Preface

These Proceedings represent the papers prepared and published as part of the Technical Program for the 2021 Regional Conference on Permafrost and 19th International Conference on Cold Regions Engineering. The U.S. Permafrost Association is co-organizing the Conference with the American Society of Civil Engineers. The goal of the Conference organizers was to provide a venue for permafrost science and frozen ground engineering topics to be meaningfully merged together for the first time. Increasingly, basic research and applied engineering efforts must incorporate a broad variety of information to ensure project results. From carbon itemization to bridge design, project deliverables must have meaningful and broad impact. Many of the difficult challenges facing permafrost researchers and engineers are the result of ongoing and projected future climate warming in earth’s cold regions. To adequately address this challenge, we must find ways to cross disciplines and share best practices and lessons learned.

These Proceedings include papers dealing with Permafrost Detection, Monitoring, and Change; Mountain Permafrost and Rock Glaciers; Cold Regions Transportation Infrastructure; Impacts of Permafrost on Engineered Structures; and Permafrost Issues Experienced by the Trans Alaska Pipeline System (TAPS). Authors submitted Draft Papers based on accepted abstracts. Each Draft Paper received two peer reviews which guided authors in preparing their Final Papers.
Acknowledgments

The U.S. Permafrost Association and the American Society of Civil Engineers are acknowledged for their efforts in advertising and promoting the Conference including reaching out to their members and colleagues to offer the opportunity to prepare a paper for the Proceedings. The authors of the papers are thanked for their efforts in preparing the papers and diligently responding to questions and comments of the peer reviewers. Great thanks goes out to all of the peer reviewers who provided comments to improve the quality of the submitted papers. The American Society of Civil Engineers is thanked for the final paper compilation and publication of these Proceedings.

The numerous sponsors, supporters, and committee members are also acknowledged for their great effort into assembling the most diverse and collaborative Technical program possible.

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Long-Term (2000–2017) Response of Lake-Bottom Temperatures and Talik Configuration to Changes in Climate at Two Adjacent Tundra Lakes, Western Arctic Coast, Canada

Trevor S. Andersen 1; Patrick A. Jardine 2; and Christopher R. Burn, D.Sc. 3

1Dept. of Geography and Environmental Studies, Carleton Univ., Ottawa, ON, Canada. E-mail: TrevAndersen@cmail.carleton.ca
2Dept. of Geography and Environmental Studies, Carleton Univ., Ottawa, ON, Canada. E-mail: PatJardine@cmail.carleton.ca
3Dept. of Geography and Environmental Studies, Carleton Univ., Ottawa, ON, Canada. E-mail: Christopher.Burn@carleton.ca

ABSTRACT

Lakes, commonly underlain by taliks, are principal agents of disturbance to permafrost. We have measured lake-bottom temperatures with submerged loggers on near-shore terraces and in deep central pools at two tundra lakes on Richards Island, NT, to determine inter-annual lake thermal responses to climate variation. We have modelled associated potential adjustments in talik geometry. In 2000–17, annual mean temperatures varied between -5.7 and 2.1°C for terraces and 1.1 and 4.5°C for pools. Permafrost in the terraces is warmer than surrounding the lakes: talik configuration varies with horizontal terrace extent and terrace and pool temperatures. The talik break-through depth declines as terrace size increases. Using the four warmest and coldest years as an analogue for climate change—an adjustment that may occur this century—the increase in talik depth may be up to 100 m, but it may take millennia for talik geometry to reach equilibrium.

INTRODUCTION

Tuktoyaktuk Coastlands, like Alaska’s North Slope, is a land of lakes (Fig. 1a) (Mackay 1992; Hinkel et al. 2012). Many of the lakes are large enough to generate through taliks, even though the continuous permafrost is 400 to 700 m thick (Judge et al. 1987; Burn 2002). The lakes typically have shallow littoral terraces extending 10s of m from the shore and a deep central pool (Fig. 1b). Permafrost is maintained within the terraces, though they are warmer than the ground surrounding the lakes (Burn 2002, 2005; O’Neill et al. 2020). The thermal regime of the lakes provides the greatest natural disturbance to permafrost temperatures from conditions determined by climate (Lachenbruch et al. 1962).

We have measured lake-bottom temperatures at two tundra lakes on Richards Island from 2000 to 2017. One of these lakes, informally named Todd Lake (Figs 1b, 2) (Burn 2002), consists of littoral terraces < 0.75 m deep that extend up to 130 m from the shore and a central pool up to 16 m deep. The other lake, nearby, is informally named Andrew Lake (Fig. 1b), and is the product of three smaller lakes coalescing (Burn 2005). The littoral terraces are smaller, and the central pool is shallower in this lake than in Todd Lake.

From 2000 to 2017, mean annual air temperature averaged -9.0 °C at Tuktoyaktuk, the regional weather station (Fig. 2). In 2003 to 2007, mean annual ground temperatures at the top of permafrost (~ 1 m depth) in undisturbed sites near the western Arctic coast were between -7.0 and -6.0 °C (Burn and Kokelj 2009). The climate has warmed relatively quickly in Canada’s western Arctic, with most of the warming in autumn and winter. At Tuktoyaktuk, the increase in annual mean air temperature from 2000 to 2017 was 0.12 °C/yr (R = 0.61, p = 0.007, n = 18). The warming began in the early 1970s (Burn and Kokelj 2009). Air temperature is well correlated through the
Mackenzie Delta area, so the warming is a regional phenomenon (Burn and Kokelj 2009, Table 1, Fig. 4). The consequences of climate warming for permafrost configuration beneath the lakes may take some time to be noticed, because the taliks are likely already well-developed, given the time since their formation in the Early Holocene (Rampton 1988).

Figure 1. (a) Aerial view of tundra lakes on northern Richards Island, NT. Image from near lake 5 of Fig. 2b. (b) Todd Lake, Richards Island. The lake has prominent littoral terraces (A) and a deep central pool (B). Figure 2 in Burn (2002) presents a bathymetric cross-section of the lake. The lakes lie in topographic depressions oriented by antecedent processes (Mackay 1963).

Figure 2. (a) Outer Mackenzie delta area, NT, with location of Richards Island and study sites. The red box identifies the area in Fig. 2b. The inset gives the position of the Mackenzie Delta area in western Arctic Canada. (b) Location of 12 lakes on Richards Island where lake-bottom temperatures were measured in late winter and summer (Fig. 15 in Burn 1997) The study sites are near Lake 5. Original cartography by H. B. O’Neill, adapted for this paper.

