Canadian Geotechnical Journal
Revue canadienne de géotechnique

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Journal:	<i>Canadian Geotechnical Journal</i>
Manuscript ID	cgj-2017-0076
Manuscript Type:	Article
Date Submitted by the Author:	31-Jan-2017
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Quantifying exposure of linear infrastructures to earthquake-triggered transverse landslides in permafrost thawing slopes

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Abstract

The seismic shaking can cause slope instability in otherwise relatively stable permafrost terrains. In addition, rapid ice melting in low-permeability fine-grained soils can lead to excess pore water pressure build up and cause instability in slopes even at small angles. This study addresses the active layer detachment (ALD) slope instability hazard and develops a systematic risk assessment framework for existing and future linear infrastructures such as energy pipelines, bridges and roads traversing permafrost regions. Mild slopes, with average gradient of 7° , are considered in this study as the most representative of actual field conditions. The potential for earthquake-triggered ALD is analytically quantified. State-of-the-art Canadian North ALD morphological statistics are combined with seismic slope stability analyses to determine: (i) the probability of linear infrastructure exposure to permanent ground deformations (PGD) caused by ALD, and (ii) the extent of the potential PGD that the linear infrastructure may be subjected to. Monte Carlo technique is applied to simulate and assess the sensitivity of the model to parameters such as earthquake magnitude and source-to-site distance. The findings from this study can be used to evaluate the vulnerability of linear infrastructures exposed to the ALD hazard.

Keywords: linear infrastructures, landslide hazard, permafrost, probability of exposure, permanent ground deformation.

71 Introduction

72 Southwestern Yukon, British Columbia, the Mackenzie and Richardson Mountains and beneath the
73 Beaufort Sea are zones of high seismicity in Western Canada (Hyndman et al. 2005). In particular,
74 the high seismicity along the Mackenzie Valley, Richardson Mountains and offshore beneath the
75 Beaufort Sea represents a potential threat to the safety and integrity of the existing and projected
76 linear infrastructures in the region. For example, two seismic effects can develop critical stress and
77 strain levels in pipelines and impact their integrity: the transient ground shaking that can further
78 be altered by the local site effects with respect to the presence of permafrost and unconsolidated
79 sediments (Hyndman et al. 2005; and Dadfar et al. 2016); and the significantly more dangerous PGD
80 due to earthquake induced landslides, slope instabilities and sediment liquefaction. As a result of
81 the M_w 6.9 and M_w 6.7 Nahanni earthquakes (1985), rock falls and rock avalanches occurred in the
82 Mackenzie Mountains and liquefaction was observed at Little Doctor Lake, located 80 km away
83 from the Mackenzie gas project right-of-way (Savigny et al. 2005).

84 The active-layer detachment (ALD) is probably the most common type of landslide observed in
85 permafrost terrains (Aylsworth et al. 2000; Dyke 2004; Lipovsky and Huscroft 2006). The active
86 layer, located on top of the permafrost table, is the surficial soil layer that is subject to annual
87 freeze-thaw cycles. Instability and downslope movement over the permafrost table surface is
88 generally referred to as active-layer detachment. ALDs have been detected in the Mackenzie Valley
89 and Fosheim Peninsula in Northwest Territories, in the Canadian Arctic Archipelago, and in Alaska
90 (Lewkowicz 1990). The concept of ALD in the literature may refer to two different types of failure
91 mechanisms (Lewkowicz 1990): flow (Hughes et al. 1973; McRoberts and Morgenstern 1974; and
92 Aylsworth et al. 2000), and slide (Lewkowicz 1990; Harris and Lewkowicz 1993, 2000; Lewkowicz
93 and Harris 2005). Despite the different failure mechanisms and material transfer, McRoberts and
94 Morgenstern (1974), and Lewkowicz and Harris (2005) have employed the concept of infinite slope
95 stability analyses to characterize the ALD. Warm summer temperatures, intensive rainfalls and loss

of vegetation cover due to forest fires or construction can trigger ALD. In low-permeability fine-grained soils, rapid ice melting can lead to excess pore water pressure build up within the active layer and cause instability in slopes even at small angle (McRoberts and Morgenstern 1974; Morgenstern and Nixon 1971). As elsewhere, the seismic shaking also causes slope instability in otherwise relatively stable permafrost terrains, e.g., in the Mackenzie Valley following the Nahanni earthquakes (1985) (Savigny et al. 2005). It is therefore desirable to address the ALD hazard and develop a systematic risk assessment framework for existing and future linear infrastructures.

The objective of this study is to analytically quantify the potential and the extent of transverse ALD landslides that pose threats to the integrity of linear infrastructures. To explain the methodology, this study focused on buried energy pipelines that are good examples of linear structures. The probability of occurrence of transverse ALDs along a specified route was represented by a Poisson distribution. Probabilistic seismic slope stability analysis was carried out by Monte Carlo simulation technique. Since linear infrastructures are rarely built in the direction perpendicular to the steepest slopes, only slopes with average gradient of 7° were considered. The outputs determine the portion of potential ALDs that impact the linear infrastructure (probability of exposure) as well as the extent of PGD that endangers the infrastructure.

Linear infrastructure exposure to transverse ALD hazard

Transverse ALDs represent a significant threat when their runout zone crosses the aboveground linear infrastructure axis. In case of underground infrastructure, however, the threat is more likely where the infrastructure is located within the detached layer. To assess the likelihood of a linear infrastructure segment being exposed to PGD resulting from an earthquake-induced ALD, mechanisms of material transfer should be identified first. According to Mathewson and Mayer-Cole (1984), and Lewkowicz (1990), the ALD transfer mechanism integrates both the translational and compressional movement of a block of active soil material (Fig. 1). In cases where the resistance against the detachment of the block is not sufficient, the movement tends to be translational.

