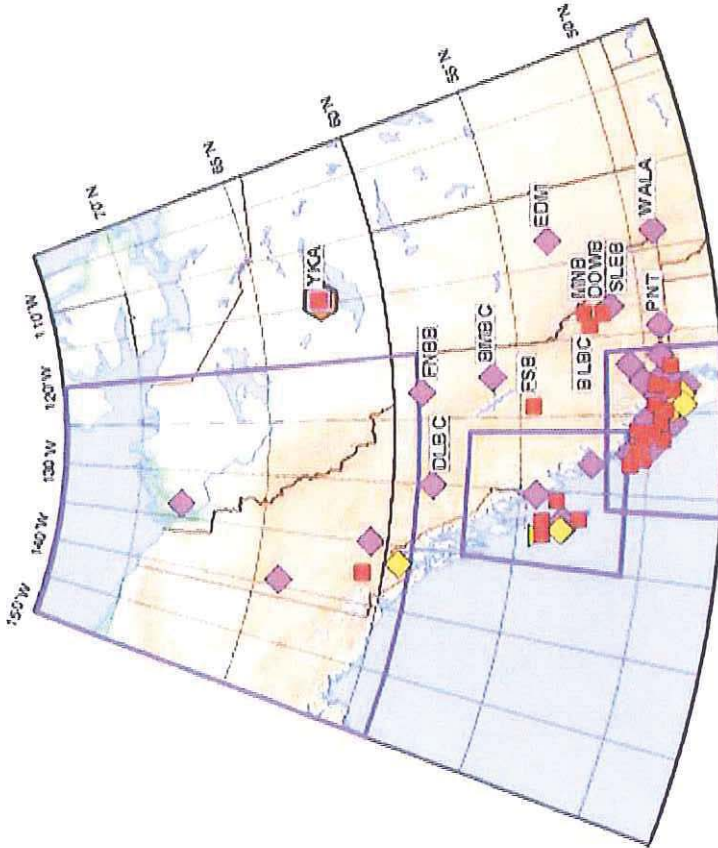
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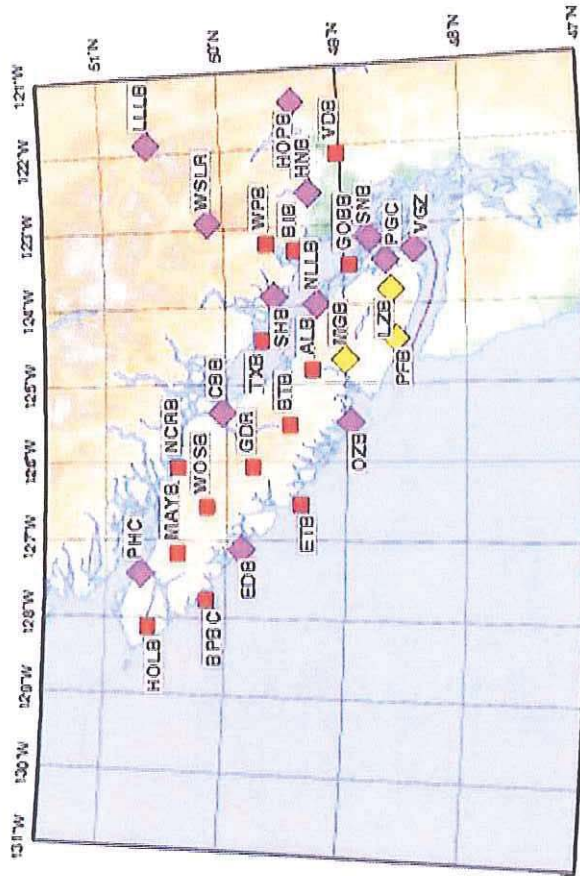
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PROJECT NO: M09713A01	FIGURE NO: 2.10



Earthquakes Canada
Seismes Canada

Canadian National Seismograph Network

- ◆ Very Broadband
- Extremely Short Period
- ◆ Broadband
- ◆ High Broadband
- Extremely Short Period Digital
- ▲ Regional Analogue



Earthquakes Canada
Seismes Canada

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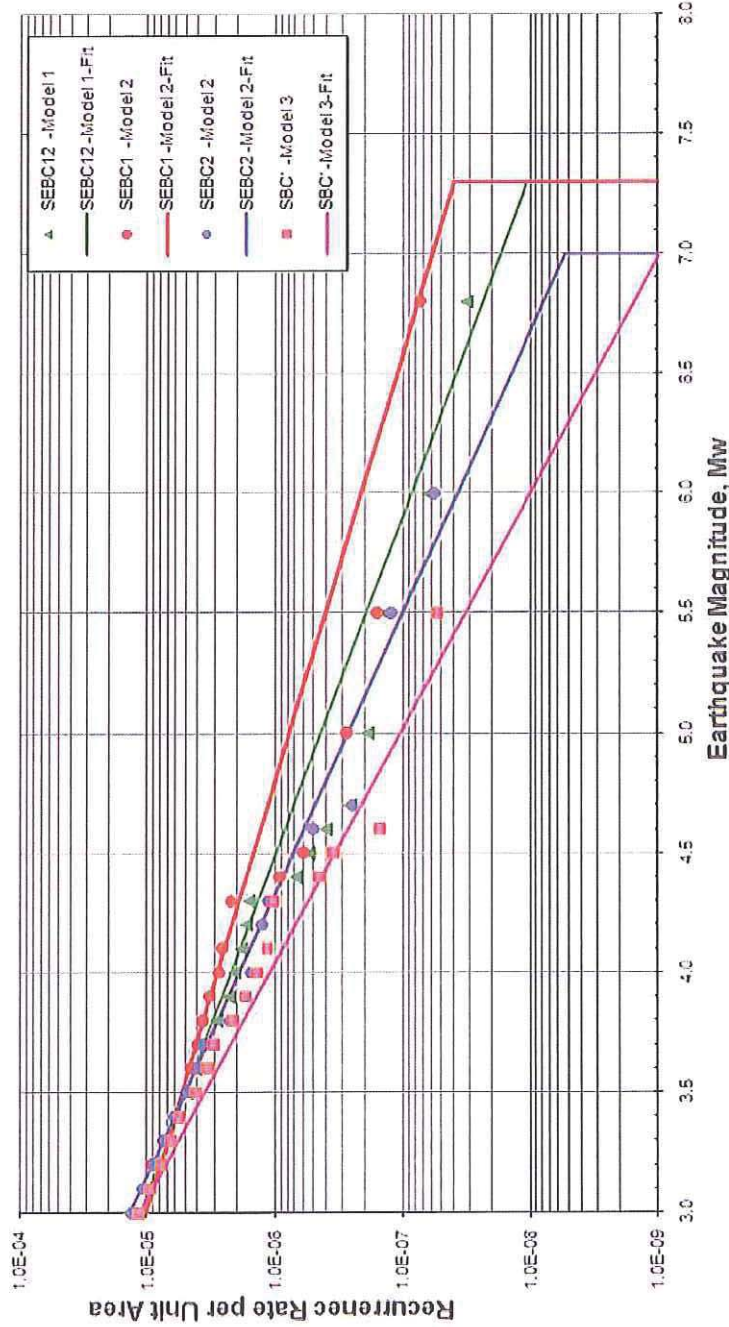
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PROJECT: AFTON TAILINGS IMPOUNDMENT
TITLE: SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

PROJECT NO: M09713A01
FIGURE NO: 2.11

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FOR ZONES LOCATED EAST OF SUBDUCTION ZONE



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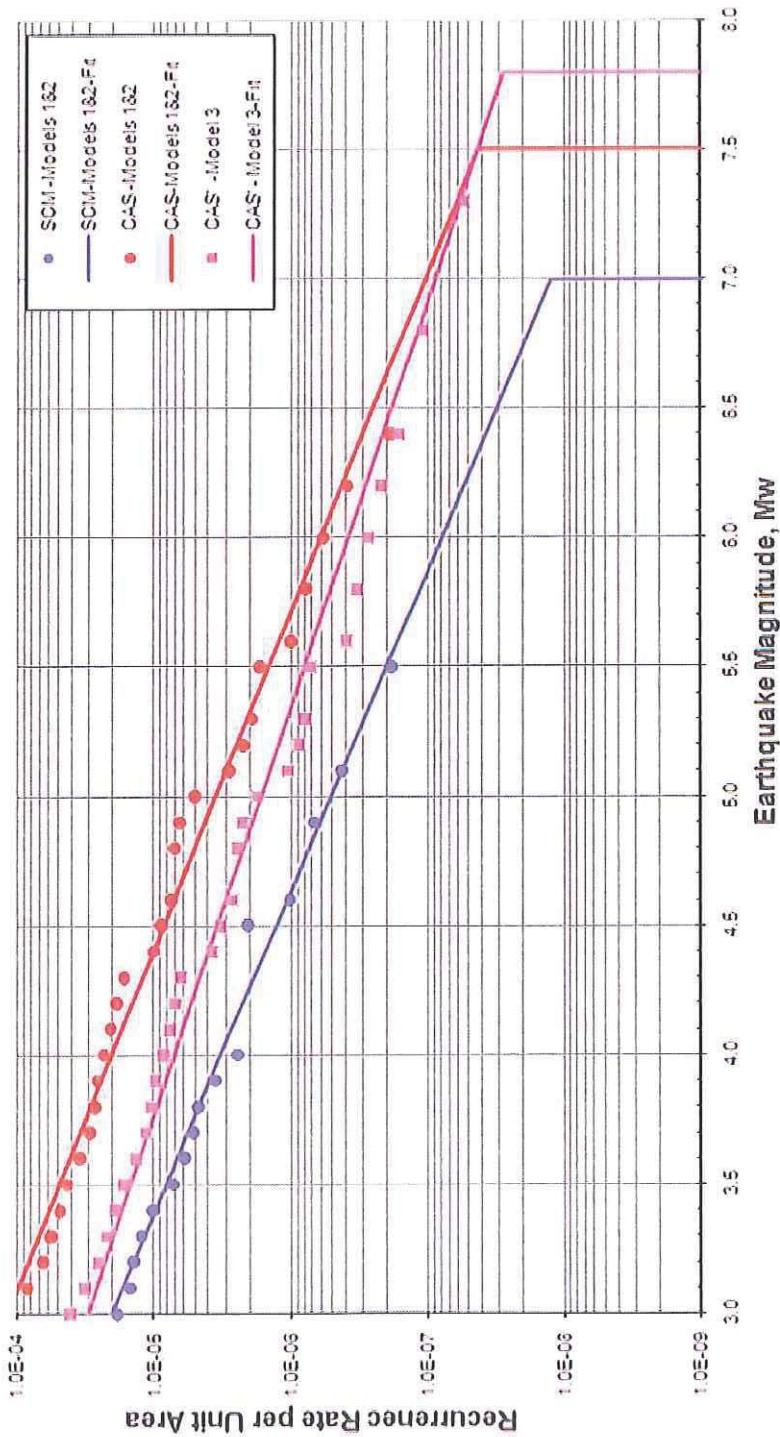
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SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

TITLE: MAGNITUDE RECURRENCE RELATIONSHIPS FOR ZONES EAST OF SUBDUCTION ZONE

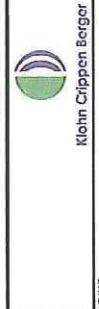
PROJECT#: M09713A01
FIGURE #: 2.12

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FOR ZONES WITH THE SUBDUCTION ZONE



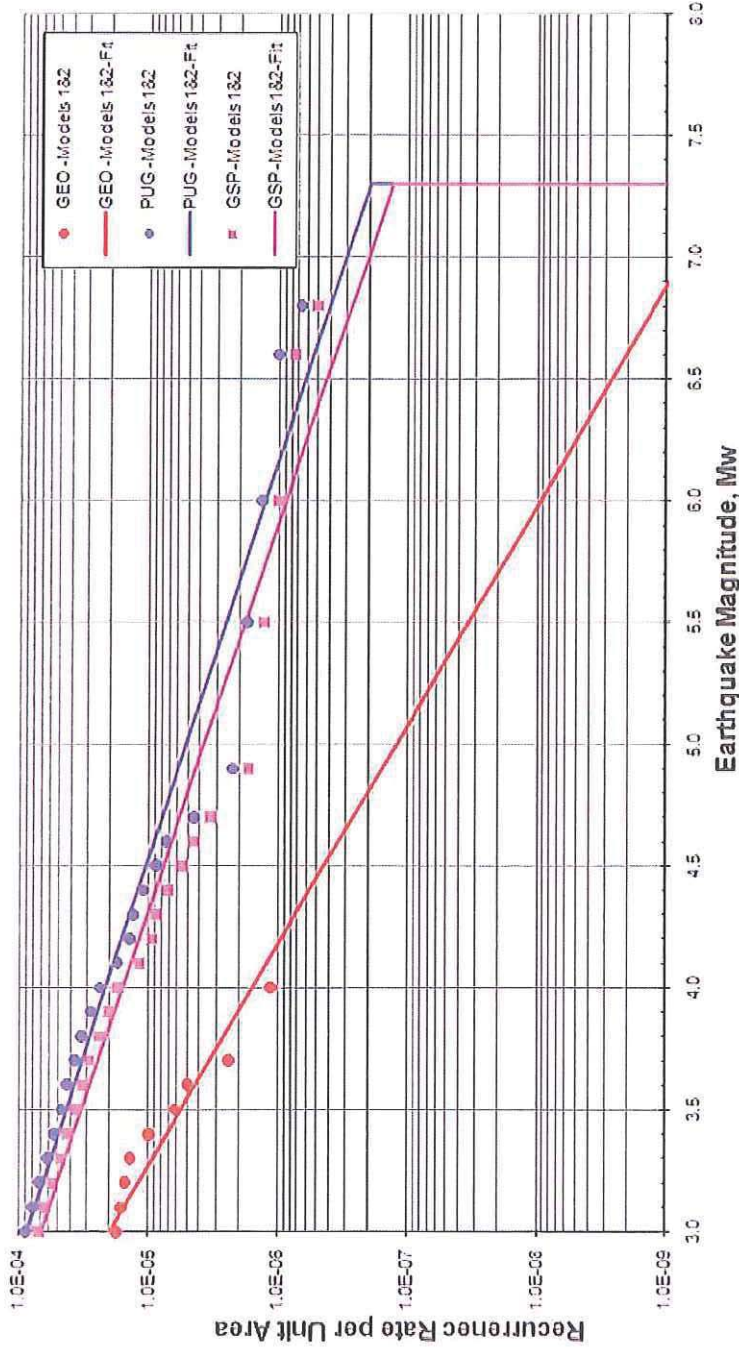
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PROJECT	AFTON TAILINGS IMPOUNDMENT
TITLE	SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
FIGURE NO.	MAGNITUDE RECURRENCE RELATIONSHIPS FOR ZONES WITH THE SUBDUCTION ZONE
PRODUCT NO.	M09713A01
FIGURE NO.	2.13