The thermal regime of tundra lakes in Tuktoyaktuk Coastlands (Fig. 1) may be divided into two or three periods, depending on the presence of surface ice (Fig. 3). During the ice-free summer, the lakes are well-mixed and of relatively uniform temperature (Burn 2002; Arp et al. 2016). In winter, ice forms to the lake-bottom above the shallower terraces, but not in the deeper central pools. The central pools experience higher and more consistent temperatures than the terraces.
Lake-bottom temperatures do not fall below 0 °C in the central pool of Todd Lake, but ice may reach the bottom in parts of the shallow pool at Andrew Lake especially in late winter (Fig. 7 in Burn 2005). The lakes become thermally stratified beneath the ice, with the highest temperatures recorded in the deepest water. Andrew Lake is sufficiently shallow to cool at all depths over winter, but the depth at Todd Lake and the association of water’s density with temperature below 4 °C ensure that heat flowing out of the talik warms the dense bottom water. Temperatures are sustained near 0 °C for a short period in late May and early June when the ice is thawing.

Following Burn (2002), this paper will: (1) demonstrate the inter-annual variability of the lake-bottom thermal regime; (2) determine the correspondence of variation in climate and lake-bottom temperature; (3) simulate the effect of changes observed in lake-bottom temperature on talik configuration; and (4) estimate the time required for such responses in talik configuration to occur. The paper also presents contextual data on regional lake thermal conditions (see Fig. 2b).

FIELD METHODS

Data presented in this paper comprise measurements of lake-bottom temperature made at five locations in Todd and Andrew lakes using miniature data loggers (Hobo H8 series with internal sensors, Onset Corp, Bourne, MA) sealed in watertight cannisters and attached to pillars on the shoreline using steel-cored, plastic-coated cables. The manufacturer-specified precision of measurements for these loggers is ±0.25 °C and the accuracy is ±0.5 °C including uncertainty due to precision. The cannisters were weighted to the lake-bottom with steel pipe couplings. At Todd Lake the loggers were positioned in depths of 0.5 m (terrace) and 10 m (deep pool); at Andrew Lake the water depths were 0.4 m (terrace), 1.2 m (shallow pool), and 2-4 m (central pool).

Figure 3. (a) Mean daily temperature at Todd Lake central pool (Deep) and terrace (Terrace), 1 July 2002 - 1 October 2003. (b) Mean daily temperature at Andrew Lake central pool (Deep), shallow pool (Shallow), and terrace (Terrace), 1 July 2002 - 1 October 2003. The zero curtain occurred in October and November for Todd Lake and October through February for Andrew Lake. The zero curtain is the period of near-zero temperatures at the shallower loggers due to latent heat released during phase change of water to ice. Air temperatures measured at the site are also shown.

The loggers were either placed on the lake terraces directly or dropped in the central pools from a helicopter on floats. The bathymetry of Todd Lake ensured the loggers were positioned at about the same depth in the central pool, but the central pool at Andrew Lake is shallower and more spatially variable. The loggers were retrieved annually in mid- to late August in most years.
and replaced on the same day. When loggers were not retrieved, data for two years were recovered the following year. Loggers were programmed to record temperature every four or six hours. In some years, the loggers were moved on the terraces by ice during breakup, and loggers from the deep pools were occasionally dragged up onto the terraces. Loggers were always replaced as close to the original location as possible, determined by the length of the cable and a bearing between the pillar on shore and a landmark across the lake. All data reported in this paper are available from: <https://doi.org/10.18739/A2NG4GS7W>.

Air temperature was measured at a site between the two lakes by a HOBO HTI -37+46 data logger with internal sensor, installed in a stack of white steel plates acting as a radiation shield. The logger was programmed to record air temperature five times per day (every 4.8 h). Data from the field site were used to obtain the relation between conditions there and at Tuktoyaktuk: \( T_{TL} = 0.97T_{TK} - 1.00 \) (\( R = 0.99, p <0.001, n = 1883 \)), where \( T_{TL} \) is the daily mean air temperature at Todd Lake and \( T_{TK} \) is the value for the same day at Tuktoyaktuk (Environment and Climate Change Canada 2020).

![Figure 4](image.png)

**Figure 4.** Lake-bottom temperature for 12 lakes on Richards Island (Fig. 2b) measured on 8-10 April 1995 and 12 August 1997, plotted against latitude. The decrease in lake-bottom temperature with latitude is statistically significant for the data collected in August (\( R = -0.95, p < 0.001, n = 12 \)), but not for late winter measurements (\( R = -0.55, p = 0.062, n = 12 \)).

**REGIONAL LAKE-BOTTOM CONDITIONS**

Lake-bottom temperature surveys (April, 1995, and late August, 1997) at 12 lakes from north to south across Richards Island (Figs 2b, 4) show that by the end of winter there may be no statistically significant difference in lake-bottom temperature over this region, but temperatures may systematically increase southwards in summer (Fig. 4). The warming of these lakes reflects the increase in air temperatures that occurs with distance from the coast, as observed on the Alaska North Slope (Haugen and Brown 1980; Burn 1997). In winter, the variation with latitude is confounded by the lake depths. When the data in Fig. 4 are used in a multiple regression with lake depth as an additional independent variable, both relations are statistically significant. For summer data, \( R = 0.97, p \) (latitude) < 0.001, \( p \) (depth) = 0.031, \( n = 12 \); for winter data, \( R = 0.80, p \) (latitude) = 0.018, \( p \) (depth) = 0.018, \( n = 12 \). Figure 4 shows that lake-bottom temperatures in the central pools on Richards Island are spatially relatively uniform and the annual range is about 12 °C.

Ice thickness on these lakes varies considerably inter-annually (Fig. 5a) and, in any year, due to variations in snow cover (Fig. 5b). Fig. 5a presents 18 measurements of ice thickness (mean =
175 cm, std = 23 cm, range = 85 cm) at Todd Lake that were made in April 1992-2018 at sites with minimal or no snow cover. Data are missing for the following years: 1996, 2000, 2006, 2008, 2011-14, and 2016. From 1992 to 2018 (excluding 1993 to 1995 due to incomplete data), annual mean air temperatures at Tuktoyaktuk increased from -10.1 to -8.3 °C (R = 0.40, m = 0.067, p = 0.054, n = 24). The linear relation between winter air temperature at Tuktoyaktuk and ice thickness on the lake is statistically significant. In all years surveyed, ice thickness was greater than water depth on the littoral terraces.