Assuming that the geometry of the block remains constant, the PGD extent along the scar zone is uniform and equal to the scar length (L_s) at each point (Fig. 1a). In this case, a pipeline buried in the active layer is exposed to the PGD if its axis is located within a maximum distance of $(L - L_s)$ from the scar crown, i.e., $S < (L - L_s)$. On the other hand, for cases where considerable resistance is exerted against the movement, the material is compressed and piled at the toe of the landslide and the block length is shortened. Assuming that the PGD extent vary linearly along the landslide length (L), as indicated in Fig. 1b, a pipeline will be subject to PGD for $S < L$ and the PGD is inversely proportional to the distance (S) between the pipeline axis and the scar crown.

Fig. 1. Mechanisms of material transfer and distribution of the transverse PGD for: (a) translational and (b) compressional movement. L represents the total landslide length, L_s is the scar zone length and S is the distance of scar crown to pipeline axis.

It is now important to determine the probability of linear infrastructure exposure to PGD. The distance S from the infrastructure axis to the scar crown is a site-specific parameter that depends on the surficial geology and soil mechanical properties, vegetation cover, slope angle, slope aspect, permafrost coverage and ice content (Blais-Stevens et al. 2010). Due to its flexibility in representing natural phenomena, the standard lognormal distribution is proposed herein as a theoretical distribution for S . The index of exposure (I_E) can be defined as:

$$(1) \quad I_E = L - S$$

Accordingly, the linear infrastructure will be impacted by transverse ALD only for positive I_E . Then, the probability of the exposure event outcome (E) defined for $I_E > 0$ is given by:

$$(2) \quad P(E) = P(I_E > 0) = 1 - P(I_E < 0)$$

In case of buried pipelines, the thickness of detached layer (H) should be large enough to impact the pipeline with burial depth of Z_p , i.e., $H \geq Z_p \cdot \cos \theta$. As can be seen in Fig. 2, both L and S are required to determine the linear infrastructure exposure to transverse PGD. The occurrence of transverse

ALDs along a specified route can be expressed by a Poisson distribution with mean occurrence rate of v_{ALD} (Fig. 2).

Fig. 2. Top view of a hypothetical linear infrastructure segment exposed to potential transverse active-layer detachments (ALD) and the relevant parameters: ALD width (W) and length (L), length of scar zone (L_s) and distance of scar crown to infrastructure axis (S). ALDs that impact the infrastructure are shown in grey.

ALD geometry

Based on the inventory of meteorologically-triggered ALDs at three different sites in the Fosheim Peninsula, a continuous permafrost region in the Canadian territory of Nunavut, Lewkowicz (1990) presented certain statistical aspects of typical ALD morphological characteristics. The distribution of the ALD slide length (L) and width (W) at the three sites appear more or less similar and positively skewed and the depths of failure range between 0.2 and 0.65 m. Study of Lewkowicz and Harris (2005) on ALDs in the discontinuous permafrost region of the central Mackenzie Valley revealed that they may occur anywhere from the slope top to its bottom. The statistical averages for the morphology and morphometry of the ALD were quite similar for both studies. Two typical geometries were observed: compact and elongated. The compact ALD geometry is characterised with length and width of less than 30 m and runout distances of only a few meters. The elongated forms, on the other side, may extend all the way from the top to the bottom of the slope with length-to-width ratios greater than 20 and runout distances attaining more than 500 m. The ALD widths in both regions were lognormally distributed, whereas the ALD lengths were lognormally distributed only in Fosheim Peninsula. At the Mackenzie Valley site, ALD lengths seem slightly better represented by the normal distribution. As an example, Table 1 shows the estimated lognormal distribution parameters based on the data presented in Lewkowicz (1990) for ALDs at “Hot Weather Creek” site on the Fosheim Peninsula.

In parallel, investigating well-documented non-permafrost landslide events including about 25,000 cases occurred in USA, Italy and Guatemala with different triggering mechanisms, i.e., earthquake,

rapid snow melt and heavy rainfall, Malamud et al. (2004) suggested a three-parameter inverse-gamma distribution to represent the frequency of occurrence of a given landslide area. The area distribution of ALDs at “Hot Weather Creek” and the one suggested by Malamud et al. are compared in Fig. 3. The two distributions show major differences in case of smaller landslide areas and different mean and standard deviation.

Fig. 3. Landslide area distributions of active-layer detachments in permafrost region (Hot Weather Creek) and some global landslides from Malamud et al. (2004).

Although “Hot Weather Creek” averages were derived using a relatively restrained number of ALDs (146 ALDs) when compared to those of Malamud et al., due to the peculiar triggering mechanism and shallow depths it is assumed that the mean and the standard deviation for the ALD area are statistically representative. As evidence, the medians of ALD width and length for the locations of “Black Top Creek”, “Hot Weather Creek”, and “Big Slide Creek” reported by Lewkowicz (1990) are compared in Table 2 with those presented later by Lewkowicz and Harris (2005) based on the updated data base. As can be seen, the increase of the number of the landslides contributes to only slight decrease of the medians of the width and length.