FOR ZONES REPRESENTING DEEP IN-SLAB EARTHQUAKES



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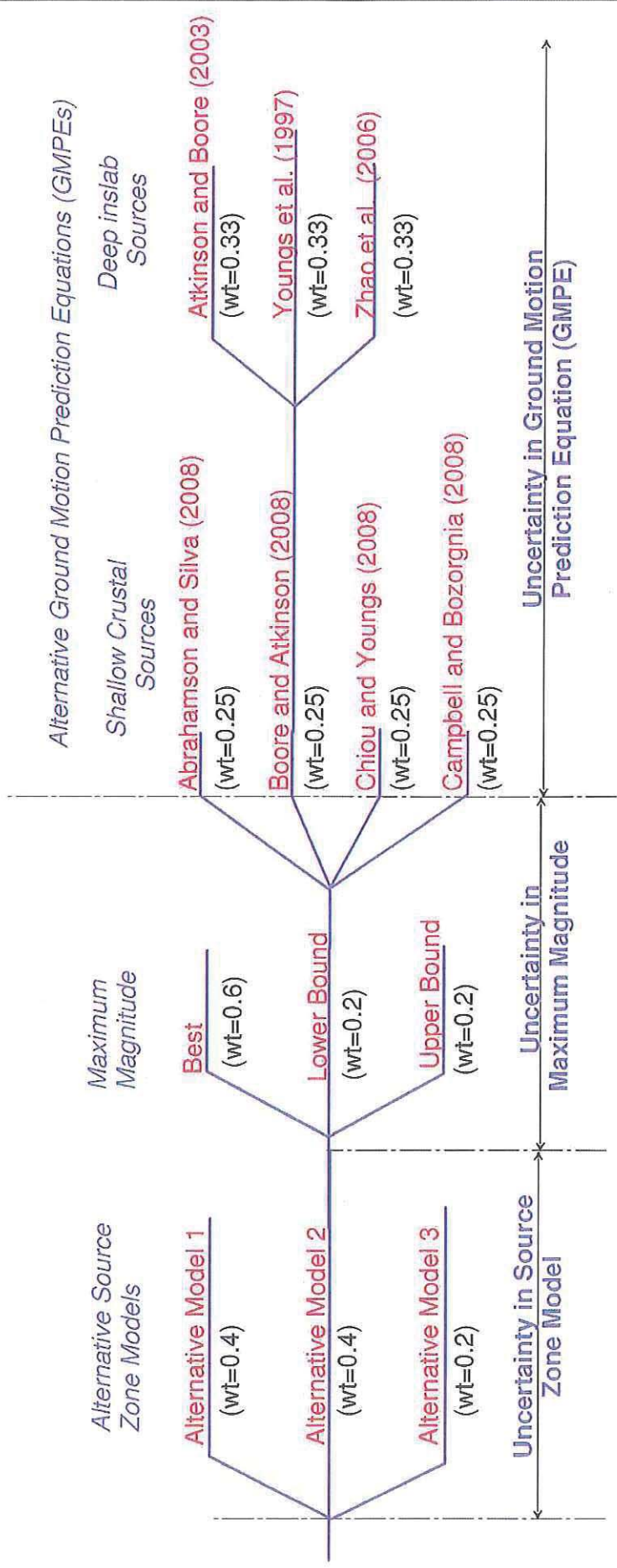
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 TITLE: SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

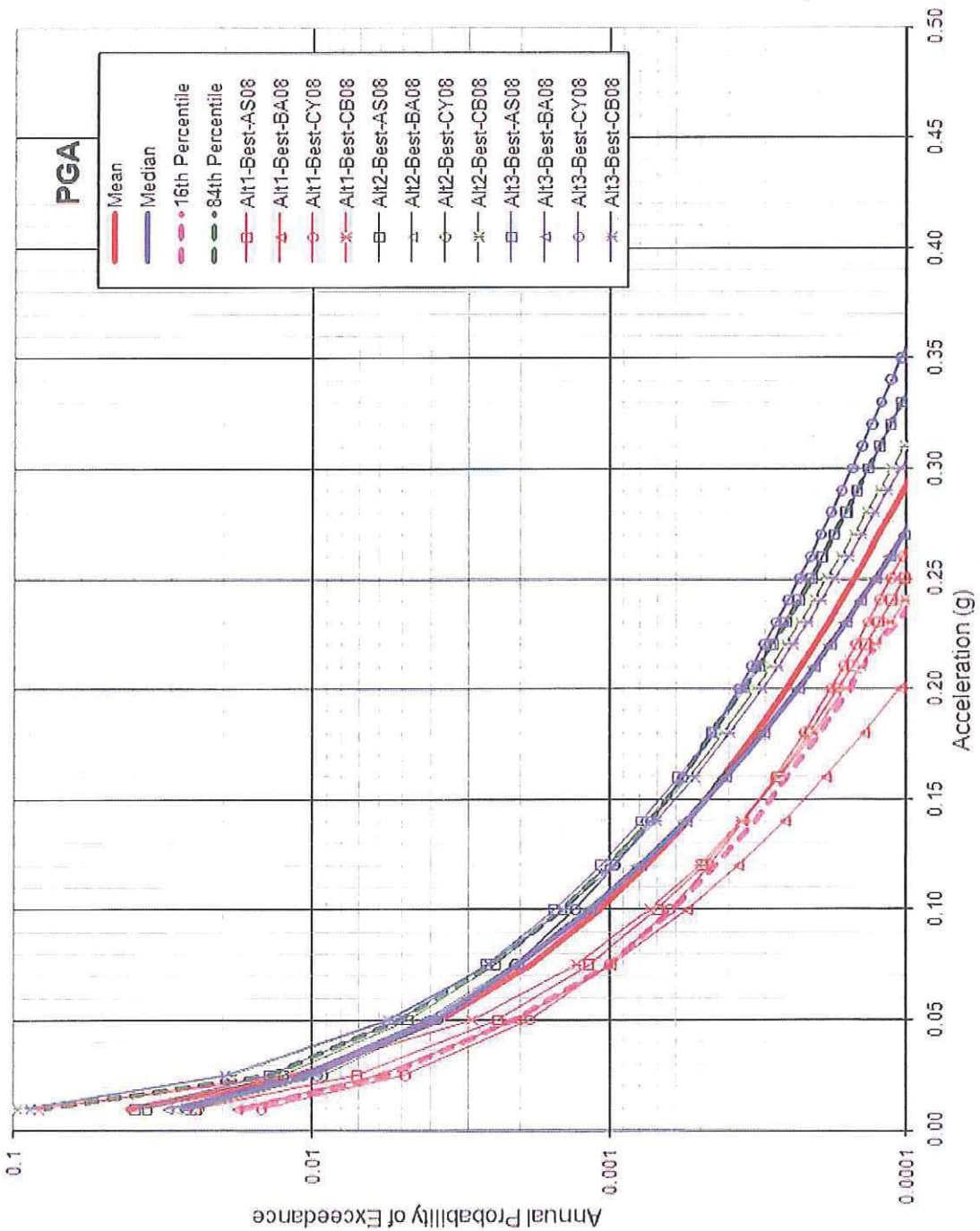
MAGNITUDE RECURRENCE RELATIONSHIPS
 FOR ZONES REPRESENTING DEEP IN-SLAB EARTHQUAKES

PROJECT NO: M09713A01
 FIGURE NO: 2.14



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		TITLE LOGIC TREE USED IN PROBABILISTIC SEISMIC HAZARD ANALYSIS
		PROJECT NO. M09719A01
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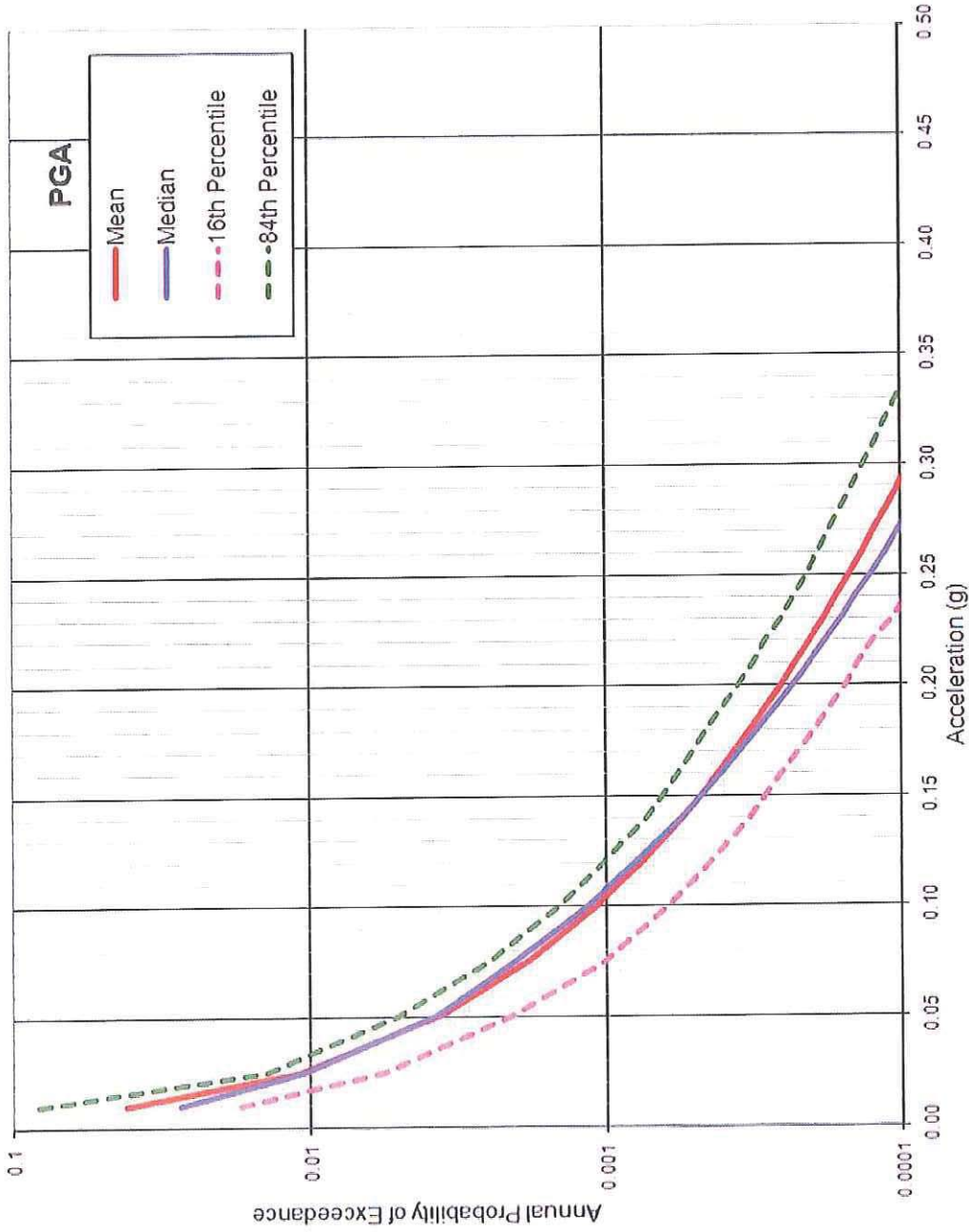


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PROJECT	AFTON TAILINGS IMPOUNDMENT
TITLE	SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
CLIENT	PROBABILITY OF ANNUAL EXCEEDANCE VERSUS PGA FOR THE AFTON SITE
PROJECT NO.	M09713A01
FIGURE NO.	2.16

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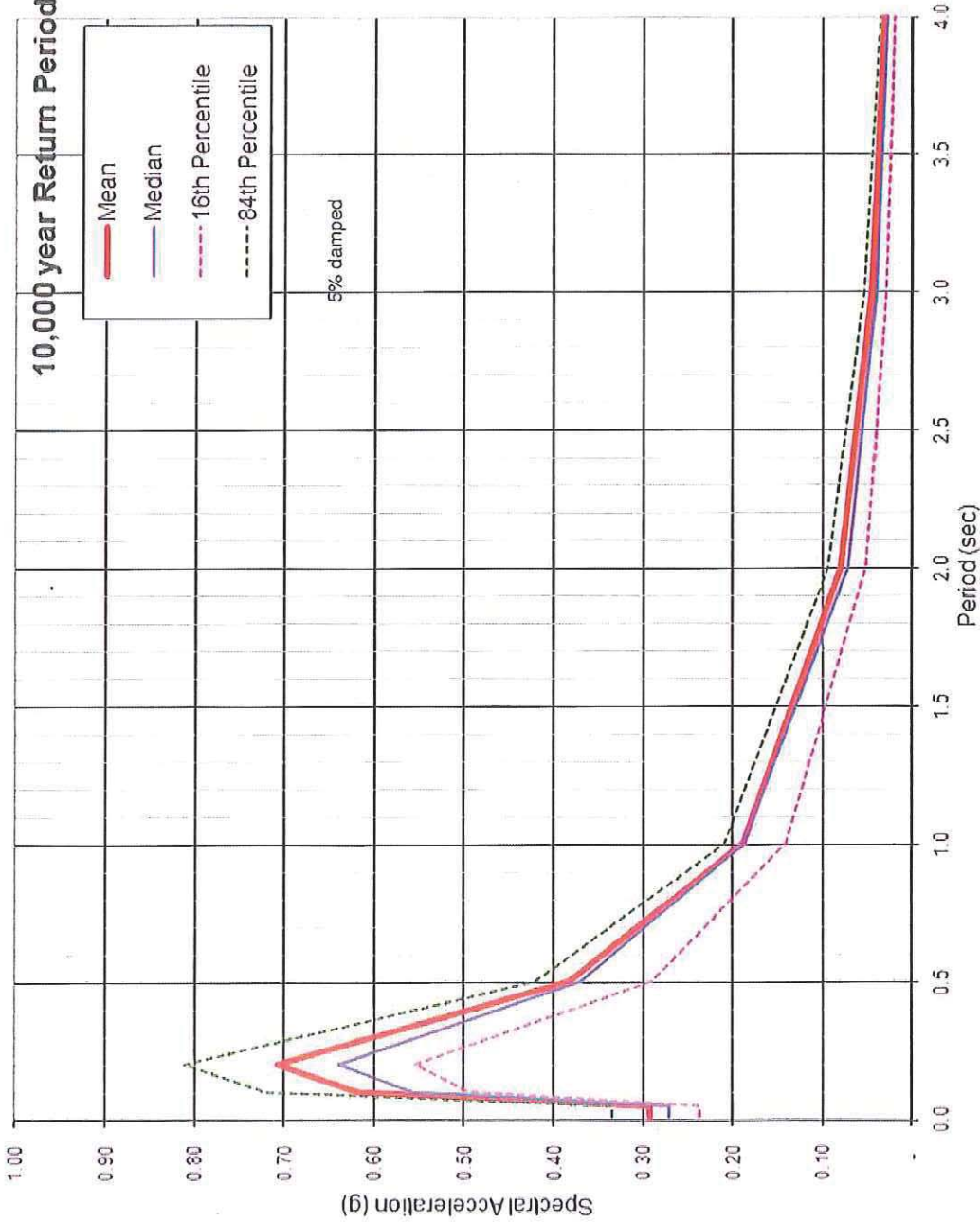
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SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

TITLE: MEAN, MEDIAN, 16TH AND 84TH PERCENTILE
PGA HAZARD CURVES FOR THE AFTON SITE

PROJECT NO: M09713A01

FIGURE NO: 2.17

10,000 year Return Period



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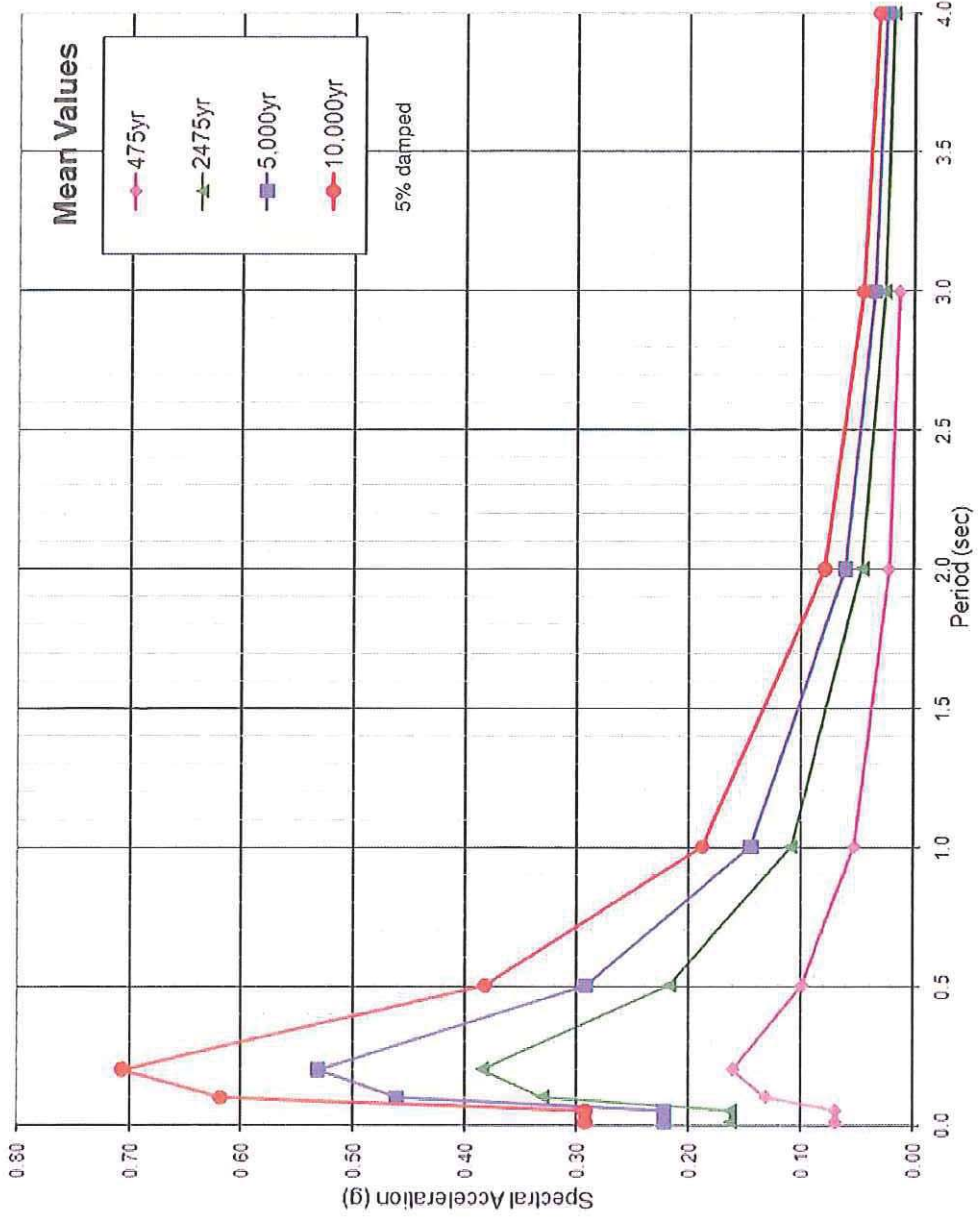