Figure 5. (a) Relation between late-winter (April) ice thickness at Todd Lake and mean air temperature at Tuktoyaktuk during the preceding November – March, for 1992 to 2018 (R = -0.71, m = -8.8 cm/°C, p = 0.001, n = 18). Measurements are from locations with <10 cm snow cover. (b) Relation between ice thickness and snow cover at 20 locations on Todd Lake, 8 April 2001. The datum for 2001 in Fig. 5a is (-23.7, 148).

Table 1. Average mean annual temperatures, thawing degree-days (TDD), and freezing degree-days (FDD) for each of six sensors in the study area, with n in years. Variability is the standard error of the mean. The central pool of Todd Lake does not cool below 0 °C so winter TDD is reported. FDD are not presented for the central pool of Andrew Lake, as freezing only occurred in some winters. Temperatures measured on or in: TLA (Todd Lake air), TLT (Todd Lake terrace), TLD (Todd Lake deep pool), ALT (Andrew Lake terrace), ALS (Andrew Lake shallow pool), and ALD (Andrew Lake deep pool).

<table>
<thead>
<tr>
<th>Location</th>
<th>Dates</th>
<th>MAT (°C)</th>
<th>n</th>
<th>TDD (°C d)</th>
<th>n</th>
<th>FDD (°C d)</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>TLA</td>
<td>2000-17</td>
<td>-9.6 +/- 0.2</td>
<td>17</td>
<td>870.3 +/- 37.1</td>
<td>16</td>
<td>4393.4 +/- 78.0</td>
<td>17</td>
</tr>
<tr>
<td>TLT</td>
<td>2000-17</td>
<td>-2.6 +/- 0.6</td>
<td>14</td>
<td>1039.8 +/- 39.9</td>
<td>14</td>
<td>1914.1 +/- 206.8</td>
<td>16</td>
</tr>
<tr>
<td>TLD</td>
<td>2000-17</td>
<td>3.7 +/- 0.1</td>
<td>12</td>
<td>1002.4 +/- 37.2</td>
<td>16</td>
<td>(373.3 +/- 100.0)</td>
<td>12</td>
</tr>
<tr>
<td>ALT</td>
<td>2001-10</td>
<td>-0.3 +/- 0.8</td>
<td>8</td>
<td>992.0 +/- 44.2</td>
<td>8</td>
<td>1048.8 +/- 242.9</td>
<td>9</td>
</tr>
<tr>
<td>ALS</td>
<td>2001-10</td>
<td>-0.1 +/- 0.6</td>
<td>8</td>
<td>965.0 +/- 41.9</td>
<td>8</td>
<td>1007.0 +/- 164.7</td>
<td>9</td>
</tr>
<tr>
<td>ALD</td>
<td>2001-10</td>
<td>2.3 +/- 0.5</td>
<td>5</td>
<td>951.2 +/- 23.1</td>
<td>5</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Ice thickness on Todd Lake was measured at 20 sites with varying snow cover in April 2001 (Fig. 5b). There exists a well-defined, but scattered relation between these two variables. The winter temperature for 2000-01 (-23.7 °C) was close to the mean for the period from 1992 to 2018 (mean = -23.3 °C, std. = 1.8 °C). In April 2001, the range in measured ice thicknesses was 36 cm
and the mean 132 cm. No systematic areal sampling of snow depth or ice thickness has been carried out on Todd Lake, so it is not possible to indicate the overall mean. However, from the data presented in Figs 5a and 5b, it is reasonable to infer that the interannual variation in ice thickness is greater than the annual spatial variation.

INTER-ANNUAL VARIATION IN LAKE-BOTTOM TEMPERATURES

Figure 3 indicates that lake-bottom temperatures are closely coupled to air temperatures in the summer due to the well-mixed, isothermal nature of tundra lakes (Burn 2002, 2005). Seasonally, thawing degree days (TDD) at the lake-bottom have similar variability as the atmosphere (Table 1). For all five lake-bottom temperature series, TDD are on average higher than in the atmosphere, because lake-bottom temperatures do not respond to short, cool, summer periods (Fig. 3). In winter, the lake-bottoms on the shallow terraces are systematically warmer than the atmosphere (Table 1) due to a prolonged zero curtain (Fig. 3), snow accumulation on the lake surface and heat supplied by relatively warm ground in the terraces (Burn 2002). Measured and reconstructed air temperatures at Todd Lake resulted in a mean temperature of -9.6 °C for 2000 to 2017, 0.6 °C lower than at Tuktoyaktuk. Central pool lake-bottom temperatures were between 11.9 and 13.4 °C higher, comparable to the 13 °C difference reported by Burn (2005). In the shallows, this difference was approximately 7 °C for Todd Lake, and 9 °C for Andrew Lake, also similar to data in Burn (2005). Mean annual temperatures increased with depth. Mean TDD on the lake bottom terraces varied by less than 10%, but freezing degree days (FDD) by 90%, indicating the sensitivity of terrace temperatures to factors such as snow accumulation and the length of the zero curtain.

CLIMATE VARIATION AND LAKE-BOTTOM TEMPERATURES

Relations between thawing degree-days in the air (TDD$_a$) and at the lake bottom are illustrated in Figs 6a and 6b for Todd and Andrew lakes, respectively. Key statistics are summarized in Table 2. All relations are statistically significant and have a correlation coefficient of at least 0.90, with the exception of data collected from the central pool of Andrew Lake (ALD). The slopes of these lines are generally around 0.90. The data for ALD were compromised by the variability in depth of logger placements within the range (2-4 m) where this is a material concern (see Fig. 7 in Burn, 2005). Relations for the terraces between freezing degree-days (FDD$_a$) in the atmosphere and lake
bottom were weak and not statistically significant with correlation coefficients no greater than 0.43 (see Table 3), primarily due to the factors described above.

Table 2. Relations between TDD at lake bottom and in the air at Todd and Andrew lakes. The relations are for the least-squares linear regression lines (Fig. 6) and are of the form $TDD_{lb} = mTDD_a + c$, where $TDD_{lb}$ and $TDD_a$ are, respectively, TDD at the lake bottom and in the air.