Lewkowicz (1990), and Lewkowicz and Harris (2005) also reported statistical parameters for the ALD normalized scar length ($\bar{L}_S = L_S/L$) for the same three study areas in northern Canada. Based on the pre-year 1989 data, the normalized scar length varied between 5 to 80% and was correlated to particle size distribution of the active layer material. It was concluded that ALD results in shorter scar zones in fine-grained soil when compared to sand-size material. The mean values of the normalized scar length for the fine-grained and sandy soils were 33% and 53%, respectively. Based on the pre-year 2000 updated data, medians of the normalized scar lengths are only slightly different: 50% for the sites covered by fine-grained soils (including a site in Mackenzie Valley) and 35% for the site with sandy soil.

Considering the lower and upper bounds as well as the means and medians of the normalized scar length, it appears that beta distribution appropriately represents the variations. For the random variable $\bar{L}_S$ in the range between 0.05 and 0.80, the beta distribution can be given by (Ang and Tang 2007):

$$(3) \quad f_{\bar{L}_S} = \frac{1}{B(q,r)} \frac{(\bar{L}_S - 0.05)^{q-1} (0.80 - \bar{L}_S)^{r-1}}{0.75^{q+r-1}}$$

where, q and r are the parameters and B is the beta function. Assuming a symmetric distribution (skewness=0), $q=r$, the mean and the variance are:

$$(4) \quad \mu = 0.425, \sigma^2 = \frac{0.141}{2q+1}$$

A mean value of 0.425 falls well between the reported 0.33 and 0.53, and represents the overall average value for both fine-grained and sandy soils. Thus, the scar length can be calculated as:

$$(5) \quad L_S^{st} = \bar{L}_S \cdot L^{st}$$

where, L_S^{st} and L^{st} are the scar length and ALD length, respectively. The superscript “st” stands for ALDs driven by static forces. For earthquake-induced ALDs, the scar length (L_S) and the length (L) are calculated as a sum of the displacements caused by both static and dynamic forces:

$$(6) \quad L_S = L_S^{st} + L_S^{dy}$$

$$(7) \quad L = L^{st} + L^{dy}$$

where, L_S^{dy} is the scar displacement caused by dynamic forces only.

Assessment of ALD deformations

Standardised methods for determining scar length of earthquake-induced ALDs are discussed herein. Two different mechanisms govern earthquake triggered slope instability: weakening of the soil shear strength such that it cannot resist earthquake-induced stresses (weakening instability), and generation of inertial deformations that cause failure in the soil (inertial instability) (Kramer 1996). Depending on the type of the instability that takes place, i.e., weakening or inertial, a different approach for estimation of the PGD is applied. Weakening instabilities are investigated

using models that account for the effect of excess pore water pressure on the shear strength of soil. On the other hand, inertial instabilities are usually simulated using the analogy of the behaviour of a soil mass with that of a block sliding on an inclined surface (Newmark 1965). In this study, flow failure and lateral spreading are considered as consequences of weakening instabilities.

Weakening instabilities

The geologic history of soil deposits may roughly determine whether they can be considered as susceptible to liquefaction. The surficial soils in the Mackenzie Valley include till, lacustrine, glaciofluvial, colluvial and alluvial fine-grained sediments (Aylsworth et al. 2000) deposited during the last continental Pleistocene glaciation (more than 10,000 years ago) (Monroe and Wicander 1992; Duk-Rodkin and Lemmen 2000). When fully saturated, these unconsolidated sediments show low to moderate susceptibility to liquefaction (Youd and Perkins 1978).

In addition to the geologic criteria, the geotechnical properties should be considered as well in assessing the liquefaction potential. Boulanger and Idriss (2006) categorized fine-grained soils according to their plasticity index (PI) to soils that exhibit clay-like ($PI \geq 7$) and sand-like ($PI < 7$) behaviour. The former group is essentially not liquefiable, whereas the latter can be liquefied. Limited information, however, was found in the literature: the Atterberg limits of samples collected from the proximity of thaw front in some ALDs of the Fosheim Peninsula and Mackenzie Valley reveal low to medium plasticity with PI in the range between 5 and 30 (Lewkowicz and Harris 2005); Wang et al. (2005) reported silty clay and clayey silts as the most common soil type within a 20 km-wide corridor east of the Mackenzie River. Although this dataset is not representative for the whole region, it provides an insight in the general soil properties for the liquefaction study. During the 2002 $M_w 7.9$ Denali earthquake, Alaska, extensive liquefaction was observed in fine-grained soils of Mabel Creek area with average PI of 5.3 and standard penetration test (SPT) values of 4 to 12 (Zhang 2009). According to this limited information, it can be concluded that in northern regions

unfrozen low-plasticity clayey silts with $5 \leq PI \leq 7$ can show liquefaction potential. Therefore, study of the likelihood of “weakening instability” is incorporated in this study.

After checking geologic and compositional criteria for liquefaction susceptibility, to represent the triggering conditions, a factor of safety against weakening instability (FS_w) is defined as:

$$(8) \quad FS_w = \frac{CRR}{CSR}$$

where, CRR is the cyclic resistance ratio that characterizes the soil resistance against liquefaction and CSR is the cyclic stress ratio. Several assumptions are made to solve Equation (8):

- CRR is obtained using SPT results (Seed et al. 1985) that are normalized to overburden pressure of 1 ton/ft² and hammer efficiency of 60%, presented as $(N_1)_{60}$. In this study, based on Seed et al. (1985) and the recommendations of the 1996 NCEER workshop (Youd et al. 2001), the following simplified relationship between CRR and normalized SPT values of clean sand, $(N_1)_{60CS}$, for $M_w 7.5$ earthquakes is developed:

$$(9) \quad CRR_{M_w 7.5} = \begin{cases} 0.05 & (N_1)_{60CS} \leq 5 \\ 0.0117(N_1)_{60CS} - 0.0083 & (N_1)_{60CS} > 5 \end{cases}$$

This relationship, shown in Fig. 4, is for clean sands rather than those with fines content.