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 SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
 TITLE: MEAN, MEDIAN, 16TH AND 84TH PERCENTILE 10,000 YEAR RETURN PERIOD UHRS FOR THE AFTON SITE

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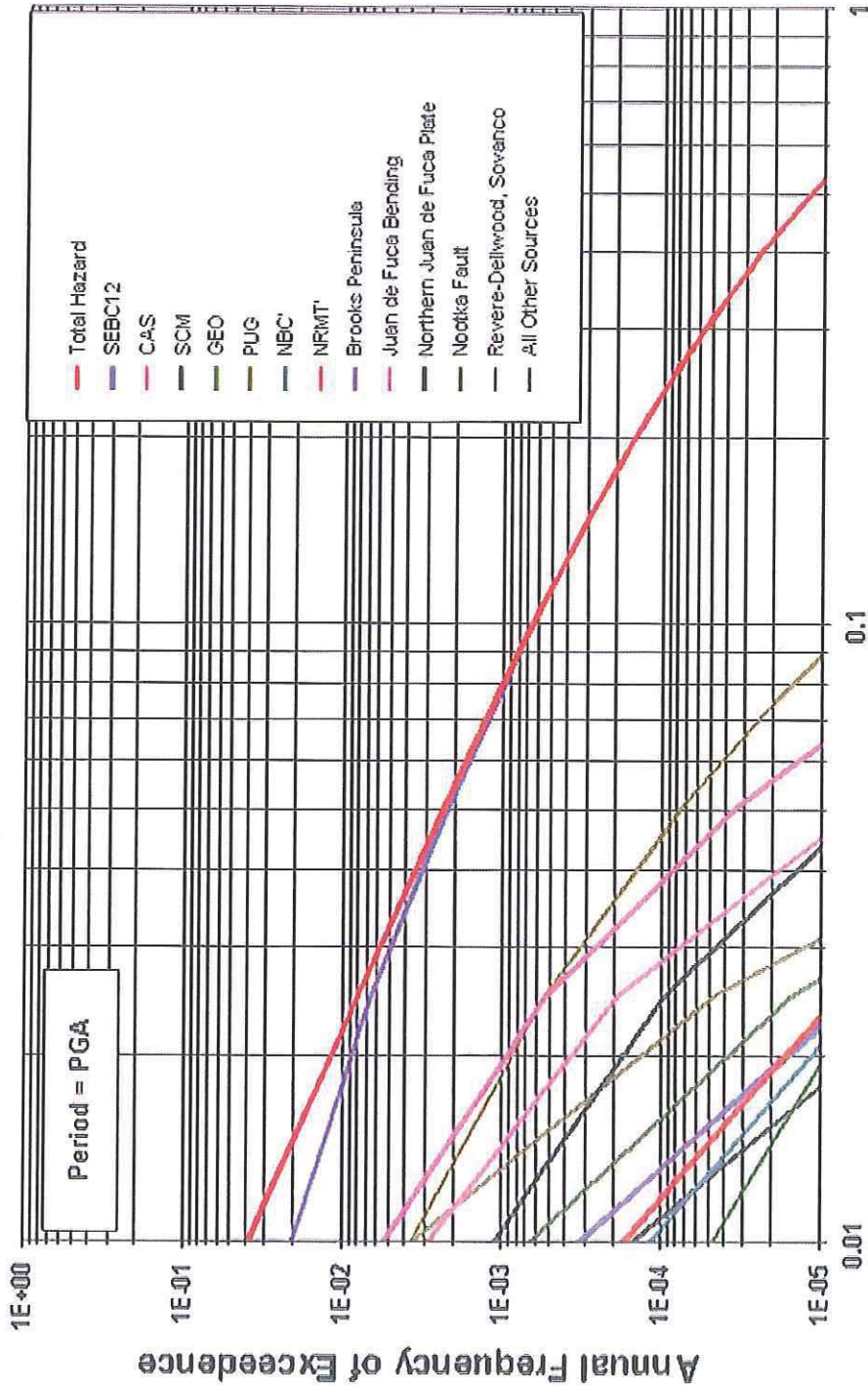


Kleinfelder
Kleinfelder Engineering

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PROJECT	AFTON TAILINGS IMPOUNDMENT
TITLE	SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
FIGURE NO.	MEAN UHRS FOR 475, 2475, 5000 AND 10000 YEAR RETURN PERIODS FOR THE AFTON SITE
FIGURE NO.	MO9713A01
FIGURE NO.	2.19

ALT1B



Peak Ground Acceleration, (g)

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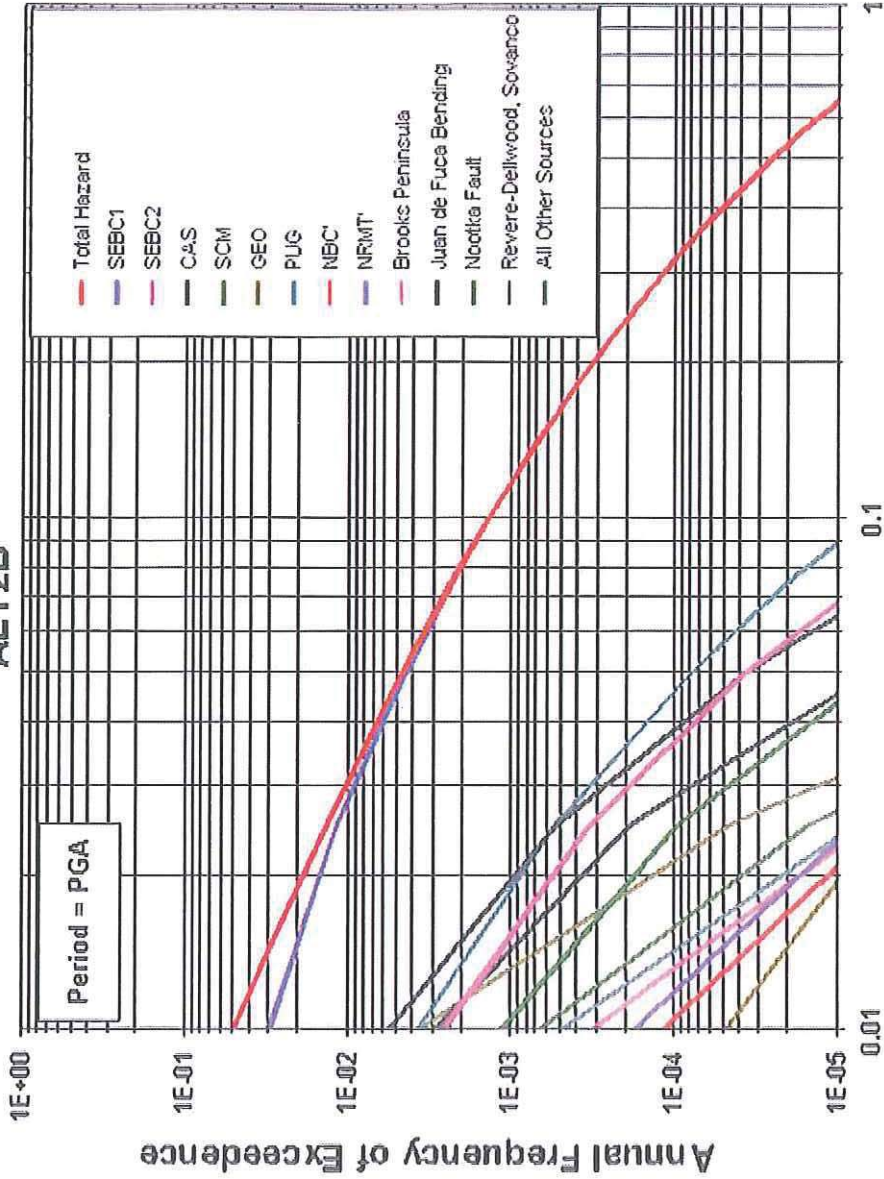


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TITLE	SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
PROJECT NO.	M09713A01
REVISION	2.20

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ALT2B



Peak Ground Acceleration, (g)

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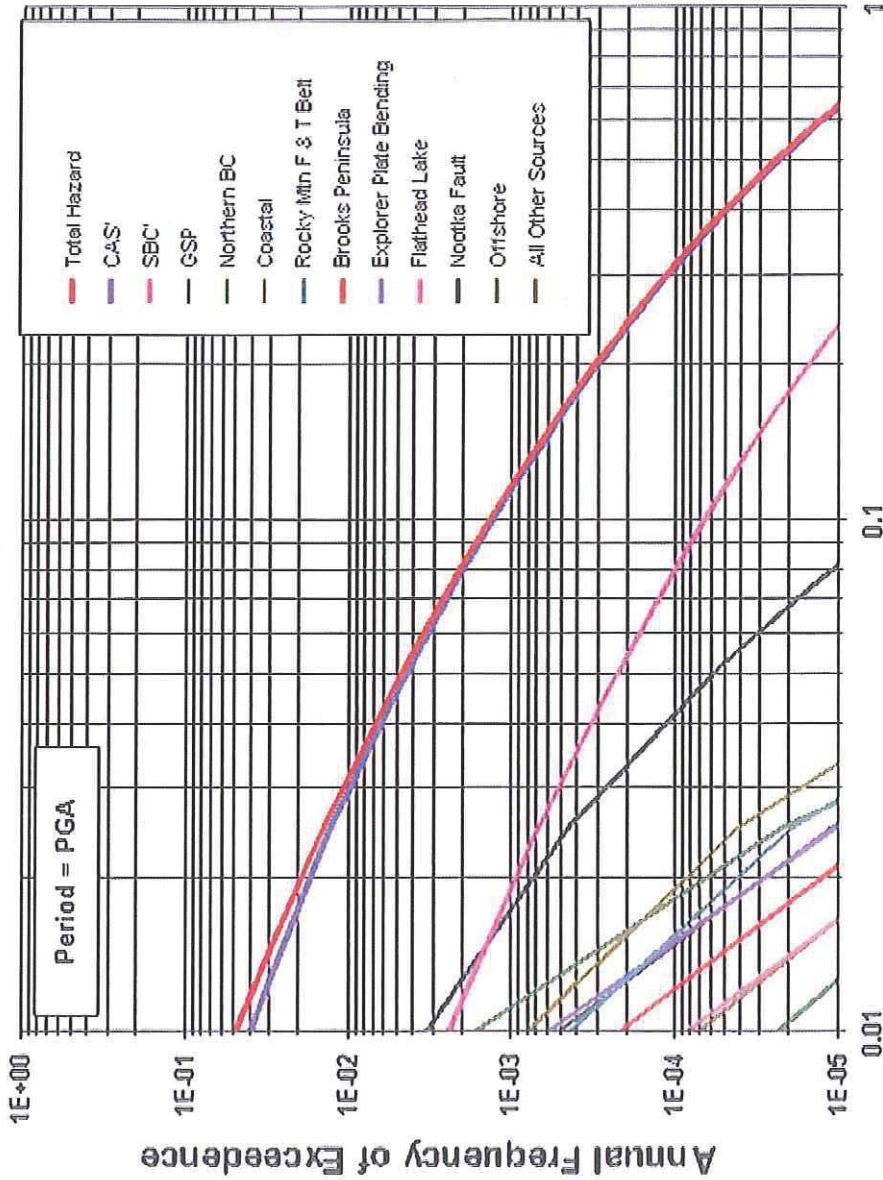
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TITLE: SOURCE ZONE CONTRIBUTION TO PGA HAZARD AT AFTON SITE FOR MODEL 2

PROJECT No: M09713A01

FIGURE No: 2.21

ALT3B



Peak Ground Acceleration, (g)

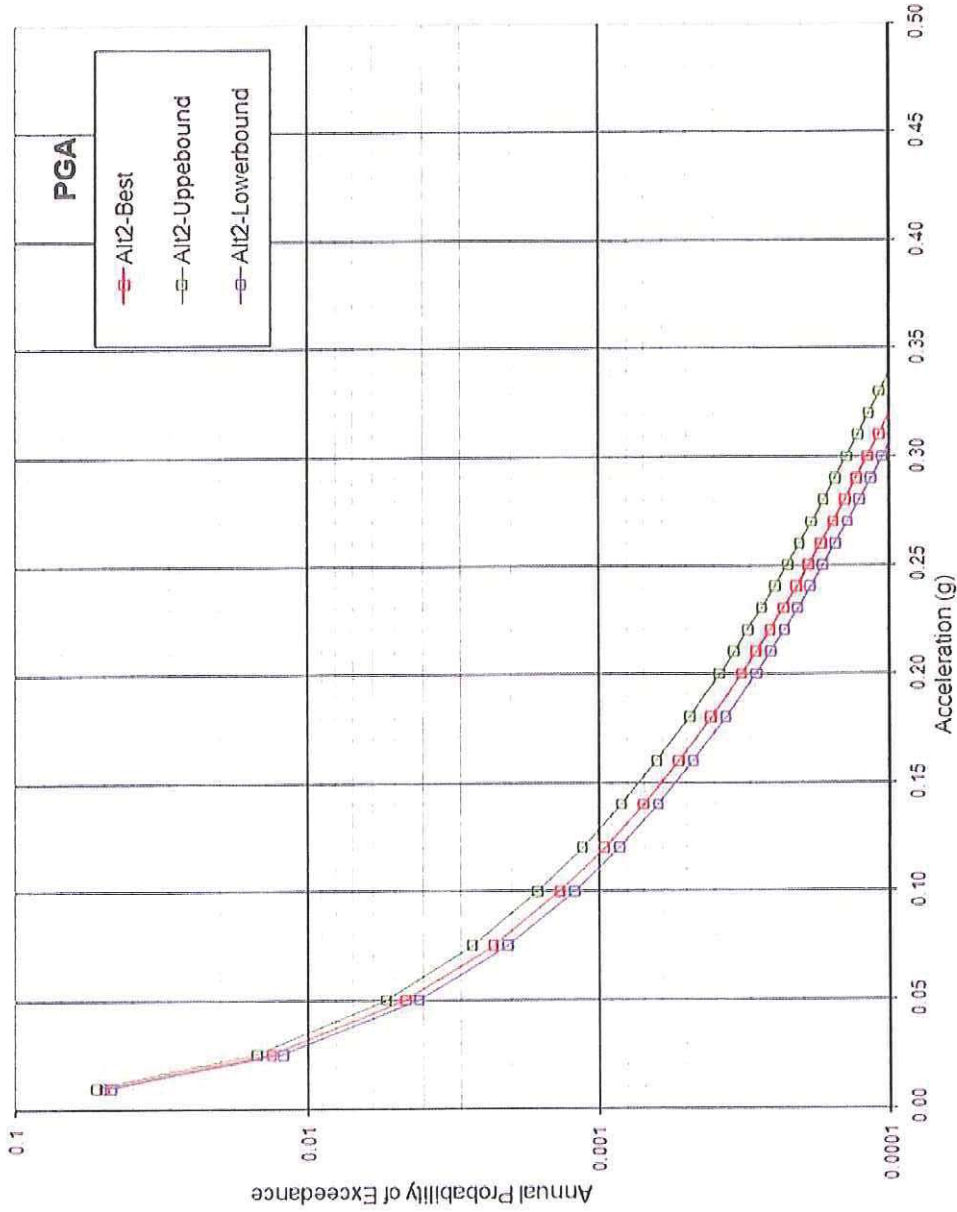
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PROJECT	AFTON TAILINGS IMPOUNDMENT
TITLE	SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
CLIENT	SOURCE ZONE CONTRIBUTION TO PGA HAZARD AT AFTON SITE FOR MODEL 3
PROJECT NO.	M09713A01
FIGURE NO.	2.22