<table>
<thead>
<tr>
<th>Location</th>
<th>Dates</th>
<th>Relation</th>
<th>$R$</th>
<th>$p$</th>
<th>$n$</th>
<th>Missing years</th>
</tr>
</thead>
<tbody>
<tr>
<td>TLT</td>
<td>2001-16</td>
<td>0.92TDD_a + 251</td>
<td>0.92</td>
<td>&lt;0.001</td>
<td>14</td>
<td>2010, 11</td>
</tr>
<tr>
<td>TLD</td>
<td>2001-16</td>
<td>0.93TDD_a + 178</td>
<td>0.90</td>
<td>&lt;0.001</td>
<td>12</td>
<td>2004, 09, 15, 16</td>
</tr>
<tr>
<td>ALT</td>
<td>2002-09</td>
<td>0.91TDD_a + 260</td>
<td>0.93</td>
<td>0.002</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>ALS</td>
<td>2002-09</td>
<td>0.89TDD_a + 246</td>
<td>0.93</td>
<td>0.002</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>ALD</td>
<td>2002-09</td>
<td>0.47TDD_a + 585</td>
<td>0.80</td>
<td>0.133</td>
<td>5</td>
<td>2005, 06, 07</td>
</tr>
</tbody>
</table>

The measurements of lake-bottom temperature indicate that conditions in deep central pools are resilient to changes in winter climate because the thermal regimes are disconnected in this season. Over 12 winters at Todd Lake the range in TDD at TLD was 390 °C d, a value greater than the mean. In summer, however, clear relations have been obtained between air and lake bottom temperatures (Table 2). This means that changes in lake-bottom temperature due to climate may only be inferred reliably from the summer regime. The rate of change of thawing-season climate over the last 20 years in the western Arctic has not been as high as during the freezing season. At the study site, the rate of change in TDD_a and FDD_a over 2000-17 have been, respectively, 10 °C d/yr ($R = 0.47$, $p = 0.051$, $n = 18$) and -27 °C d/yr ($R = -0.44$, $p = 0.080$, $n = 17$). A statistically significant monthly temperature increase occurred only in May ($R = 0.59$, $p = 0.014$, $n = 17$) suggesting earlier thaw season starts, an alteration anticipated by Burn (2002). An apparent change of -0.75 d/yr ($R = -0.47$, $p = 0.064$, $n = 16$) for thaw initiation date is observed at TLT in late May.

The absence of a statistically significant relation between TDD_a or FDD_a and time directs this research towards analysis of analogue years to assess potential future tundra lake thermal regimes. Table 4 presents values from mean annual temperatures at Todd Lake for the mean year and the coolest and warmest four years. The difference in annual temperature between the coolest and warmest sets of years is most apparent on the terrace with a value of about 5.4 °C. For the years with complete data between 2000/01 and 2016/17, the relation between annual mean air ($T_a$) and terrace ($T_t$) temperatures at Todd Lake is: $T_t = 0.8T_a + 5.3$ ($R = 0.29$, $p = 0.318$, $n = 14$), a weak association that suggests an increase in $T_a$ of 6.7 °C may be required to generate a change in $T_t$ of 5.4 °C (Table 4). Given that the rate of increase in annual mean air temperature at Tuktoyaktuk (2000-17) has been 0.12 °C/yr, and over a longer period, 1970-2017, 0.06 °C/yr ($R = 0.61$, $p < 0.001$, $n = 45$), between about 55-110 years may be required to effect such a change in the lake thermal regime.

TALIK CONFIGURATION AND CLIMATE CHANGE

The terraces are significantly cooler than the lake bottoms in the central pools of Todd and Andrew lakes during the freezing season, causing the terraces to moderate the warming effect of the lakes on permafrost. Tundra lakes without terraces have a bowl-shaped talik, with the talik boundary at the surface near the lake shore. The talik depth beneath the centre of the lake increases with lake radius until a through talik is created at a critical radius for the lake. Up-thawing from
the permafrost base, dependent on the geothermal flux, also assists through talik creation. As lake diameter increases, the depth of the talik also increases (Mackay 1962). For lakes with terraces, the critical radius of the central pool declines as terrace width increases, since permafrost beneath the terraces is warmer than outside the lake basin (Burn 2002, Fig. 12a). The equation for the temperature at equilibrium \( T_z \) beneath a circular tundra lake at depth \( z \) is (Burn 2002, eq. 2, after Mackay 1962, eq. 8):

\[
T_z = T_g + \frac{z}{I} + \left( T_p - T_g \right) \left( 1 - \frac{z}{\sqrt{z^2 + R_p^2}} \right) + \left( T_{tp} - T_g \right) \left( \frac{z}{\sqrt{z^2 + R_p^2}} - \frac{z}{\sqrt{z^2 + R_{tp+}^2}} \right)
\]  

where \( T_p, T_{tp}, \) and \( T_g \) (°C) are, respectively, the mean annual temperatures of the central pool, permafrost in the terrace, and permafrost encircling the lake; \( I \) (m °C\(^{-1}\)) is the geothermal gradient; \( R_p \) (m) is the radius of the central pool and \( R_{tp+} \) (m) is the radius of the pool and terrace combined.

We measured \( T_t \) directly and calculated \( T_{tp} \) by estimating the thermal offset \( \Delta T = T_t - T_{tp} \) using eq. 13 in Romanovsky and Osterkamp (1995). Use of this equation requires TDD and FDD at the terrace surface and the thermal conductivity of saturated terrace sediments when frozen (\( \lambda_f \)) or thawed (\( \lambda_t \)). These conductivities may be determined from the volumetric geometric mean thermal conductivities of sand particles, ice, and water, approximately 5, 2.24, and 0.56 W m\(^{-1}\) °C\(^{-1}\), respectively (Woo 2012, eq. 2.24b). At porosity of 0.35, calculated values for \( \Delta T \) ranged from 0.7 to 1.5 °C, generally increasing with \( T_{tp} \) (Table 4).

### Table 3. Relations between FDD at lake bottom and in the air at Todd and Andrew lakes.

Temperatures in the deep pools generally did not fall below 0 °C, and so no FDD were recorded.

<table>
<thead>
<tr>
<th>Location</th>
<th>Dates</th>
<th>Relation</th>
<th>( R )</th>
<th>( p )</th>
<th>( n )</th>
<th>Missing years</th>
</tr>
</thead>
<tbody>
<tr>
<td>TLT</td>
<td>2000/01-2016/17</td>
<td>0.46FDD(_a) - 87</td>
<td>0.11</td>
<td>0.689</td>
<td>16</td>
<td>2010-11</td>
</tr>
<tr>
<td>ALT</td>
<td>2001/02-2009/10</td>
<td>1.27FDD(_a) - 4426</td>
<td>0.43</td>
<td>0.250</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>ALS</td>
<td>2001/02-2009/10</td>
<td>0.78FDD(_a) - 2472</td>
<td>0.28</td>
<td>0.463</td>
<td>9</td>
<td></td>
</tr>
</tbody>
</table>

### Table 4. Mean annual temperature in the central pool \( T_p, n = 12 \), on the terrace \( T_t, n = 14 \) and at the top of permafrost in the terrace \( T_{tp}, n = 14 \) at Todd Lake. \( T_{tp} \) has been calculated using Romanovsky and Osterkamp (1995, eq. 13), allowing for assessment of the thermal offset.