- The effect of fines content on the $(N_1)_{60}$, studied by Idriss and R. B. Seed, is considered applying the following corrective equation (Youd et al. 2001):

$$(10) \quad (N_1)_{60CS} = \alpha + \beta(N_1)_{60}$$

where, α and β are functions of fines content (Table 3).

Fig. 4. Simplified relationship of the cyclic resistance ratio (CRR) in $M_w 7.5$ earthquakes and SPT results, developed for this study based on Seed et al. (1985), and recommendation of NCEER workshop (1996), published by Youd et al. (2001).

- The effects of earthquake magnitude other than $M_w 7.5$, soil plasticity and terrain slope on the final resistance against liquefaction are accounted for using corresponding correction factors (Youd et al. 2001):

$$(11) \quad CRR = C_m \cdot C_p \cdot C_s \cdot CRR_{M_w 7.5}$$

where, C_m , C_p and C_s are the correction factors for earthquake magnitude, soil plasticity and terrain slope, respectively (Table 3).

- CSR, which actually represents the equivalent harmonic shear stress to the liquefaction triggering earthquake-induced cyclic stresses, was presented by Seed and Idriss (1971) as:

$$(12) \quad \text{CSR} = 0.65 \left(\frac{\sigma_v}{\sigma'_v} \right) \text{PGA} \cdot r_d$$

where, PGA is the peak ground acceleration (fraction of g), r_d is the reduction factor for depth (Table 3), σ_v and σ'_v are the total and effective vertical stresses at the depth where liquefaction is being studied.

Both flow failure and lateral spreading are weakening instabilities that may result from liquefaction. When $\text{FS}_w < 1$, Equations (6) and (7) apply to compute the scar length (L_s) and the total length (L) in both cases. However, which mechanism will be triggered depends mainly on the sloping angle:

- For small sloping angle of $\theta < 6^\circ$, the lateral spreading represents the governing failure mechanism. The corresponding maximum displacement can be estimated with the empirical expression proposed by Youd et al. (2002) and developed for gently sloping terrains (without free-face):

$$(13) \quad \log D_H = -16.213 + 1.532M_w - 1.406 \log(R + 10^{0.89M_w - 5.64}) - 0.012R + 0.338 \log S + 0.540 \log T_{15} + 3.413 \log(100 - F_{15}) - 0.795 \log(D50_{15} + 0.1)$$

where, M_w is the earthquake moment magnitude, R is the earthquake source-to-site distance (km), S is the ground slope (%), T_{15} is the total layer thickness (m), F_{15} is the average fines content (%), and $D50_{15}$ is the average mean grain size of the granular soil layer with $(N_1)_{60} < 15$ in millimeters. A standard deviation equal to 0.197 for $\log D_H$ is reported by Gillins and Bartlett (2013). For lateral spreading $L_s^{\text{dy}} = 0$, $L_s^{\text{st}} = D_H$, and L^{st} is calculated from the lognormal distribution with parameters given in Table 1.

- For higher sloping angles, $\theta > 6^\circ$, the flow failure mechanism is triggered. In this case, L_S^{st} is calculated from Equation (5), L^{st} is found in a similar way to the lateral spreading case and $L_S^{dy} = 0$.

Inertial instabilities

The inertial earthquake-induced slope deformations can be separated into three different types of deformations (Ambraseys and Srbulov 1995): (i) co-seismic deformations, which occur during the ground shaking as a function of the earthquake magnitude and duration, geometry of slope and undrained mobilized strength at the slip surface; (ii) post-seismic deformations triggered immediately after the end of the ground shaking, provided that the factor of safety against inertial instability (FS_i) at the end of the co-seismic stage is smaller than 1. Here, only gravity drives the block, whereas the mobilized undrained residual strength of the slip surface resists against the motion and this continues until $FS_i > 1$; and (iii) indirect deformations caused by phenomena such as creep, consolidation processes and redistribution of pore pressures as the developed ground cracks are filling in with water. They may occur immediately or slightly after the first or the second types of deformations.

The co-seismic deformations can be estimated using the Newmark's sliding block approach assuming rigid body behaviour. During the ground shaking, acceleration may exceed critical levels for the potential slip surface and the block will experience permanent deformation. It is obtained by summing up the double integrals of the acceleration time history over the duration of the exceedance time, also referred to as the Newmark displacement. Several regression models have been proposed in the literature to facilitate the computation. These models correlate Newmark displacement (D_N) to critical acceleration of the slope (a_c) and to ground motion parameters, such as the PGA, Arias intensity and moment magnitude. As an example, Jibson (2007) derived the following equation based on 875 Newmark displacements resulted from some worldwide strong motions:

325 (14) $\log D_N = 2.401 \log I_a - 3.481 \log a_c - 3.230 \pm \sigma$

326 where, D_N is in cm, I_a is the Arias intensity in m/sec, a_c is in terms of g, and $\sigma = \pm 0.656$ represents
 327 the standard deviation of the model. Equation (14) allows for site-consistent Arias intensity
 328 attenuation models, such as those developed by Wilson and Keefer (1985), and Travarasrou et al.
 329 (2003), to correlate I_a and D_N to the earthquake magnitude and the source-to-site distance. In this
 330 study, equation developed by Wilson and Keefer that has fewer input parameters is used:

331 (15) $\log I_a = M_w - 2 \log R - 4.1$

332 where, I_a is in m/sec.