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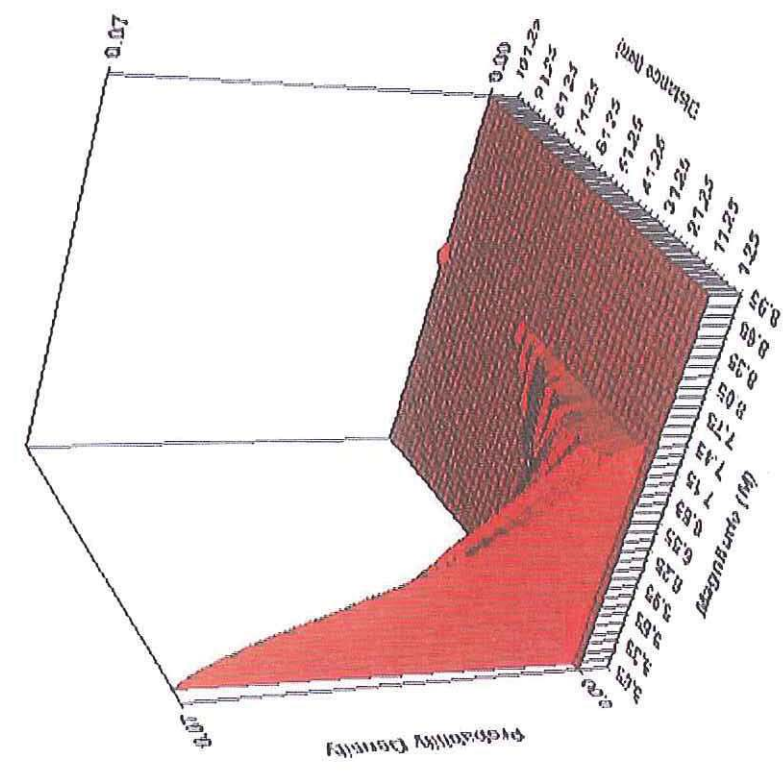


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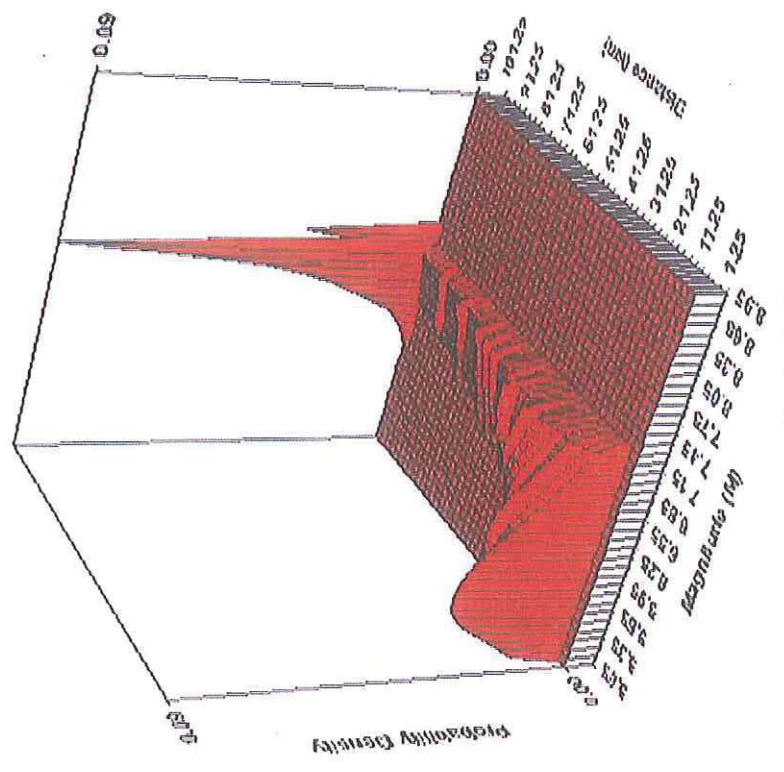
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TITLE	SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
	EFFECT OF MAXIMUM MAGNITUDE ON PGA AT THE AFTON SITE
PROJECT NO.	M09713A01
FRAME NO.	2.23

Magnitude-Distance Deaggregation



Period: PGA
Amplitude: 0.24
Hazard: 9.96976e-005
Mean Magnitude: 6.05
Mean Distance: 15.79

Magnitude-Distance Deaggregation



Period: 1
Amplitude: 0.145
Hazard: 9.95778e-005
Mean Magnitude: 6.57
Mean Distance: 40.98

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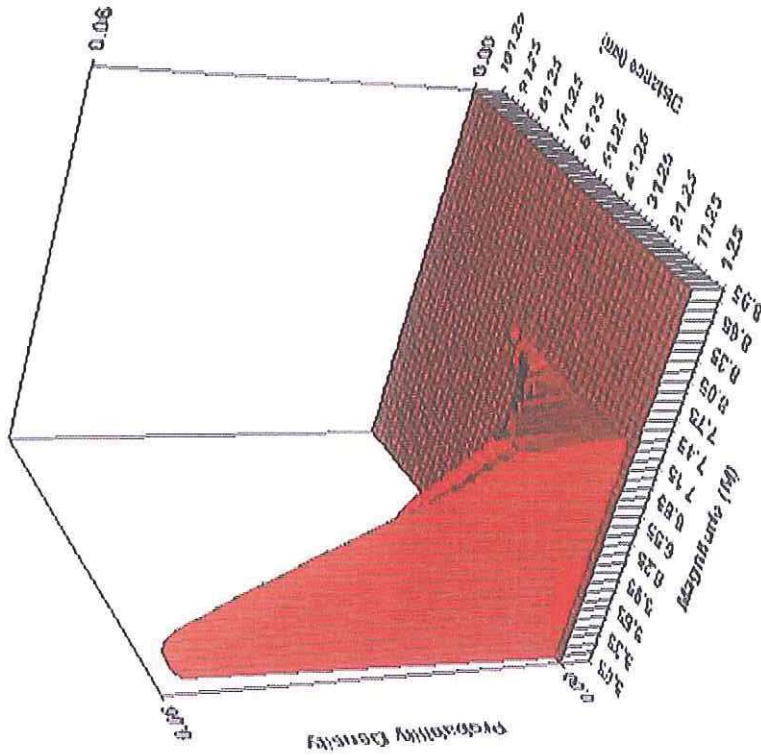
PROJECT: AFTON TAILINGS IMPOUNDMENT
 SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

TITLE: DE-AGGREGATION ANALYSIS RESULTS
 FOR MODEL 1

PROJECT NO: M09713A01
 DRAWING NO: 2.24

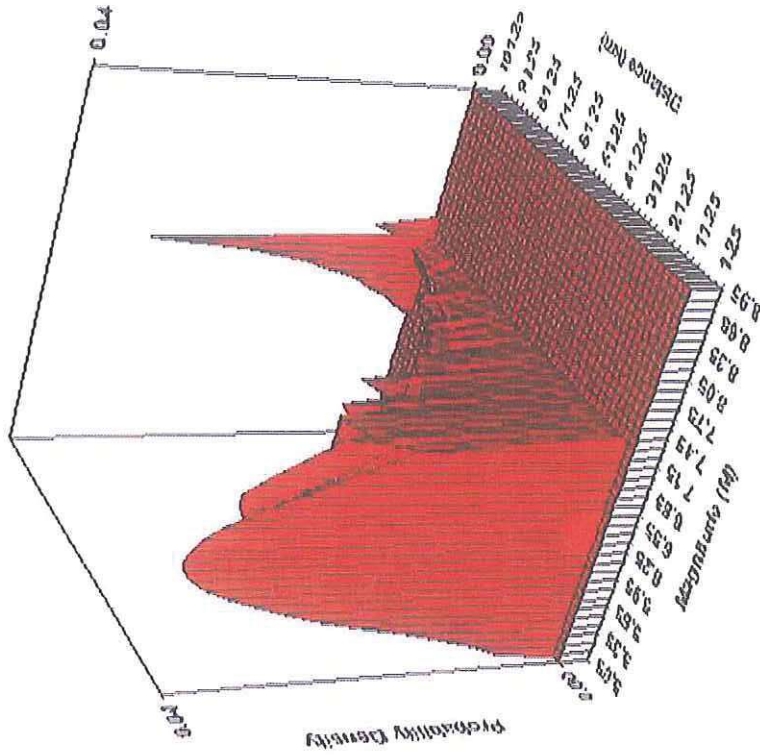
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Magnitude-Distance Deaggregation



Period: PGA
Amplitude: 0.319
Hazard: 0.000100227
Mean Magnitude: 6.20
Mean Distance: 13.73

Magnitude-Distance Deaggregation



Period: 1
Amplitude: 0.199
Hazard: 0.000100595
Mean Magnitude: 6.64
Mean Distance: 26.39

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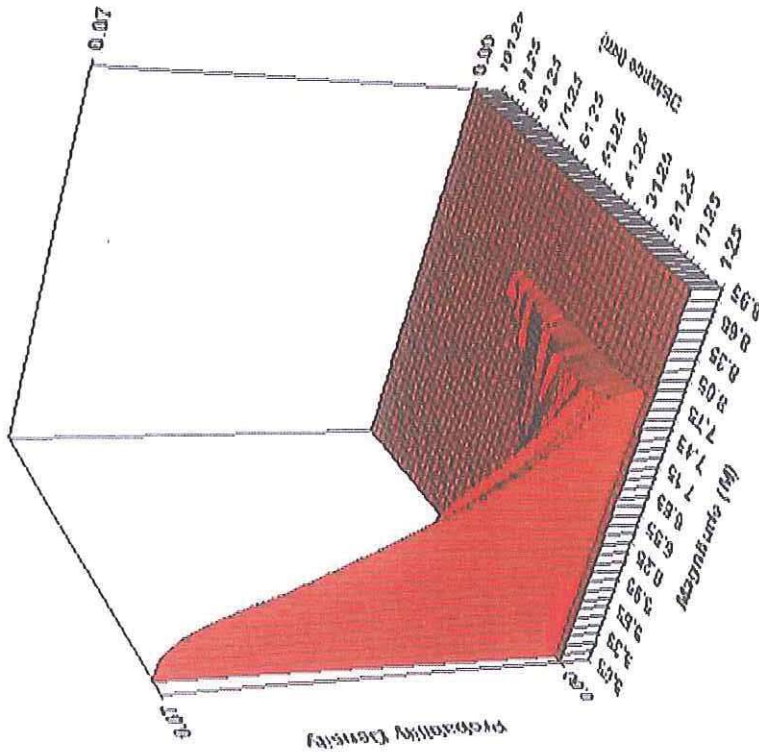
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PROJECT: SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT
 FILE: DE-AGGREGATION ANALYSIS RESULTS
 FOR MODEL 2

PROJECT NO: M09713A01
 FIGURE NO: 2.25

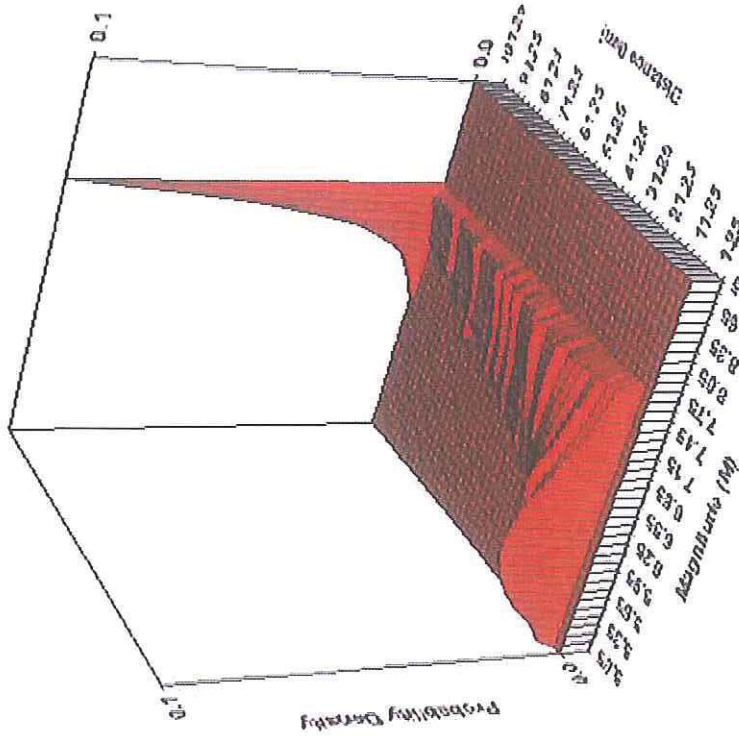
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Magnitude-Distance Deaggregation



Period: PGA
Amplitude: 0.317
Hazard: 9.98976e-005
Mean Magnitude: 6.30
Mean Distance: 15.05

Magnitude-Distance Deaggregation



Period: 1
Amplitude: 0.215
Hazard: 0.000100209
Mean Magnitude: 6.97
Mean Distance: 40.65

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CLIENT

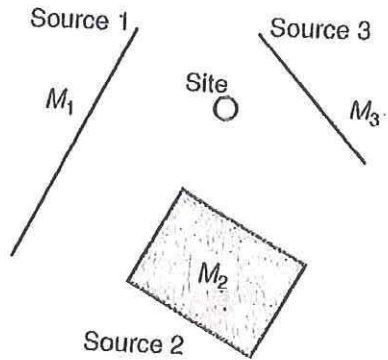
AFTON OPERATING CORPORATION

PROJECT: AFTON TAILINGS IMPOUNDMENT SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

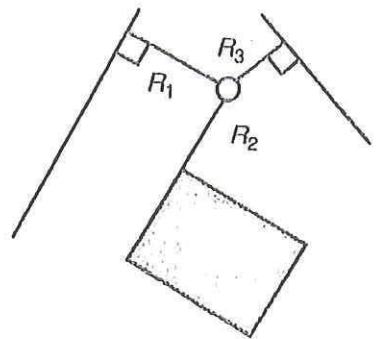
TITLE: DE-AGGREGATION ANALYSIS RESULTS FOR MODEL 3

PROJECT NO: M09713A01

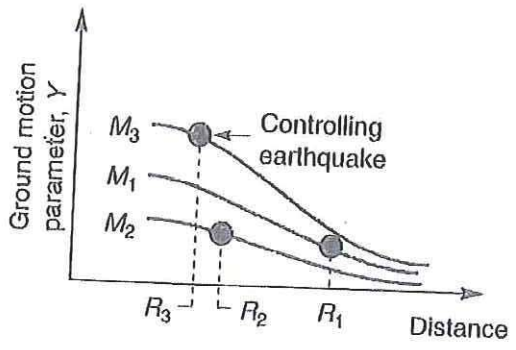
FIGURE NO: 2.26



STEP 1



STEP 2



STEP 3

$$Y = \begin{Bmatrix} Y_1 \\ Y_2 \\ \vdots \\ Y_N \end{Bmatrix}$$

STEP 4

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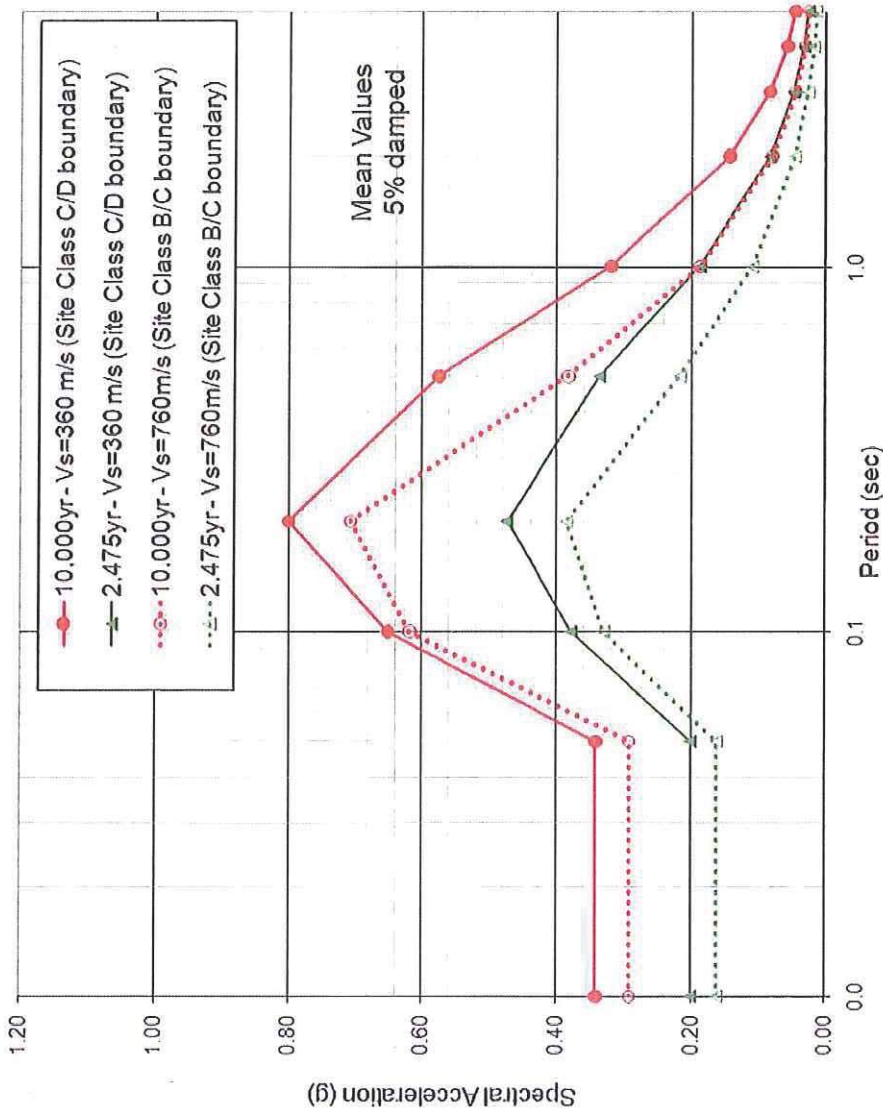
PROJECT
AFTON TAILINGS IMPOUNDMENT
SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