<table>
<thead>
<tr>
<th>Lake-bottom scenarios</th>
<th>( T_p )</th>
<th>( T_t )</th>
<th>( T_{tp} )</th>
<th>Thermal Offset</th>
</tr>
</thead>
<tbody>
<tr>
<td>Warmest 4 years</td>
<td>4.29</td>
<td>0.42</td>
<td>-0.77</td>
<td>1.19</td>
</tr>
<tr>
<td>Mean year</td>
<td>3.73</td>
<td>-2.56</td>
<td>-3.66</td>
<td>1.10</td>
</tr>
<tr>
<td>Coolest 4 years</td>
<td>3.33</td>
<td>-4.97</td>
<td>-5.96</td>
<td>0.99</td>
</tr>
</tbody>
</table>

Figure 7a presents critical \( R_p \) and \( R_{tp+} \) for a range of terrace sizes, using values in Table 4 for \( T_p \) and \( T_{tp} \). \( T_g \) is -6.3 °C and \( I \) is 45 m °C\(^{-1}\). As terrace width increases, the critical radius of the central pool \( (R_p) \) decreases, but at a rate less than the increase in terrace width. As a result, the combined radius of the central pool and terrace, \( R_{tp+} \), i.e., the critical radius of the lake, increases.
Figure 7. (a) Variation in critical radius of the lake ($R_{p+t}$) and central pool ($R_p$) with terrace size for the warmest (max), mean, and coolest (min) temperature scenarios. (b) Talik breakthrough depths near critical radius for the three scenarios in Fig. 7a as functions of terrace size.

Figure 8. Variation with terrace size of talik depth and depth to the base of permafrost beneath a lake near critical radius in equilibrium with $T_p$ and $T_{tp}$ for the warmest four years and of the same sized lakes with $T_p$ and $T_{tp}$ from the mean year or coolest four years. The talik depth and base of permafrost are at the same point for the warmest scenario.

Therefore, as terrace width increases, so do the sizes of lakes with through taliks. Figure 7b shows talik breakthrough depths just short of the critical radius (since through taliks are produced at the critical radius) for the same terrace widths as in Fig. 7a under the three scenarios in Table 4. Figure 7b illustrates a distinction between lakes with small and large terraces. For small terraces, the talik depth is greatest with the warmest central pool. As terrace size and $T_{tp}$ increase, the talik depth at critical radius declines, but more rapidly with the warm terraces, so that for terrace widths near 80 m and above, the talik depth is inversely related to lake-bottom temperature. Figure 7b demonstrates the thermal effect of the terraces on the base of permafrost beneath the lake, for the decrease in talik depth must be equivalent to an increase in elevation of the base of permafrost beneath the lake for a through talik. The figure implies that the base of permafrost rises with terrace width. Note that for the three cases in Fig. 7b the width of the central pool and hence the width of the lake vary separately with terrace width.
Figure 8 presents the graph of warmest conditions in Fig. 7b, and depths to the bottom of the talik and permafrost for lakes of the same geometries but with reduced lake-bottom temperatures. For the warmest case, as terrace size increases from 0 to 150 m, the central pool radius declines from 145.1 to 74.2 m (Fig. 7a). Figure 8 shows the change in talik depth and adjustment to the base of permafrost that may occur beneath these lakes as bottom temperatures increase from the coolest measured field conditions up to conditions that produce a through talik. The coolest conditions in the smallest central pool disturb the permafrost base from its initial condition (283.5 m) by 19.1 m. The time required to transform a small talik, formed under cold conditions, to a through talik may be estimated using a modified Stefan solution as developed by Roy-Leveillee and Burn (2019). There is a small change in $T_p$, but the change in $T_{tp}$ reduces the heat fluxes below the talik so that thawing is required to establish the new equilibrium (Table 4).

Roy-Leveillee and Burn (2019) considered the case of two-step thawing beneath a lake where bottom temperature changed at time $t_{tr}$ after thawing began, writing:

$$z(t) = \sqrt{\frac{2\lambda T_i t}{L}} \text{ for } t < t_{tr}$$

$$z(t) = \sqrt{\frac{2\lambda_i}{L}} (T_2 (t-t_{tr}) + T_1 t_{tr}) \text{ for } t \geq t_{tr}$$

Here, $z(t)$ is the thaw depth at time $t$ after thawing begins. Using eq. [2], the model estimates the time required to thaw the initial (cold) condition ($t_{tr}$), given thermal conductivity $\lambda_i$ of the talik sediments, the initial temperature at the lake bottom ($T_i$) and the latent heat of fusion of permafrost ($L$). The subsequent time required to increase talik depth ($t - t_{tr}$) under a new lake-bottom temperature, $T_2$, to the new value may then be calculated by rearranging eq. [3]:

$$t - t_{tr} = \left( \frac{z(t)^2 L}{2\lambda_i} - T_1 t_{tr} \right) / T_2$$

Using a porosity of 0.35, $\lambda_i$ is 2.5 W m$^{-1}$ °C$^{-1}$ and $L$ is 1.05 x 10$^8$ J m$^{-3}$. Table 5 presents the time required to deepen the sub-lake talik from an initial condition formed with relatively cool terraces to a break-through depth under warmer conditions as represented by data in Table 4.

Where terrace widths are small, the original talik is quite deep. Therefore, a relatively short period is required to create a through talik. With wider terraces, the original talik is shallow and a longer period is required, but in the shallowest cases, where the terraces are widest, the initial rate of talik deepening is greatest due to surface proximity of the thaw plane. Thus, the time required for thawing down to the new talik depth may decline even for a greater thickness of thawed ground. This is evident in comparison of conditions at terrace widths of 100 and 150 m, where 99.3 m and 103.0 m are thawed, respectively, in transition over 3115 and 2806 years. In most cases, considerably more time is required to raise the base of permafrost and create a through talik. The geothermal flux (0.05 W m$^{-2}$) affects the up-thawing. The times for down-thawing are less than for up-thawing, with the smallest difference being 660 years for lakes without terraces. The time required for up-thawing escalates with terrace widths from 4780 to 8347 years between widths of 25 to 150 m. Table 5 makes clear that the time scale of climate change is quite different from the time scales involved in subsequent adjustment of permafrost conditions. In this example, the climate change is anticipated over a century, while the adjustments in talik geometry are estimated to take millennia. The time for talik adjustment is sensitive to porosity; if this declines to 0.1, the times required for adjustment are still on the order of centuries to millennia.
Table 5. Time required to establish changes in talik size associated with presently anticipated climate scenarios. The initial talik depth has been modelled for the coldest conditions.