333 Based on the equation of Ambraseys and Menu (1988), Jibson (2007) also presented another
 334 expression for D_N that is applicable to $5.3 \leq M_w \leq 7.6$:

335 (16) $\log D_N = -2.710 + \log \left[\left(1 - \frac{a_c}{PGA} \right)^{2.335} \left(\frac{a_c}{PGA} \right)^{-1.478} \right] + 0.424 M_w \pm \sigma$

336 where, $\sigma = \pm 0.454$.

337 By considering the parallel- and perpendicular-to-slope components of horizontal earthquake
 338 acceleration in the calculations (as shown in Fig. 5) and setting the pseudo-static factor of safety of
 339 the infinite slope equal to 1, the critical acceleration (a_c) can be calculated in terms of g as:

340 (17) $a_c = \frac{\tan \theta}{1 + \tan \theta \tan \phi'} (FS_I - 1)g$

341 where, θ is the inclination angle and ϕ' is the effective friction angle. Using the limit equilibrium
 342 conditions, FS_I for infinite shallow slope is defined by:

343 (18) $FS_I = \frac{c' + \{H[(1-m)\gamma + m\gamma_{sat}] \cos \theta - u\} \tan \phi'}{H[(1-m)\gamma + m\gamma_{sat}] \sin \theta}$

344 where, γ and γ_{sat} are the bulk and saturated unit weights of soil, c' is the effective cohesion, H is the
 345 thickness of thawed active layer, u is the pore water pressure, and m indicates the saturated
 346 portion of the active layer's depth measured from the interface of the active layer with the
 347 permafrost table (the potential slip surface). The parameters c' , ϕ' and u should be measured at the
 348 location of the potential slip surface. These parameters are shown in Fig. 5.

Fig. 5. Infinite thawed slope in cold regions with related parameters and the corresponding Newmark's sliding block model subject to horizontal earthquake acceleration, $a_g(t)$.

The Newmark displacement can be used as a susceptibility index for the prediction of landslide likelihood after calibration against observed landslides (Jibson 2011). Jibson et al. (2000) compared the Newmark displacements from the 1994, $M_w 6.7$ Northridge earthquake with the triggered landslides, and presented the probability of slope instability. The fitted Weibull distribution shows that the probabilities of failure are 0.45, 0.83 and 0.96 for Newmark displacements less than 5, 10 and 15 cm, respectively. It can therefore be concluded that the majority of the landslides occur when the Newmark displacement is less than 15 cm and this value can be considered as a threshold Newmark displacement. Other threshold displacements proposed in the literature are given in Table 4.

The post-seismic deformations are larger compared to the co-seismic deformations and their magnitude depends on the local site conditions such as slope inclination angle and undrained residual shear strength at the slip surface. Separation of the co-seismic from the post-seismic deformations in the field is often difficult and so is the validation of analytical models against the observed field deformations. Since the post-seismic movements have similar kinematic conditions to the deformations of non-seismically triggered ALDs, the available records of fire- and meteorological-triggered ALDs presented in Section 3 may be used as a substitute. Thus, in the case of inertial instabilities, $L_S^{dy} = D_N$ and L^{st} is obtained from the lognormal distribution with parameters given in Table 1. When D_N is smaller than the threshold $L_S^{st} = 0$ and when D_N is larger than the threshold, L_S^{st} is not zero and should be calculated from Equation (5). Equations (6) and (7) should be used to find the total scar length (L_S) and the total ALD length (L).

Since the soil shear strength during and after the earthquake is the key parameter in determining the type of instability, the occurrence of "weakening" mode is verified first. If the active layer was

not susceptible to liquefaction, the “inertial” mode is investigated then. According to this logic, the flowchart shown in Fig. 6 summarizes the successive steps for estimating the earthquake-induced ALD deformations (scar length).

Fig. 6. Flowchart showing the procedure of earthquake-induced ALD scar length calculations.

Effect of pore water pressure

Two thaw conditions can be considered in this model to calculate the pore water pressure: slow and rapid. Under the slow thawing condition, no pore pressure is assumed to be produced in excess of the hydrostatic pressure. The pore water pressure u in Equation (18) is then simply computed for the saturated portion of the active layer:

$$(19) \quad u = mH\gamma_w \cos \theta$$

Under the rapid thawing condition, on the other hand, excess pore pressure is generated as a result of “thaw-consolidation”. The thaw-consolidation is a phenomenon exclusive to fine-grained ice-rich soils in cold regions when thawing rate of the active layer is faster than drainage and consolidation rates. It can cause slope instabilities for angles smaller than those predicted by the classic slope stability theories. The rapid thawing usually occurs as a result of forest fire- or construction-caused loss of surface vegetation and heavy rainstorms (Dyke 2004; Lewkowicz and Harris 2005). Morgenstern and Nixon (1971) developed a thaw-consolidation model combining Terzaghi’s linear consolidation theory and Neumann’s one-dimensional melting solution with the resulting excess pore water pressure given as:

$$(20) \quad \Delta u = \frac{mH(\gamma_{sat} - \gamma_w) \cos \theta}{1 + \frac{1}{2R_{tc}^2}}$$

where, γ_w is the unit weight of water and R_{tc} is the thaw-consolidation ratio between the input and output water in the thawing ground system defined by:

$$(21) \quad R_{tc} = \frac{\alpha_h}{2\sqrt{c_v}}$$

where, α_h is a heat conductivity-related constant and c_v is the coefficient of consolidation of the thawing soil. Substituting Equation (21) into Equation (20), the total pore water pressure applied in Equation (18) under the rapid thawing condition is obtained as:

$$(22) \quad u = mH \left[\gamma_w + \left(\frac{2R_{tc}^2}{2R_{tc}^2 + 1} \right) (\gamma_{sat} - \gamma_w) \right] \cos \theta$$