TITLE
STEPS IN DETERMINISTIC SEISMIC HAZARD ANALYSIS

CLIENT:
AFTON OPERATING CORPORATION

PROJECT No.
M09713A01

FIGURE No.
2.27



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AFTON OPERATING CORPORATION

PROJECT: SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

TITLE: COMPARISON OF UHRS CORRESPONDING TO SITE CLASS B/C AND C/D BOUNDARIES

PROJECT No. M09713A01

FIGURE No. 2.28

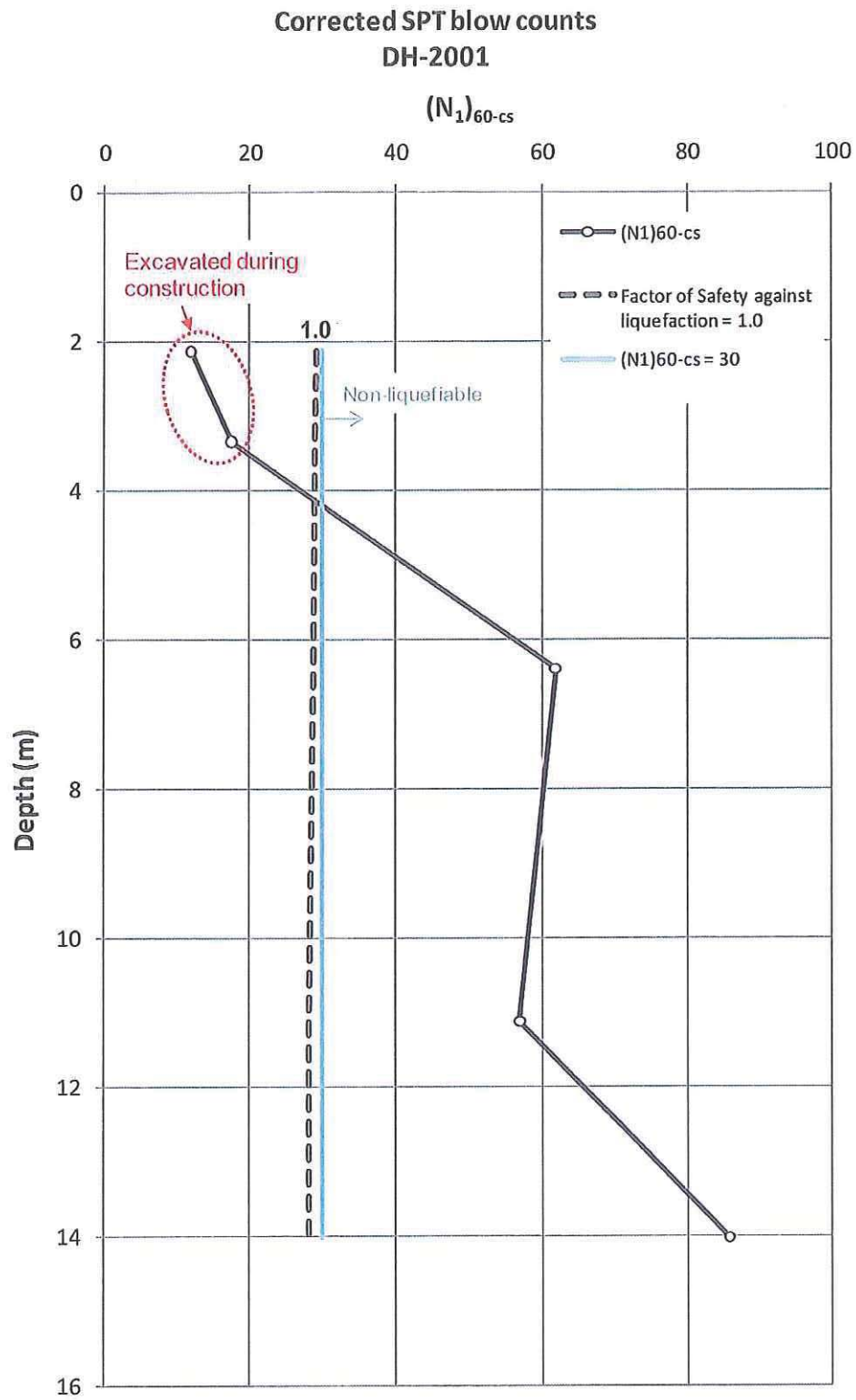


Figure 3.1 Corrected SPT blow counts – DH-2001

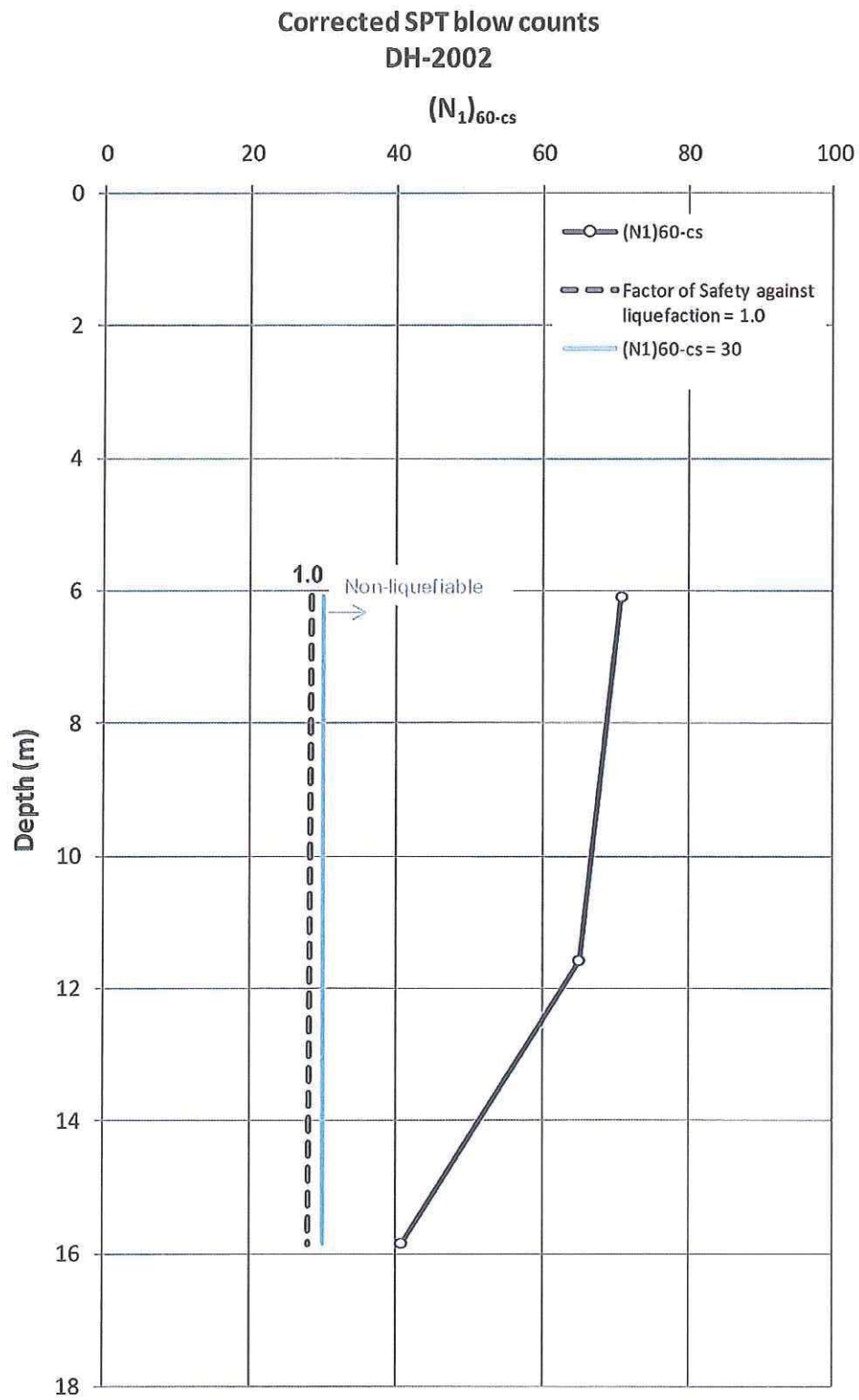