<table>
<thead>
<tr>
<th>Terrace width (m)</th>
<th>Critical radius of pool (m)</th>
<th>Talik depth for warmest conditions (m)</th>
<th>Talik depth for coldest conditions (m)</th>
<th>Permafrost base for coldest conditions (m)</th>
<th>Time for coldest talik (yr)</th>
<th>Time to reach warmest talik (yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>145.1</td>
<td>159.6</td>
<td>106.3</td>
<td>202.3</td>
<td>2283</td>
<td>2187</td>
</tr>
<tr>
<td>25</td>
<td>132.2</td>
<td>159.0</td>
<td>86.5</td>
<td>221.7</td>
<td>1511</td>
<td>2755</td>
</tr>
<tr>
<td>50</td>
<td>119.8</td>
<td>157.4</td>
<td>72.4</td>
<td>235.0</td>
<td>1058</td>
<td>3028</td>
</tr>
<tr>
<td>75</td>
<td>107.7</td>
<td>154.7</td>
<td>60.9</td>
<td>245.2</td>
<td>749</td>
<td>3138</td>
</tr>
<tr>
<td>100</td>
<td>96.0</td>
<td>150.7</td>
<td>51.4</td>
<td>253.2</td>
<td>534</td>
<td>3115</td>
</tr>
<tr>
<td>125</td>
<td>84.8</td>
<td>145.6</td>
<td>43.2</td>
<td>259.5</td>
<td>377</td>
<td>3002</td>
</tr>
<tr>
<td>150</td>
<td>74.2</td>
<td>139.2</td>
<td>36.2</td>
<td>264.4</td>
<td>265</td>
<td>2806</td>
</tr>
</tbody>
</table>

CONCLUSIONS AND IMPLICATIONS

1. Lake-bottom temperatures are closely coupled to thaw season climate and are relatively consistent interannually;

2. Lake-bottom temperatures are independent of climate and more variable in winter, due to variations in the duration of freeze up and accumulation of snow cover on the ice surface;

3. The time required for talik geometry to respond fully to climate change similar to the interannual variations reported in this study is on the order of millennia, two orders-of-magnitude greater than the time required for the climate change that may provoke such response;

4. We may anticipate lake taliks in the western Arctic will be in disequilibrium with surface conditions for the foreseeable future.

Most climate change in the western Arctic is occurring in fall and winter. The thermal regime of the central pools is relatively insensitive to such change because they are deeper than seasonal ice thickness and their temperatures remain close to 0 °C in these seasons; over the year measured mean temperatures at the lake bottom have been about 3.7 °C (Table 4). The thermal regime of terraces, however, is highly responsive to climate warming through promotion of zero curtain effects (Table 4; Fig. 3). The greatest change in talik depth described in this paper after a simplified climate change simulation occurred with the largest lake terraces. The specific talik dimensions and temporal considerations given in this paper are from models configured to circular lakes and to thawing based upon a 1-dimensional solution. In three dimensions, heat will be lost laterally into permafrost and therefore the times for talik adjustment presented in this paper are likely underestimates. Elongate lakes have smaller critical dimensions than circular lakes (Burn 2002). In consequence, talik adjustment in these lakes may occur more quickly.

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REFERENCES


The Influence of Thermal Erosion at River Bed Deformation in Permafrost Areas

Elena Debolskaya¹; Vladimir Debolskiy²; Oksana Maslikova³; Ilya Gritsuk⁴; and Alexander Ivanov⁵

¹Water Problems Institute, Russian Academy of Sciences, Moscow, Russia. E-mail: e_debolskaya@yahoo.com
²Water Problems Institute, Russian Academy of Sciences, Moscow, Russia. E-mail: vdebolsky@mail.ru
³Water Problems Institute, Russian Academy of Sciences, Moscow, Russia. E-mail: oksana68@mail.ru
⁴Water Problems Institute, Russian Academy of Sciences; RUDN Univ., Moscow, Russia. E-mail: gritsuk_ii@pfur.ru
⁵Politecnico di Torino, Corso Duca degli Abruzzi, Torino, Italia. E-mail: alexandre.ivanov@polito.it

ABSTRACT

The study is focused on the deformations caused by the impact of water flow in a river channel composed of meltable permafrost bed materials. It is based on the results of laboratory and mathematical simulation. The results of numerical calculations are compared with data of laboratory and field observations. The study shows that a comprehensive and adequate model of river channel deformations should take into account not only ablation, but also other factors, including heat transfer in the soil, sediment transport, and bank slope collapses. Experiments in a hydraulic flume were good enough to reproduce the effect of delayed collapse, consisting in nonsimultaneous impacts of channel-forming rock melting and a freshet. The major factors that have an effect on channel transformations in permafrost-zone rivers are identified.

INTRODUCTION

The importance of the studies of thermal erosion phenomena has been formulated in Veh (2015), where the need for accurate determination and evaluation of the process of thermal-erosion relief forms was mentioned, as the thermal erosion causes physical, chemical, and biological changes in both time and space.

The deformation processes in river channels in the permafrost zone during spring flood, which is accompanied by warming of bank slopes and an abrupt rise of water level, are largely determined by thermal erosion, or, more exactly, river or fluvial thermal erosion. A definition of thermal erosion as applied to the near-bank zone has been given in Are (1984).

Geomorphologically, riverine thermal erosion is a combined mechanical and thermal erosion of thawed river banks composed of permafrost bed materials. River thermal erosion manifests itself in a lateral underwashing of alluvial terraces or bedrock shelves. The specific features of its development are largely similar to the erosion of water body shores. The radical change in the structural bonds of permafrost bed materials during thawing produces a considerable effect on their resistance to erosion. This type of erosion sometimes causes bank collapses into the river. According to the definition in Gatto (1995) the thermal process includes thawing of frozen sedimentary material under the effect of turbulent water flow, followed by mechanical erosion, when hydraulic forces exceed material strength.