More details about the thaw-consolidation model of Morgenstern and Nixon and its related parameters are given in Morgenstern and Nixon (1971). According to McRoberts (1975), α_h is likely to fall in the range of 0.1 to 1.0 mm/sec^{0.5}. For the Norman Wells pipeline project, the respective c_v values of 0.0025 and 0.01 cm²/sec for ice-rich clay and till have been used (Hanna and McRoberts 1988). Paudel and Wang (2010) obtained c_v in the range between 0.01 and 0.06 cm²/sec after a number of freeze-thaw cycles in fine-grained soil samples from the Mackenzie Valley.

Ground motion parameters

The seismic ground motion parameters are necessary for evaluating CRR (Equation 11) and CSR (Equation 12). The ground motion parameters are typically defined employing an attenuation relationship often referred to as ground motion prediction equation (GMPE) consistent with the location of the study area. Consequently, the source-to-site distance, R , in the attenuation relationship should be consistent with those that are used in this study, i.e. in Equations (13) and (15). In this study, R is defined as the closest horizontal distance of site to the vertical projection of the fault rupture plane. The GMPE applicable to the Western North America (WNA) developed by Boore et al. (1997) is adopted in this study. This relationship considers different local site conditions defined with the average shear wave velocity of the top-30 m (V_{s30}) and has a standard deviation equal to 0.468 for $\ln(\text{PGA})$. It is assumed herein that $V_{s30}=620$ m/sec, which represents average soil condition within soil class C, dense soil to soft rock ($V_{s30}=360\text{-}760$ m/sec) (National Building Code of Canada 2010). Plots of PGA attenuation with distance corresponding to different values of M_w are shown in Fig. 7.

Fig. 7. Boore et al. (1997) PGA attenuation used in this study. R is the closest horizontal distance of site to the vertical projection of the fault rupture plane. $\ln(PGA)$ has a standard deviation of 0.468 and median values of PGA are plotted here.

Probability of exposure and Monte Carlo simulations

To determine the probability of exposure, i.e., the probability that a linear infrastructure is exposed to a landslide as defined with Equation (2), and to predict the extent of PGD, Monte Carlo simulations were performed using a program developed in MATLAB (The MathWorks, Inc. 2011). Table 5 summarizes the input variables along with the corresponding distributions and their statistical parameters. The mean values and coefficients of variation (COV) were assumed based on the values reported in the literature and the guidelines of Phoon and Kulhawy (1999). In Table 5, α represents the direction randomness of the horizontal earthquake excitation.

Cross-correlation coefficients among the soil properties are site-dependent and are rarely reported in the literature. For this study, only the variables ϕ' and c' were treated as dependent variables, whereas the other input parameters were assumed as independent. Uzielli et al. (2007) proposed a correlation coefficient between effective friction angle and effective cohesion in the range of -0.75 to -0.25, which can be used for practical applications in the absence of site-specific information.

The Venn diagram of the sample slopes generated by Monte Carlo technique is shown in Fig. 8. As can be seen, part of the samples with negative index of exposure (I_E^-) belong to stable slopes, whereas the other part that belongs to unstable slopes include those landslides with runout zones that do not cross the infrastructure axis. Using the Venn diagram and concentrating on exposure events with $E=I_E^+$ (Section 2), the probabilities of weakening and inertial instabilities can be defined as $P(WI|E)$ and $P(II|E)$, respectively.

Fig. 8. Venn diagram of the slope samples generated by Monte Carlo technique.

Effect of the four parameters: distance of infrastructure axis to scar crown, S , thaw-consolidation ratio, R_{tc} , earthquake magnitude, M_w , and source-to-site distance, R , on the probability of exposure,

P(E), and PGD was investigated by Monte Carlo simulations. In each parametric study, a given parameter was assigned three different values (low, average, and high) while the other parameters remained constant and equal to their average value (except for R_{tc} that was kept in its low level, i.e., the slow thawing condition). Typical frequency density plots of I_E and PGD for a Monte Carlo simulation with average parameters (i.e., $S=80$ m, $R_{tc}=0$, $M_w=6.5$, and $R=40$ km) are shown in Fig. 9.

Fig. 9. Typical frequency density plots of (a) the index of exposure (I_E), and (b) permanent ground deformations (PGD) obtained from Monte Carlo simulations ($S=80$ m, $R_{tc}=0$, $M_w=6.5$, and $R=40$ km).

According to Equation (2) and considering Fig. 8, the probabilities of exposure, $P(E)$, can be calculated (Table 6). As can be seen in Table 6, $P(E)$ is sensitive to S , R_{tc} , M_w , and R , which is shown graphically in Fig. 10. The probabilities of exposure, $P(E|S)$, $P(E|R_{tc})$, $P(E|M_w)$, and $P(E|R)$, are normalized with respect to the probability of exposure for the case with the average parameters, P_0 (Fig. 10). Extra points were added to data in Table 6 to obtain smoother curves. Fig. 10a demonstrates that the asymptotic decrease of S toward 0 increases the probability of exposure up to 7 times compared to the average case. As well, the increase of thaw-consolidation ratio from 0 to 3, has resulted in up to 12-time larger probabilities of exposure (Fig.10b). The sensitivity to magnitude-distance parameters indicates that the increase of M_w and decrease of R in the range of their potential values may considerably escalate the probability of exposure up to 11 and 13 times, respectively (Figs 10c and d).