Figure 3.2 Corrected SPT blow counts – DH-2002

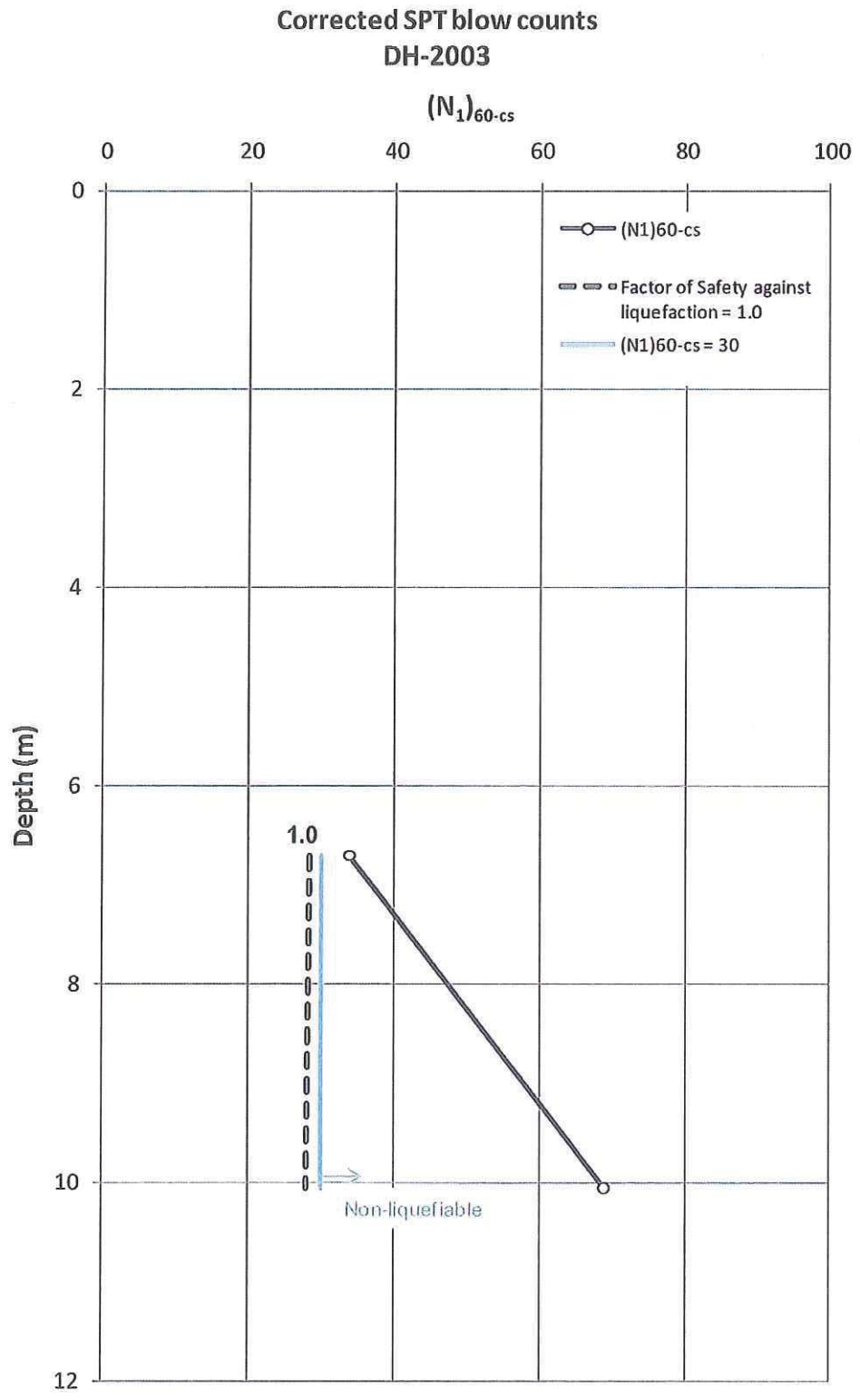


Figure 3.3 Corrected SPT blow counts – DH-2003

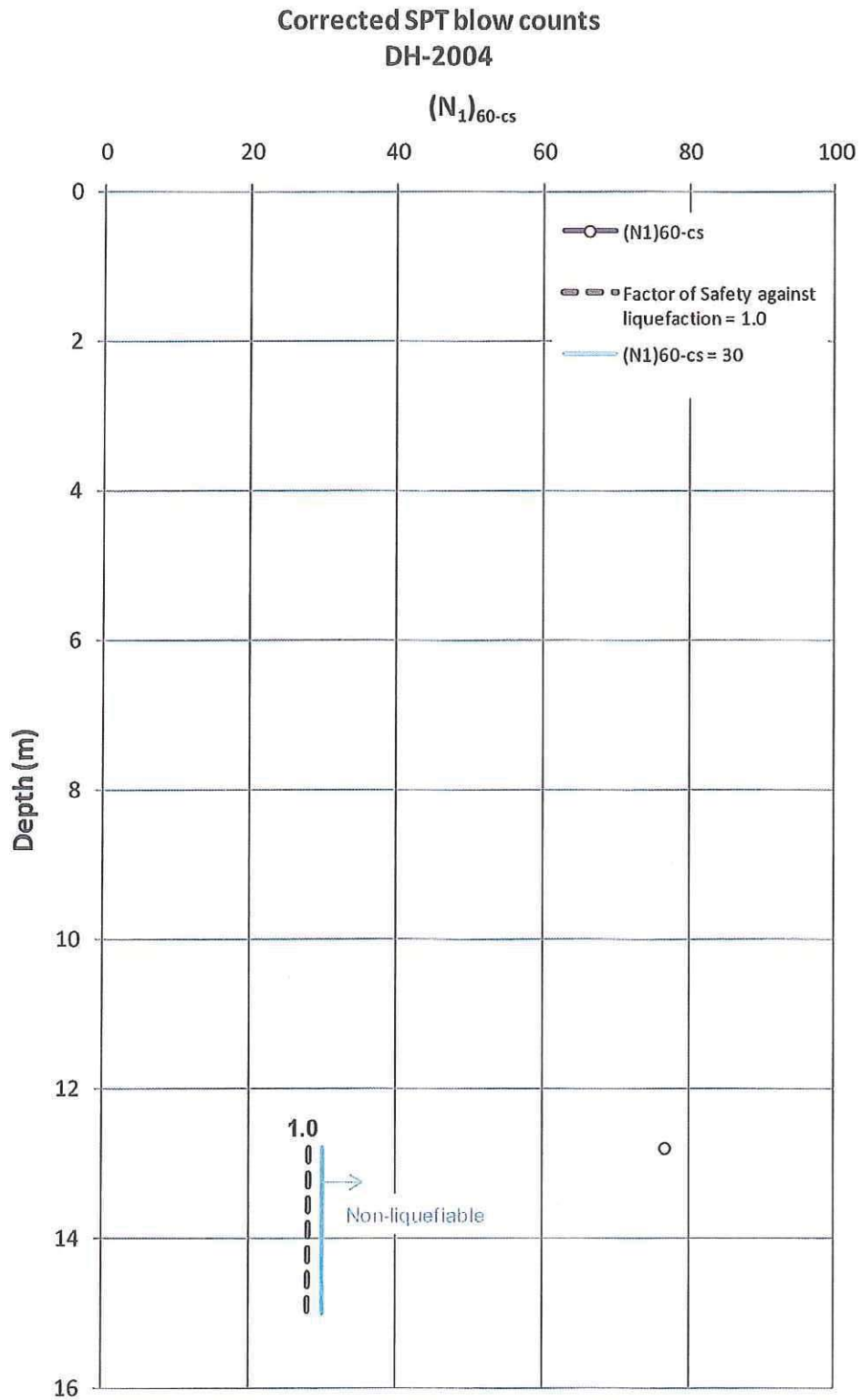


Figure 3.4 Corrected SPT blow counts – DH-2004

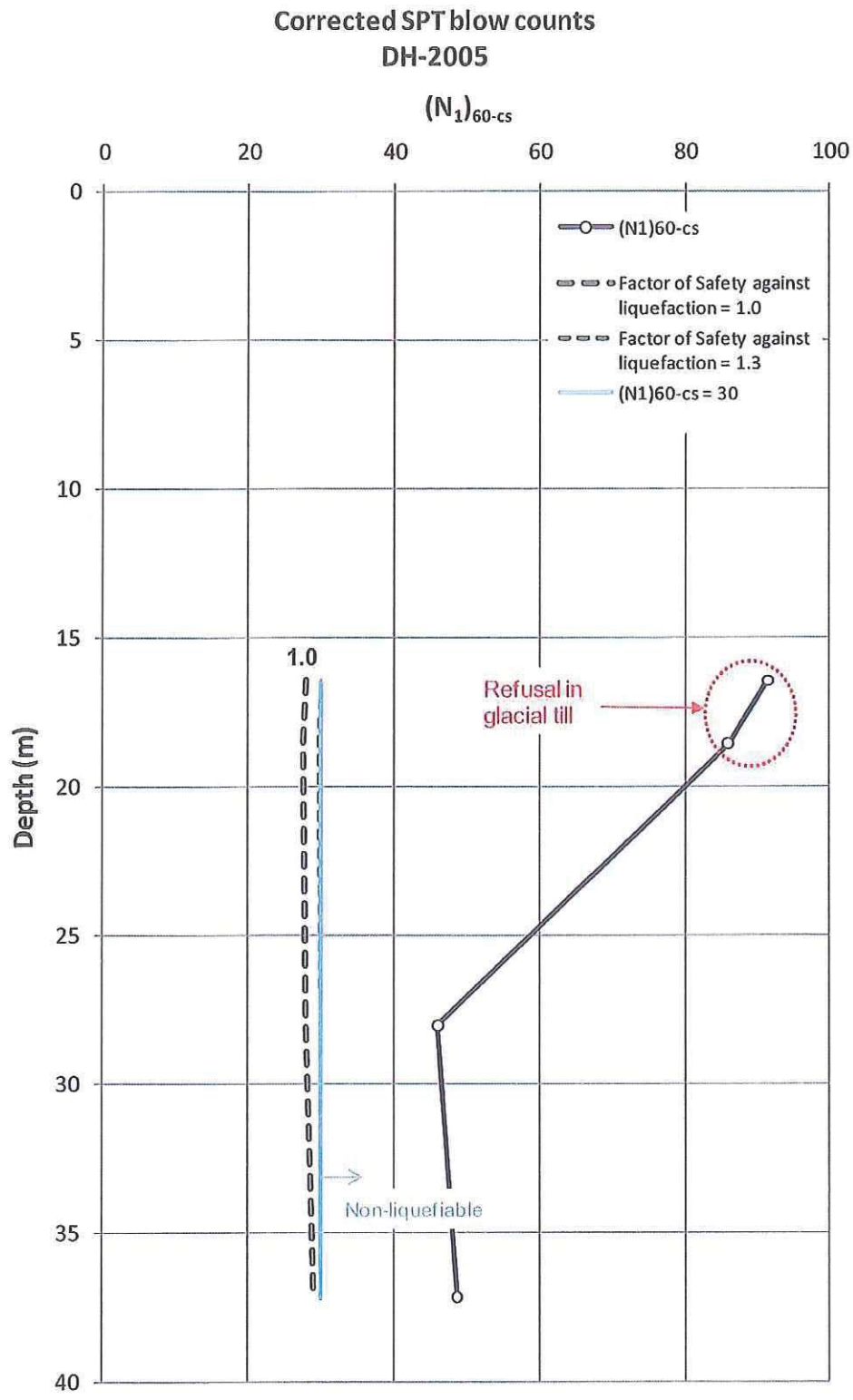


Figure 3.5 Corrected SPT blow counts – DH-2005

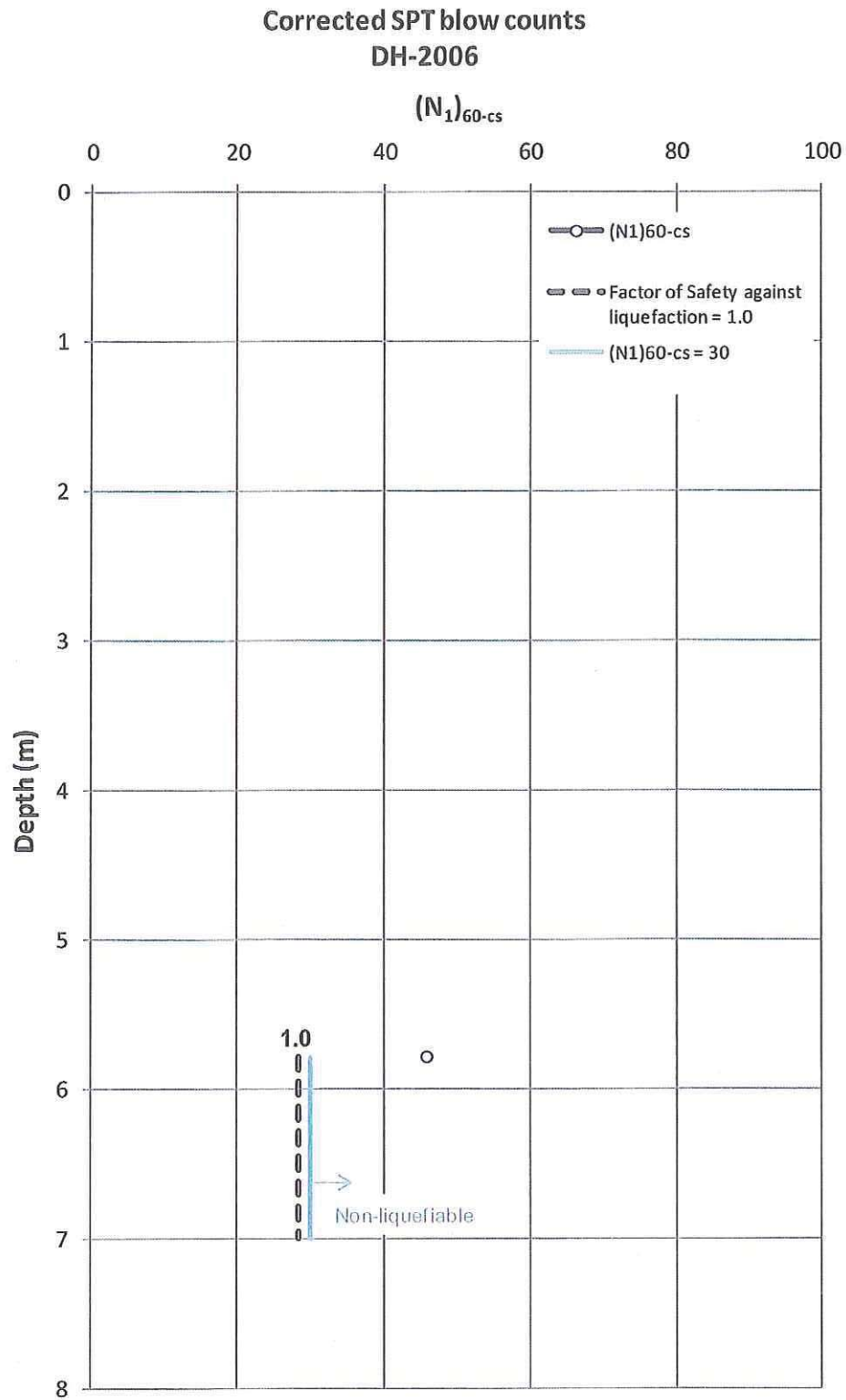


Figure 3.6 Corrected SPT blow counts – DH-2006

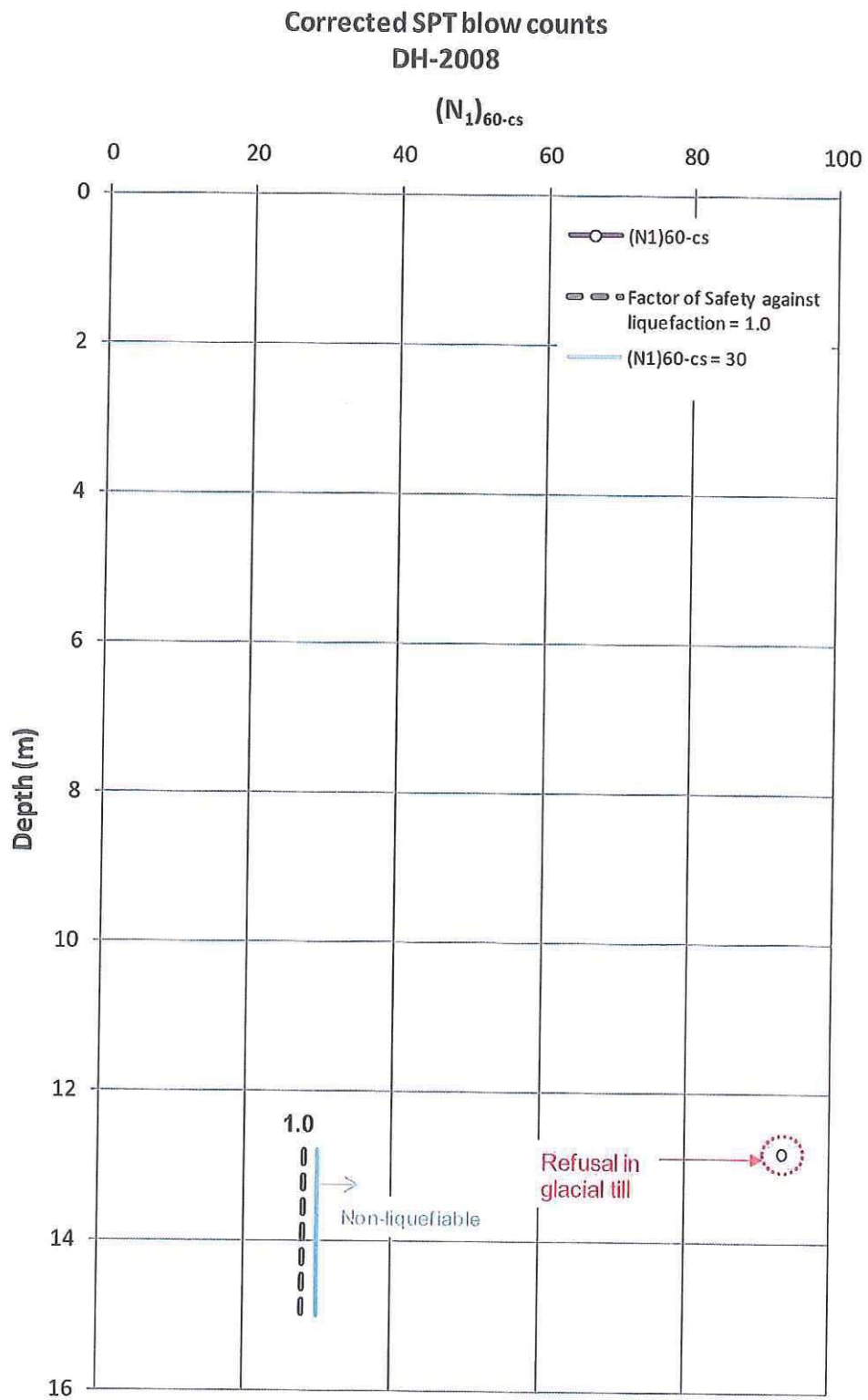


Figure 3.7 Corrected SPT blow counts – DH-2008