The statement that the joint effect of mechanical erosion, caused by a direct impact of water
flow, and the thawing of bank slopes leads to more considerable deformations and destruction is not obvious, as follows from Scott (1978). This study deserves special attention, as it represents a detailed examination of the impact of permafrost on Alaska watercourses. It formulates theses regarding the dependence of deformation rates on external impacts, the parameters of flows, and soil characteristics. The author determines two major questions that provoke stable interest to studying the behavior of arctic watercourses. What is the effect of permafrost on the rate of bank erosion—does the permafrost hamper lateral migration or does a river in a permafrost zone show higher rates of lateral erosion than that one would expect of watercourses which flow effectively for less than half of a year? Again, what is the role of spring flood in the determination of channel parameters?

When considering the first question, some authors support the conclusion made in Leffingwell (1919) at the very beginning of thorough studies into this problem and consisting in that the lateral erosion generally slows down in the presence of frozen soil. From the observations that were made during early studies of arctic rivers, the author comes to the conclusion that, although the effect of flowing water is nearly zero within eight or nine months every year, a certain portion of annual precipitation during the short summer is likely to produce a larger erosion effect than it would have done in the case of perennial flow. The author supposes that, when the soil is frozen, the erosion of alluvial banks slows down. Slow deformations can continue as ice in the soil is melting, but during floods, when bank erosion in temperate regions is high, the frozen soil cannot be destroyed rapidly. Therefore, we can expect that arctic rivers will meander slower than it is typical of rivers in warmer regions.

As noted in Cooper (1973), permafrost stabilizes the bank material, which, when thawed, will show low resistance to erosion. However, McDonald and Lewis (1973) believe that permafrost retards erosion only for short periods.

On the other hand, Ritchie and Walker (1974), Walker and Arnborg (1966) emphasize the role of permafrost in the propagation of appreciable lateral erosion (on the average, 10 m in a domain) because of lateral collapse of banks during ice-drift level rise in the Colville River in the northern part of Alaska. Walker (1973) conveys the general belief that lateral erosion has a considerable effect on rivers of arctic Alaska because of permafrost. Scott (1978) comes to the conclusion that to find out whether flows in the permafrost zone are very stable or they show lateral migration in excess of the average requires more comprehensive studies, which have been carried out later in different Alaska watercourses. As the result, relationships were established between the thawing rate of permafrost, on the one hand, and its particle size and ice content, on the other hand. The rate of thawing was shown to be proportional to the size of frozen soil particles and inversely proportional to soil ice content. An obvious result of this study was the conclusion that, in the studies of arctic river flows, permafrost is to be regarded as an important additional variable. Many studies were focused on the relationship between thermal erosion rate in rivers crossing permafrost areas and soil ice content (Aguirre-Puente 1994, Are 1984, Gillie 1990).

The role of soil ice content in the formation of thermal-erosion niches is assessed in Kobayashi (1985) and Kobayashi et al. (1999). According to Solov’ev, (1962), thawed material is being rapidly removed mechanically from river banks with moisture content of 75—90%. In Lantuit et al. (2008), a statistical relationship between soil ice content and erosion rate in the coastal areas of the Arctic was studied for 545 coastal segments. Their analysis has shown that the retreat rates (0—9 m per year) slightly increase at the volumetric content of soil ice (0—70%); therefore, the role of other factors should be studied. Observations show that sometimes the picture is inverse. For example, the results of observation of vast ice sheets on the banks of the Lena River confirm that pure ice
has been destroyed to a lesser extent than frozen rocks with lower ice content (Gautier and Costard 2000).

Such contradictory conclusions regarding the relationship between ice content and erosion rate show that the process is complex and its thermal and mechanical aspects require further studies.

The assessment of the effect of thermal and mechanical impact of water flow on the deformation of bank slope is another important aspect of the problem. Scott (1978) analyzes field measurement data to show that the ratio of the depths of thawing and mechanical erosion considerably depends on soil composition. In cohesive soils, the process of thawing is more rapid and governing, while in the case of loose soil, these processes are comparable.

Studies of river thermal erosion is of particular importance in solving the problem of destructive deformation phenomena in large rivers of northern areas and subpolar zone because of the long segments of their banks containing permafrost and ice sheets. An exhaustive survey of studies into the geomorphology of large rivers, in particular, rivers of permafrost zone, is given in (Ballentyne 2018, Large Rivers 2007), the analysis of which shows that studies of riverine thermal erosion are much fewer than those of the thermal erosion of permafrost soils along the seacoast. The main achievements in solving this problem have been made in studies of French researchers (Costard 2003, Costard 2014, Costard 2007, Dupeyrat 2011, Dupeyrat 2008, Gautier 2003, Randriamazaoro 2007). They are also referred to by the authors of reviews (Kizyakov 2016), focused on cryogenic processes. The authors note that the erosion of banks composed of permafrost with high ice content, in accordance with observations (Shur 2002), can be supplemented by thermal denudation on bank benches.

The number of studies based on field and laboratory observations of river thermal erosion is not large because they are difficult to implement and the respective problem is characterized by a large number of parameters. In this context, of particular importance is the series of Russian (Kizyakov 2016, Tananaev 2007, Tananaev 2012, Tananaev 2014, Tananaev 2016) and French studies (Costard 2003, Costard 2014, Costard 2007, Dupeyrat 2011, Dupeyrat 2008, Gautier 2003, Gautier 2000) into riverine thermal erosion, based on observations in the Lena River, laboratory experiments in a refrigerator, and mathematical modeling.

The major two differences between channel deformation processes in rivers in permafrost zone and beyond it are due to the considerable effect of thermal erosion and the nonsimultaneous effects of the melting of channel-forming materials and flood wave. Laboratory experiments (Kotlyakov 2011) and field observations (Dupeyrat 2011) have shown that the role of thermal erosion in the deformation of channels in permafrost regions is considerable and, sometimes, even greater than that of mechanical erosion. However, the second factor that distinguishes river channel deformations in permafrost zone from those in moderate-climate areas is the time lag between the manifestation of maximal erosion activity and flood peak; this factor questions both the opinion that the presence of permafrost accelerates deformation processes and the opposite opinion. As mentioned in (Tananaev 2002, Zaitsev 2008) soil freezing reduces the erosion activity in the period of snow melting and the passage of spring flood wave. However, feshets are accompanied by more active erosion, which is facilitated by the gradual thawing of permafrost soils in summer and their total saturation because of the effect of an aquiclude, i.e., the roof of permafrost rocks. (Tananaev 2002, Zaitsev 2008) believe that the sediment loads during feshets in warm seasons, even at lower water discharges, can be far in excess of their values during spring floods. Clearly, in addition to the thermal effects, which manifest themselves as thawing of frozen channel-forming rocks, the sediment transport, solifluction, and collapses also contribute much to the erosion processes in arctic rivers. The objective of the study is to assess the role of the latter processes, i.e., to reveal the significance of each factor that governs thermal erosion.
Mathematical Model: The mathematical model consists of a hydrodynamic, heat, and deformation modules. The major model equations are described in (Debol'skaya 2014). The new interpretation considered here can take into account variations in ice content of frozen inclusions. In addition to changes in the position of the surface in contact with water, the model takes into account variations in the positions of surfaces at the boundary with warm soil. In the calculation of the position of the water–ice/frozen soil boundary, the Stefan condition is modified by adding an advection term, which accounts for the transport of thawed-material particles by water flow, as it has been done in the model (Dupeyrat 2008, Randriamazaoro 2007):