The weakening of the soil shear strength was identified as the main factor of instability in the events with I_E^+ (Table 6). As well, the relatively small variation of the mean and COV of the PGD extent suggest that the statistical distribution of the ground deformations can be regarded as independent of the considered slope parameters, soil type and ground motion (Table 5). For all studied combinations, the excellent fit of the Weibull distribution with the results was confirmed with the K-S goodness-of-fit tests. The corresponding shape and scale factors of the Weibull distribution are given in Table 6.

Fig. 10. Sensitivity of the probability of exposure to (a) distance of infrastructure axis to scar crown, S , (b) thaw-consolidation ratio, R_{tc} , (c) earthquake magnitude, M_w , and (d) source-to-site distance, R . The probabilities are normalized with respect to the base case with average parameters (i.e., $S=80$ m, $R_{tc}=0$, $M_w=6.5$, and $R=40$ km) shown with red dotted lines.

Summary and conclusions

The probability that a linear infrastructure is exposed to the peak ground deformation (PGD) of earthquake-triggered active-layer detachment (ALD) in permafrost regions was investigated. Two mechanisms were assumed for material transfer: translation and compression. The extent of the PGD along ALD runout zone was determined next. The probability of exposure was determined applying Monte Carlo simulation combined with statistical distribution representing the distance between the scar crown of ALD and the axis of linear infrastructure and the computed length of the earthquake-triggered ALD. An algorithm was developed considering soil weakening and inertial instabilities triggered by earthquakes. The effects of the distance of infrastructure axis to scar crown, thaw-consolidation ratio, earthquake magnitude and source-to-site distance on the probability of exposure were studied. The results show that the distance of scar crown to infrastructure axis has a major influence on the exposure. The effect of thaw-consolidation phenomenon was investigated and it was shown that the existence of increased pore water pressure prior to an earthquake can increase the probabilities of exposure and weakening instabilities; large magnitude earthquakes and short source-to-site distances have similar effects on the weakening instabilities. It was assumed that the earthquake-induced PGDs to infrastructure follow the Weibull distribution. The scale and shape factors of the distribution were determined and it was observed that they have large coefficient of variations. The accuracy of the results obviously depends on the quality of input parameters and the assumptions made in the study. To decrease uncertainties, future research topics should include the following:

- Study of the material transfer mechanisms in ALD and determine the conditions under which each of the mechanisms occurs.

- Monitor the behaviour of the potential unstable slopes in the permafrost region and perform post-seismic investigations.

- Determine distribution of the potential ALD locations along linear infrastructure routes.

- Improve the accuracy of the input parameters of the soil and slope by performing detailed geotechnical and geological site investigations.

Acknowledgements

Funding was provided in part through the Canadian Safety and Security Program (CSSP) led by Defence Research and Development Canada's Centre for Security Science, in partnership with Public Safety Canada. Additional support was provided by the Natural Sciences and Engineering Research Council of Canada (NSERC) as grant in-aid to the second author.

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Figure Captions

Fig. 1. Mechanisms of material transfer and distribution of the transverse PGD for: (a) translational, and (b) compressional movement. L represents the total landslide length, L_s is the scar zone length and S is the distance of scar crown to pipeline axis.

Fig. 2. Top view of a hypothetical linear infrastructure segment exposed to potential transverse active-layer detachments (ALD) and the relevant parameters: ALD width (W) and length (L), length of scar zone (L_s) and distance of scar crown to infrastructure axis (S). ALDs that impact the infrastructure are shown in grey.

Fig. 3. Landslide area distributions of active-layer detachments in permafrost region (Hot Weather Creek) and some global landslides from Malamud et al. (2004).

Fig. 4. Simplified relationship of the cyclic resistance ratio (CRR) in $M_w 7.5$ earthquakes and SPT results, developed for this study based on Seed et al. (1985) and recommendation of the 1996 NCEER workshop published by Youd et al. (2001).

Fig. 5. Infinite thawed slope in cold regions with related parameters and the corresponding Newmark's sliding block model subject to horizontal earthquake acceleration, $a_g(t)$.

Fig. 6. Flowchart showing the procedure of earthquake-induced ALD scar length calculations.

Fig. 7. Boore et al. (1997) PGA attenuation used in this study. R is the closest horizontal distance of site to the vertical projection of the fault rupture plane. $\ln(PGA)$ has a standard deviation of 0.468 and median values of PGA are plotted here.

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Table 1. Statistical parameters of active-layer detachments at “Hot Weather Creek” site, Fosheim Peninsula, estimated based on Lewkowicz (1990).

ALD parameter	μ_{ln}	σ_{ln}	Median	Mean	Standard deviation
Width (m)	2.284	0.707	9.8	12.6	10.1
Length (m)	3.420	0.811	30.6	42.5	41.0
Length/Width	1.136	1.076	3.1	5.6	8.2
Area (m ²)	5.704	1.076	300.1	535.3	790.9

Note: μ_{ln} and σ_{ln} are the lognormal distribution parameters.

Table 2. ALD width and length medians at three locations on the Fosheim Peninsula: “Black Top Creek” (BTC), “Hot Weather Creek” (HWC), and “Big Slide Creek” (BSC).

	Width (m)			Length (m)			Number of ALDs		
	BTC	HWC	BSC	BTC	HWC	BSC	BTC	HWC	BSC
Pre-year 1989 median*	23	10	15	54	31	55	217	146	148
Pre-year 2000 median†	20	10	13	42	30	38	237	159	191

* Estimated from data of Lewkowicz (1990).