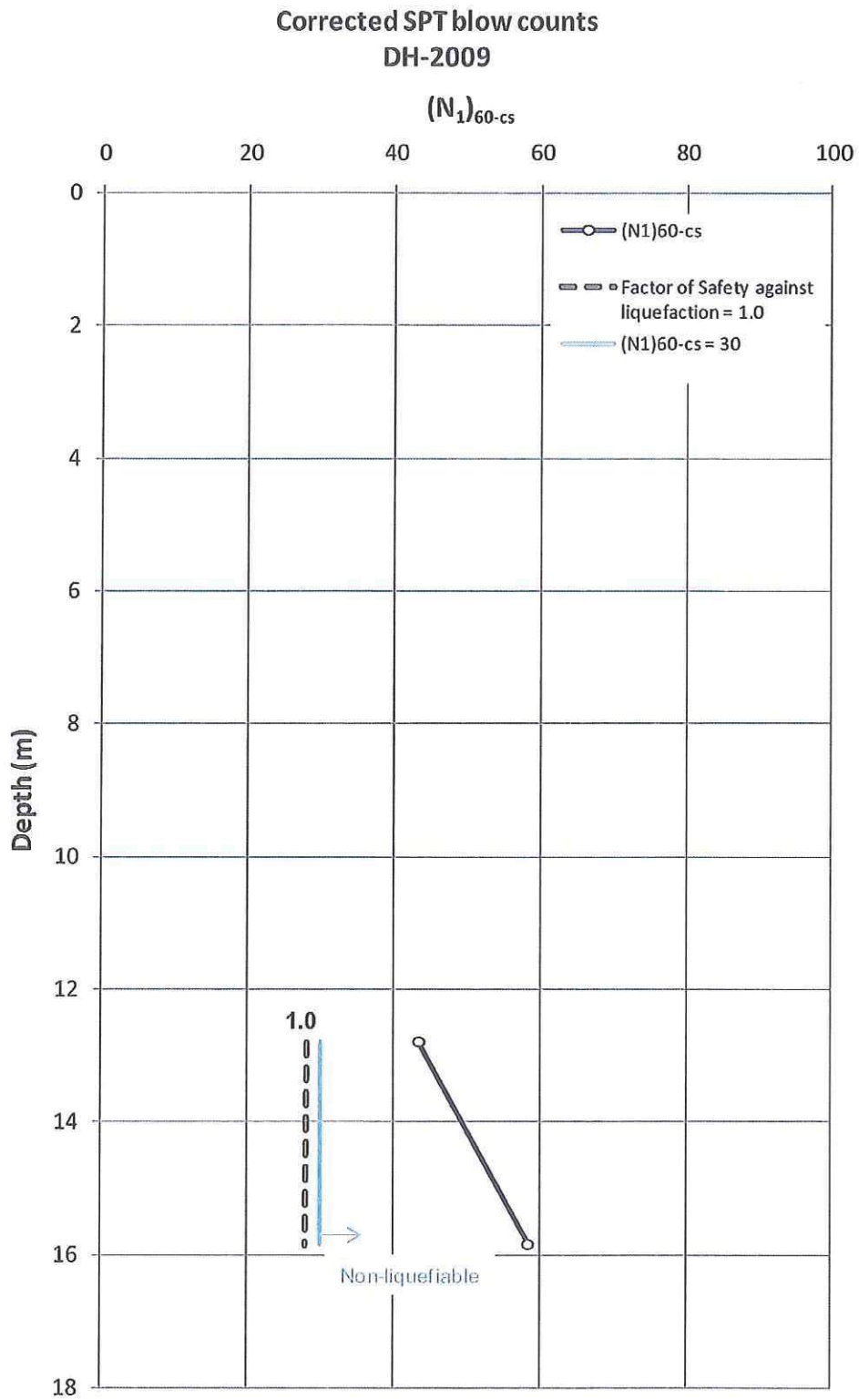


Figure 3.8 Corrected SPT blow counts – DH-2009

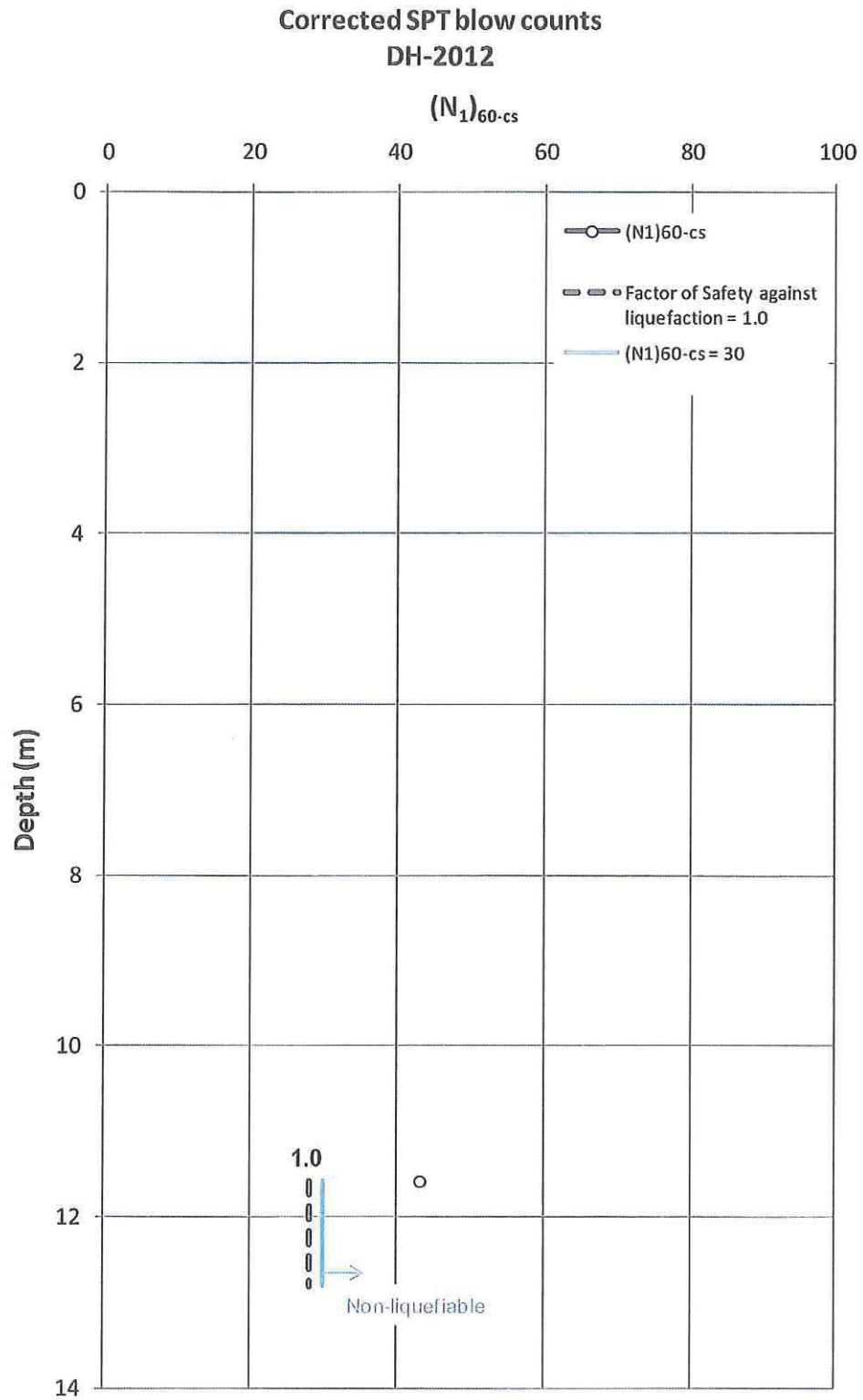


Figure 3.9 Corrected SPT blow counts – DH-2012

Corrected SPT blow counts
DH-2013

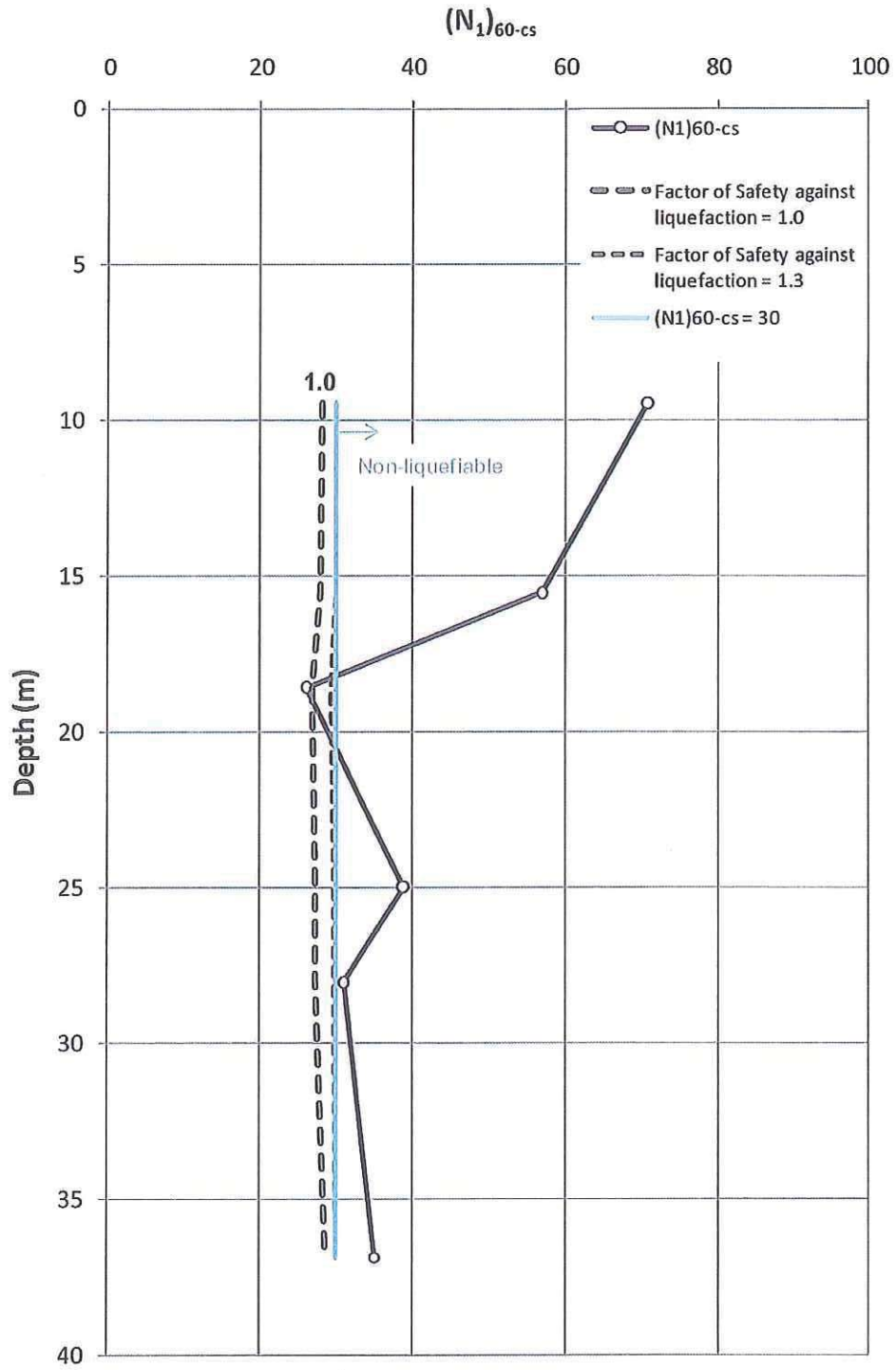


Figure 3.10 Corrected SPT blow counts – DH-2013

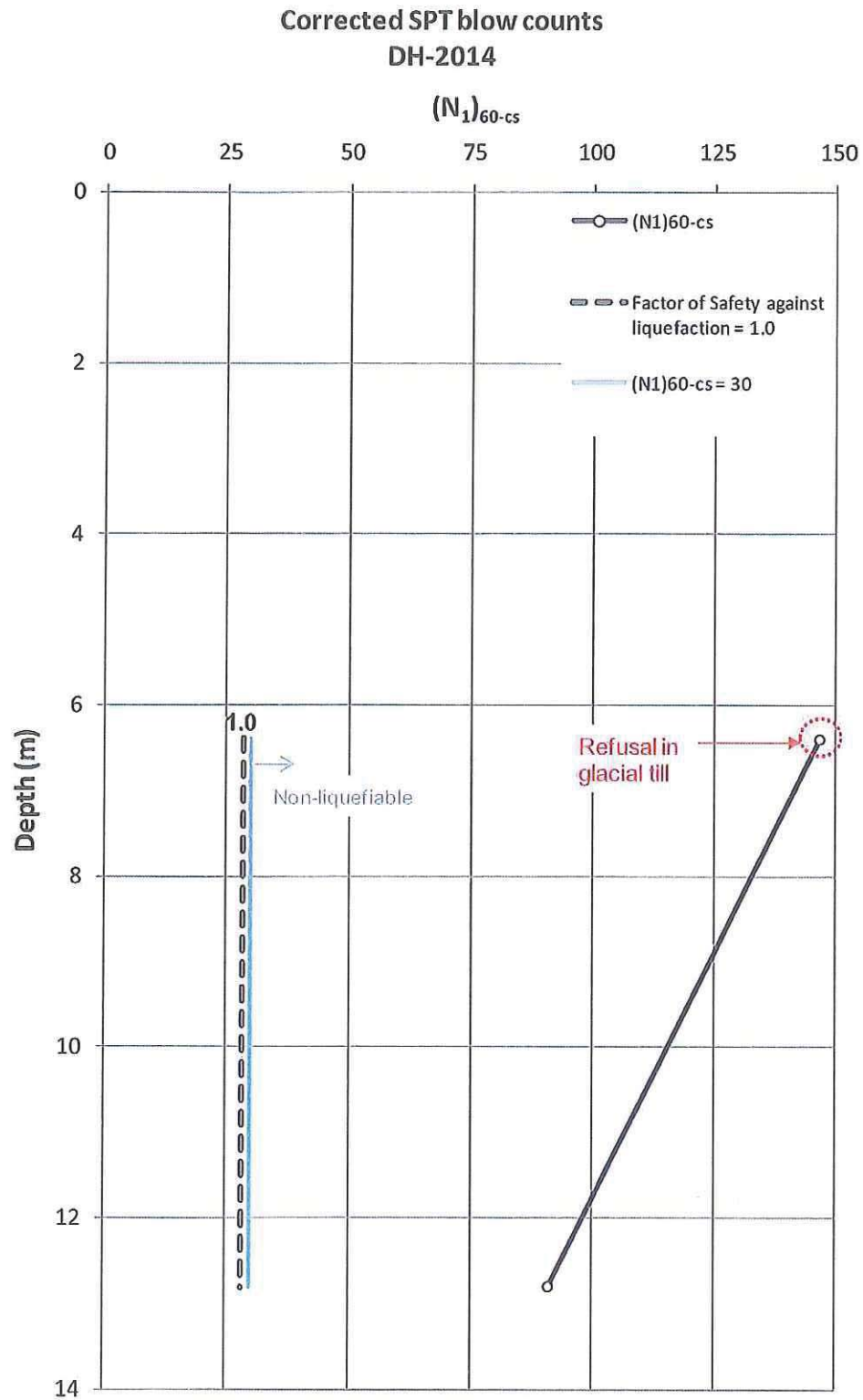


Figure 3.11 Corrected SPT blow counts – DH-2014

Corrected SPT blow counts
DH-2015

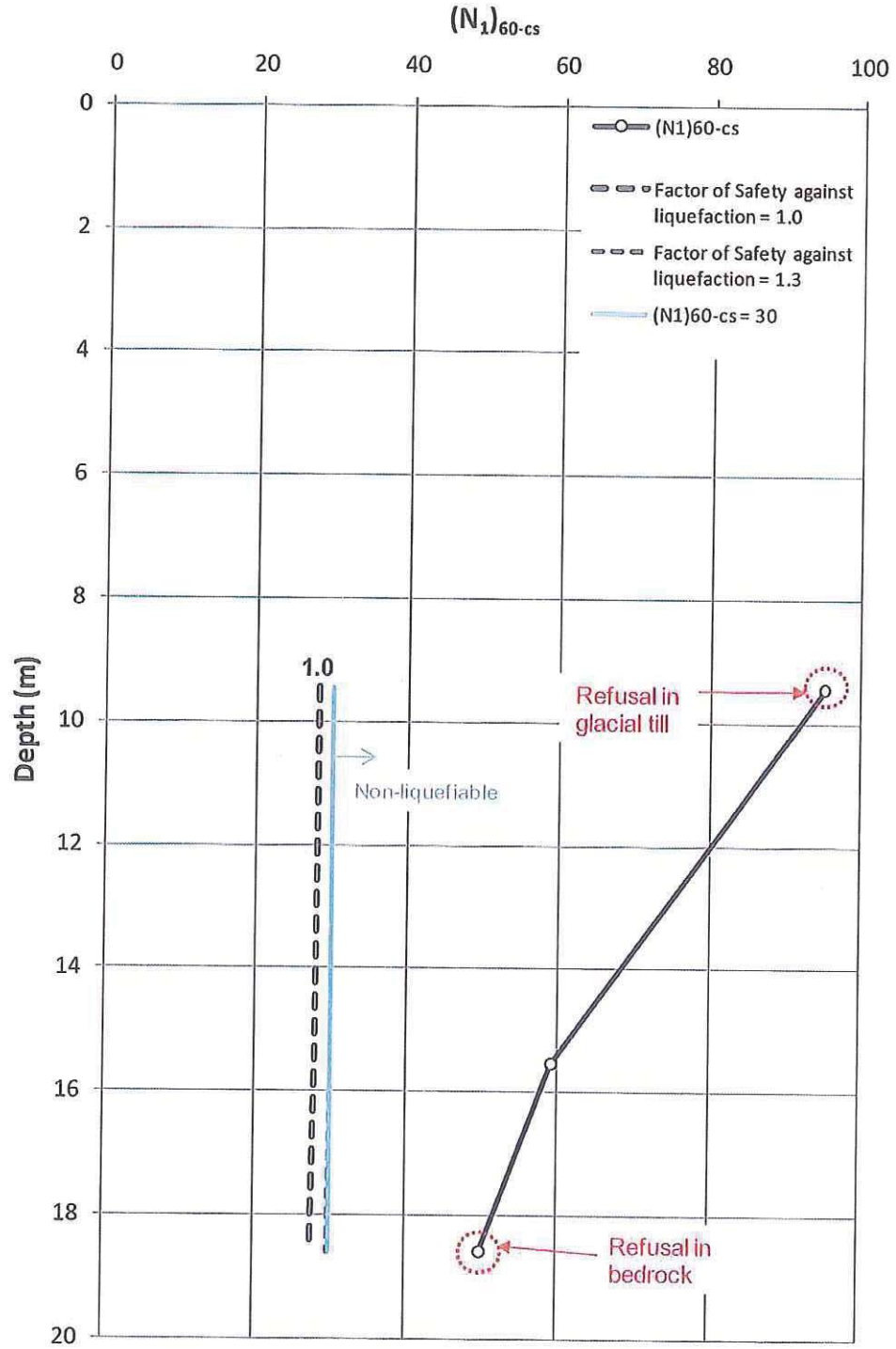
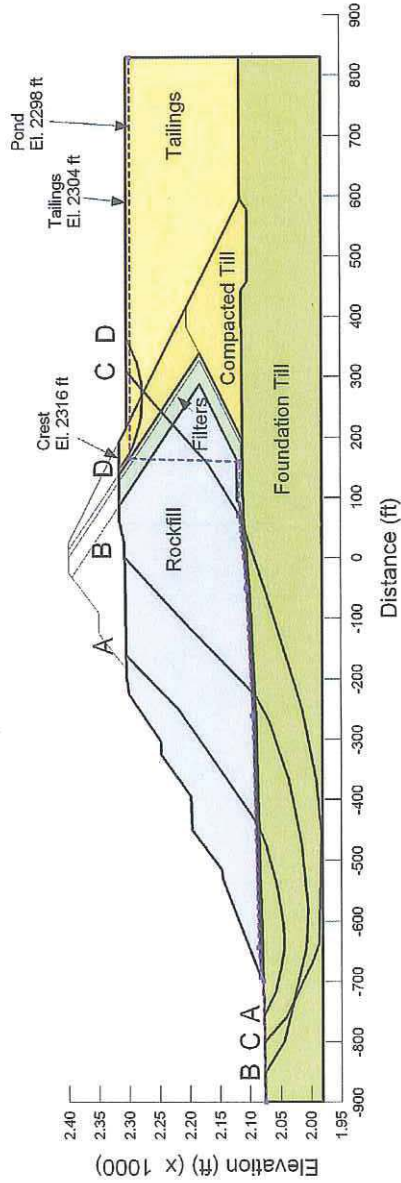