† Reported by Lewkowicz and Harris (2005).

Table 3. Correction factors used in the estimation of factor of safety against liquefaction in this study.

Correction factor	Formula	Range of parameters	Reference
Fines content (α)	0	$FC \leq 5\%$	Youd et al. (2001)
	$\exp[1.76 - (190/FC^2)]$	$5\% < FC < 35\%$	
	5	$FC \geq 35\%$	
Fines content (β)	1	$FC \leq 5\%$	Youd et al. (2001)
	$0.99 + (FC^{1.5}/1,000)$	$5\% < FC < 35\%$	
	1.2	$FC \geq 35\%$	
Depth (r_d)	$1 - 0.00765z$	$z < 9.15 \text{ m}$	Liao and Whitman (1986)
Magnitude (C_m)	$10^{2.24/M_w^{2.56}}$	$5^{1/2} \leq M_w \leq 8^{1/2}$	Youd et al. (2001)
Plasticity (C_p)	1	$PI \leq 10$	Ishihara (1993)
	$1 + 0.022(PI - 10)$	$PI > 10$	
	$-x + 1$	$D_r \approx 35\%$	
Slope (C_s)	1	$D_r \approx 40\%$	Developed based on Kavazanjian Jr et al. (1997)
	$1.9x + 1$	$D_r \approx 45 - 50\%$	
	$2.9x + 1$	$D_r \approx 55 - 70\%$	

Note: FC and PI stand for fines content and plasticity index, respectively, and $x = \tau_h/\sigma'_v$.

Table 4. Newmark displacement threshold values (Jibson 2011).

Reference	Threshold D_N (cm)	Target location	Remarks
Wieczorek et al. (1985)	5	San Mateo County, California	
Keefer and Wilson (1989)	10	Southern California	For coherent landslides.
Jibson and Keefer (1993)	5 to 10	Mississippi Valley	
Jibson et al. (2000)	2 to 15	Northern San Fernando Valley and Santa Susana Mountains	For shallow, disrupted rock falls and rock slides in fairly brittle, weakly cemented sediments.
Blake et al. (2002)	5 or 15	Southern California	Depends on slope conditions and soil properties.
California Geological Survey (2008)	0 to 15		Unlikely to be damaging.
	15 to 100	California	Enough serious to be damaging.
	Greater than 100		Very likely to be damaging.
	0 to 1		Low hazard level (shallow landslide).
Jibson and Michael (2009)	1 to 5	Anchorage, Alaska	Moderate hazard level (shallow landslide).
	5 to 15		High hazard level (shallow landslide).
	Greater than 15		Very high hazard level (shallow landslide).

Table 5. Input variables for Monte Carlo simulations.

Variable		Probabilistic			Deterministic	Remarks
		Mean	COV (%)	Distribution		
Slope:	θ (Deg)	7	30	Lognormal	-	-
	H (m)	1.0	30	Lognormal	-	-
	m	0.75	5	Beta	-	$0.5 \leq m \leq 1.0$
	S (m)	50, 80, 110	50	Lognormal	-	-
Soil:	γ_d (kN/m ³)	16	9*	Lognormal	-	-
	c' (kPa)	2.5	20*	Lognormal	-	Cross-correlated to ϕ'
	ϕ' (Deg)	26	10*	Lognormal	-	Cross-correlated to c'
	$(N_1)_{60}$	5	45*	Lognormal	-	-
	FC (%)	-	-	-	70	-
	PI (%)	15	40	Beta	-	$5 \leq PI \leq 30$
	T_{15} (m)	1.0	30	Lognormal	-	$T_{15} = H$
	F_{15} (%)	-	-	-	70	$F_{15} = FC$
	$D_{50_{15}}$ (mm)	0.01	60	Lognormal	-	-
	D_r (%)	-	-	-	40	-
	R_{tc}	-	-	-	0.0, 1.5, 3.0	-
	M_w	-	-	-	5.5, 6.5, 7.5	-
Ground motion:	R (km)	-	-	-	10, 40, 80	-
	α (Deg)	45	-	Uniform	-	$0 \leq \alpha \leq 90$

*Based on the guidelines of Phoon and Kulhawy (1999).

Table 6. Results of Monte Carlo simulations and the Weibull distribution parameters corresponding to the obtained PGDs.

	S (m)			R_{tc}			M_w			R (km)		
	50	80	110	0	1.5	3	5.5	6.5	7.5	10	40	80
P(E) (%)	0.16	0.08	0.05	0.08	0.72	0.98	0.01	0.08	0.90	1.05	0.08	0.01
P(WI E) (%)	99.7	99.7	99.9	99.7	94.3	89.6	91.1	99.7	100.	99.9	99.7	98.2
P(II E) (%)	0.34	0.30	0.15	0.30	5.73	10.5	8.93	0.30	0.00	0.10	0.30	1.81
PGD mean (m)	13.7	16.0	17.2	16.0	13.8	14.7	17.2	16.0	15.6	15.3	16.0	14.9
PGD COV (%)	170	177	197	177	177	190	198	177	180	179	177	184
Shape factor	1.56	1.70	1.50	1.70	1.05	1.20	1.42	1.70	3.13	2.47	1.70	0.70
Scale factor	0.22	0.22	0.21	0.22	0.21	0.21	0.21	0.22	0.29	0.26	0.22	0.18

Note: E, WI and II are the events of exposure, weakening instability, and inertial instability, respectively.