Figure 3.12 Corrected SPT blow counts – DH-2015



Material Properties

Material Type	Unit Weight (pcf)	Effective Friction Angle, ϕ' (degrees)	Post-Liquefaction Strength $S_v / \sigma'_v = 0.15$
Tailings	110	30	-
Compacted Till	143 saturated, 140 unsaturated	32	-
Foundation Till	143	30	-
Filters	125	32	-
Rockfill	125	37	-

Calculated Factor of Safety

Failure Surface	Factor of Safety	
	Static	Post-Liquefaction (Tailings)
Downstream		
A	1.61	1.07
B	1.86	1.19
C	2.36	1.35
D	5.21	1.83
Upstream		
D		3.96

NOT FOR CONSTRUCTION

DATE: MAY 27, 2011

SCALE: 300 FT

PROJECT: AFTON TAILINGS IMPONDMENT SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

TITLE: WEST DAM STABILITY ANALYSIS

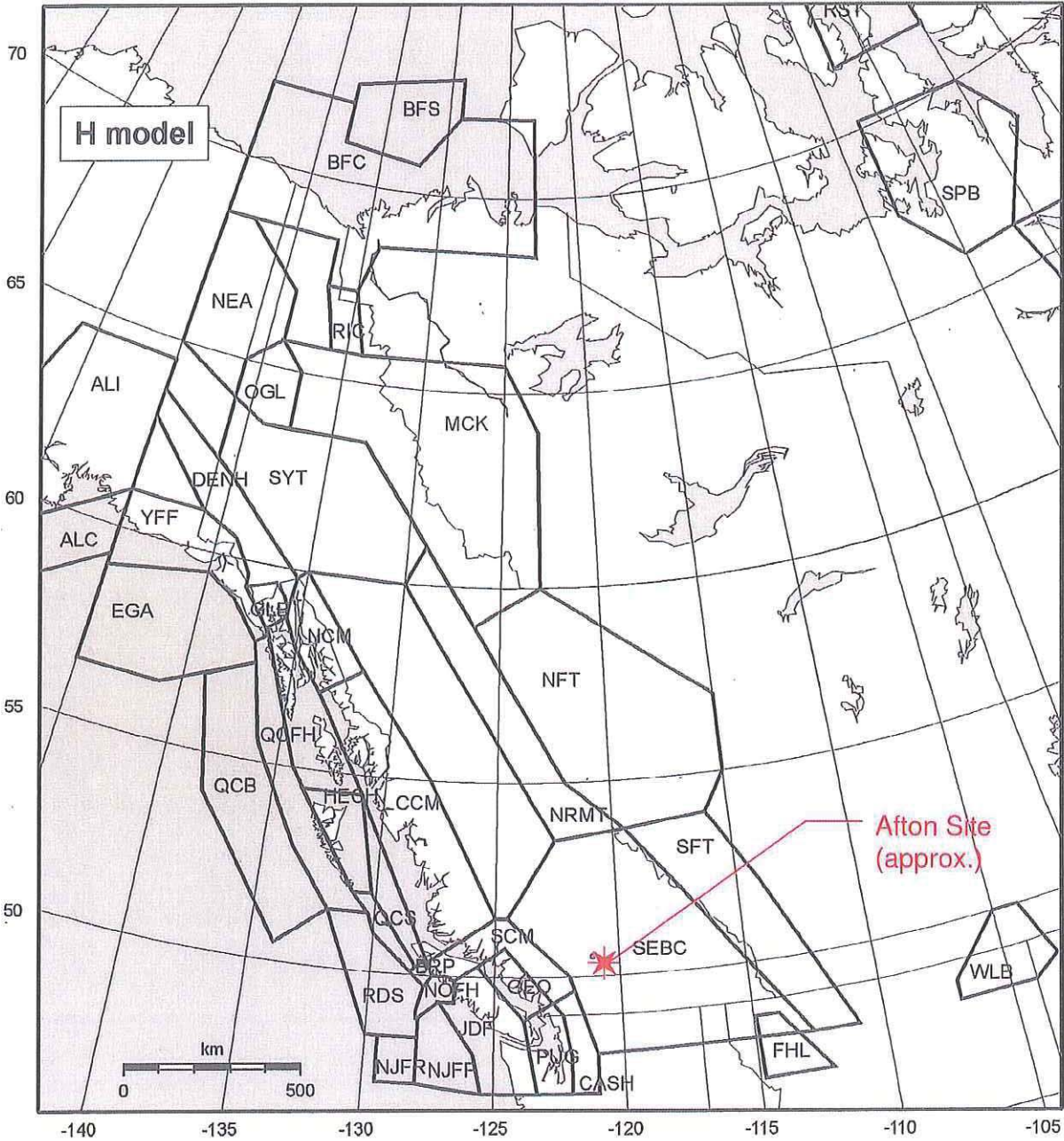
PROJECT No. M05713A01



DATE: 5/24/2011
TIME: 14:49:30
DRAWN BY: M05713A01 - Afton Tailings Pond Damages/Type 3.1.0.dwg (ceag)

APPENDIX I

Geological Survey of Canada GSC-H and GSC-R Model Source Zones



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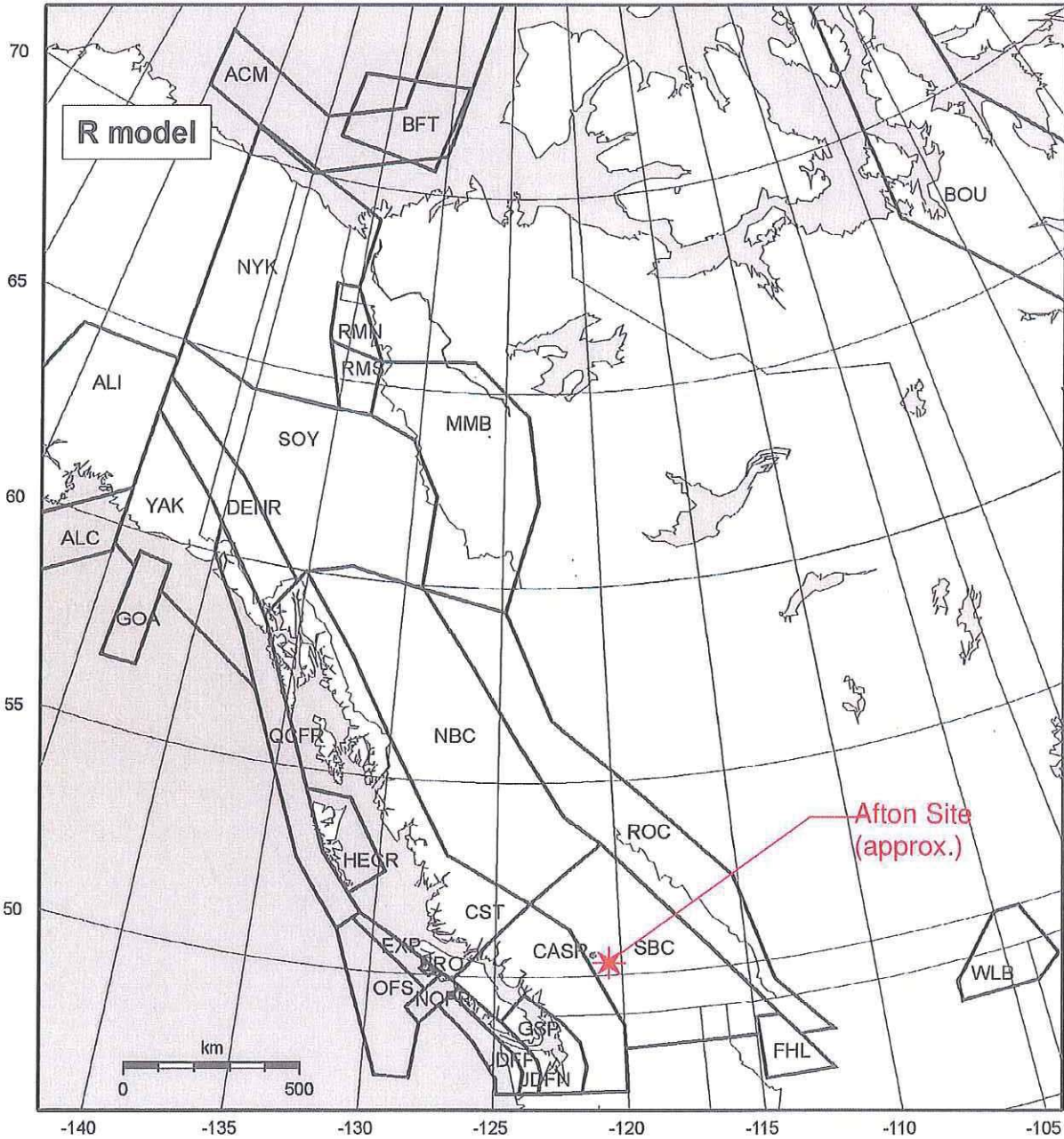
Klohn Crippen Berger

PROJECT	AFTON TAILINGS IMPOUNDMENT SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT	
TITLE	GSC H SEISMIC SOURCE ZONE MODEL	

CLIENT:
AFTON OPERATING CORPORATION

PROJECT No. M09713A01

FIGURE No. I.1



AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA, STATEMENTS, CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORT AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.



Klohn Crippen Berger

PROJECT
AFTON TAILINGS IMPOUNDMENT
SEISMIC HAZARD AND SEISMIC STABILITY ASSESSMENT

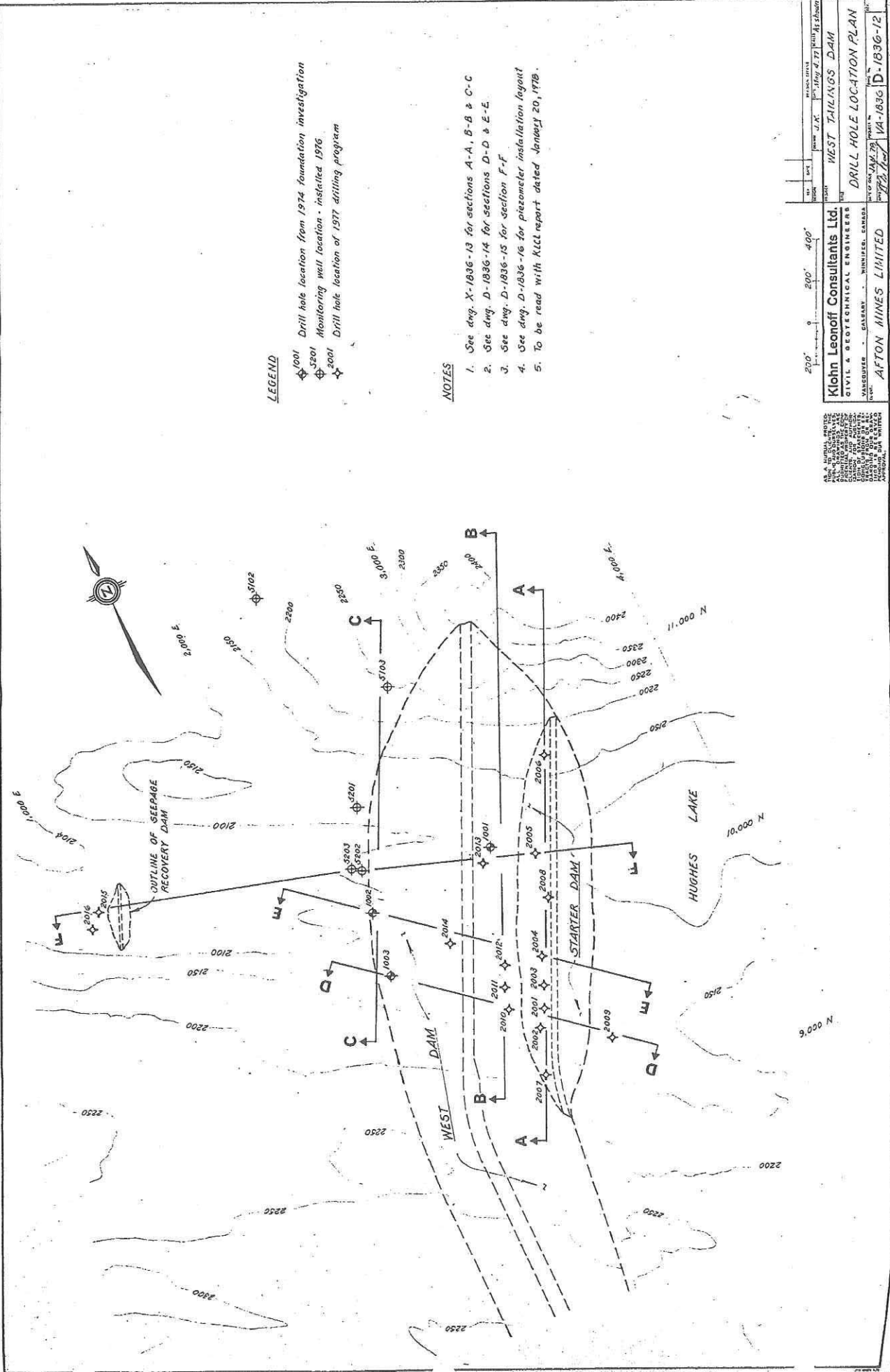
TITLE
GSC R SEISMIC SOURCE ZONE MODEL

CLIENT
AFTON OPERATING CORPORATION

PROJECT No.
M09713A01

FIGURE No.
1.2

APPENDIX II
1977 Drill Hole Location Plan



LEGEND

- ◆ 1001 Drill hole location from 1974 foundation investigation
- ◆ S201 Monitoring well location - installed 1976
- ◆ S201 Drill hole location of 1977 drilling program

NOTES

1. See diag. X-1836-13 for sections A-A, B-B & C-C
2. See diag. D-1836-14 for sections D-D & E-E
3. See diag. D-1836-15 for section F-F
4. See diag. D-1836-16 for piezometer installation layout
5. To be read with KLCL report dated January 20, 1978.

SCALE	1:1000	DATE	1978
PROJECT	WEST TAILINGS DAM		
CLIENT	AFTON MINES LIMITED		
ENGINEER	KLOHN LEONOFF CONSULTANTS LTD.		
LOCATION	VA-1836 D-1836-12		

KLOHN LEONOFF CONSULTANTS LTD.
 CIVIL & GEOTECHNICAL ENGINEERS
 1000 UNIVERSITY AVENUE
 WINDSOR, ONTARIO
 CANADA

AS A PUBLIC NOTICE
 ALL INFORMATION
 CONTAINED HEREIN
 IS UNCLASSIFIED
 DATE 01-10-2011
 BY 60322/UC/LP/STP
 FOR RELEASE UNDER
 THE ACCESS TO
 INFORMATION ACT
 (S. 19/83)

New Gold Inc. No agreement was reached with this company for the construction of berms to direct the water flow into the Afton Open Pit.

The Afton Open Pit was designated in the Afton Closure Plan as the basin for storage of flows from the spillway. This Closure Plan was approved by the South Central Mine Development Committee as part of the decommissioning of the mine.

Tailings Pond Operational Manual

The up-dating of the operational manual was not completed prior to the land sale and transfer of the Reclamation Permit M-112 to Abacus Mining and Exploration or its sister company.

Future Plans and Schedule

Future plans and schedules for the Former Afton Tailing Pond will have to be developed by the new owners of the land and Reclamation Permit holder.