$$h(T_w - T_m) = \rho L \frac{\partial s}{\partial t} - k \left( \frac{\partial T}{\partial x} \right)_{x=s(t)}$$

where $s(t)$ is the instantaneous position of the interface, $T_w$ is water temperature, $T_m$ is ice melting temperature, $k$, $\rho$, $L$ are the heat conductance, density, and the latent heat of thawing of the frozen sample. The heat conductance $h$ between the turbulent water flow and the frozen sample can be found from the relationship $h = \frac{k_w}{L_{char}}$, where $k_w$ is water thermal conductivity; $L_{char}$ is characteristic scale, $Nu$ is Nusselt number, calculated using the empirical formula $Nu = Pr^\alpha Re^\beta A$, where $Pr$ is Prandtl number, varying from 13 to 7 as water temperature rises from 0°C to 20°C, $Re = \frac{L_{char} V_w}{\nu_w}$ is Reynolds number, $V_w$, $\nu_w$ are water velocity and kinematic viscosity, respectively.

The empirical coefficients, as well as in (Costard 2007), were taken as follows: $A = 0.003031, \alpha = 0.3333, \beta = 1.1211$.

Laboratory Modeling: The laboratory experiment was carried out in a close-cycle flume 4 m in length, 0.25 to 1 m in width, and 0.15 m in depth. Water temperature was kept constant during each experiment. The bed slopes consisted of graded river sand with particle size up to 0.001 m. The cross-section of the simulated channel was trapezoidal with lateral slopes varying in individual experiments. Samples of frozen mixture of sand and water with specified ice content or pure-ice samples were placed on the bank slope of the flume. The flow velocity varied depending on water flow rate, flume bed slope, or cross-section in each experiment. The most vivid results were obtained in a laboratory experiment with flume bed width of 0.15 m and depth of 0.1 m at the slope of the side wall with frozen sample $\alpha = 30^\circ$ and water flow rate of 5.8 L/s. The samples of frozen sand or ice had the form of plates with dimensions varying slightly from one experiment to another. On the average, the plates were 25 cm in length, 15 cm in width, and 3 cm in thickness. The frozen samples were weighed and measured before and after the experiment. The change in sample weight per unit time was taken as the characteristic of the deformation dynamics of the frozen sample. Numerous studies (Cooper 1973, Are 1983, Ballentyne 2018, Kobayashi 1985, Kobayashi 1999, Lantuit 2008, Shur 2002) have shown that the major factors that have an effect on the deformations caused by thermal erosion are water and soil temperature, soil ice content, and water flow velocity. The latter parameter was the only one that could be varied within acceptable limits in the laboratory experiments; the soil ice content was represented by two values: 1 for pure ice and 0.2 for frozen sand. The temperatures of water and the surrounding soil varied depending on the weather conditions; they were taken into account as corrections to other relationships. Figure 1 gives the change of the weight of the sample of (a) pure ice and (b) frozen sand per unit time versus the mean water flow velocity.
Figure 1. Changes in the weight of (a) the sample of pure ice and (b) frozen sand per unit time versus the average velocity of the water flow: (1) measurement data, (2) numerical experiment results.

Figure 1a also gives the results of numerical experiments with an updated mathematical model, which has been developed by the authors earlier (Debol'skaya 2014). These results will be described in greater detail further below. As can be seen from the plot, pure-ice inclusions, first, show lesser effect of the flow than the samples with lesser ice content do, other conditions remaining the same, thus supporting the conclusions made in (Aguirre-Puente 1994, Costard 2003), and, second, the changes in their weight are less dependent on water flow velocity.

Figure 2. The surface of the drained channel (a) calculated by the model and (b) its photo.

The following experiment was carried out to study the nonsimultaneous effect of the thawing of channel-forming rocks and a freshet. An ice plate was placed for 12 h into the bank slope composed of wet sand with a positive temperature. The level of water flow all over this period was kept below the lower surface of the plate. A niche has formed in the volume previously occupied by the ice, and the walls of this niche collapsed after the starting of pumps and the formation of a wave to simulate a freshet. This effect, consisting in a time lag between the thawing of a frozen inclusion and the mechanical impact of water flow, i.e., a time shift in two mechanisms of thermal erosion process, can be referred to as an effect of delayed collapse. One more experiment was carried out to verify the mathematical model. In this experiment, an ice bar was placed into the
bank slope of a water flow 11 cm in depth at an average velocity of 0.24 m/s. Once the bar had melted completely, the water pumps were switched off; the channel became free of water, and a niche formed at the place of the bar; the dimensions of the niche coincided with those predicted by the model up to measurement errors. The thawing times in the laboratory and numerical experiments also coincided. Figure 2 gives a photo of the channel, drained at the end of the experiment, and its surface calculate by the model.

RESULTS AND DISCUSSION

The main objective of the study was to assess the significance of the factors involved in the process of river channel deformations in the permafrost zone, including separation of frozen particles through thawing and their transport by water flow, i.e., ablation, described by heat conductance equation and a modified Stefan condition (1); sediment transport induced by disturbance of flow homogeneity by thawed domains; collapses of bank slope, which loses the skeleton it has had in the form of frozen soil and becomes unstable as its natural equilibrium slope drops below the actual slope. The model was verified against the results of preliminary calculations for short time intervals (up to 15 min) with the comparison of the obtained results with laboratory experiments for the case of ablation alone (Debol'skaya 2020).

Figure 3. Time variations of (a) the averaged deformations and (b) cross sections of the channel, in the middle of the plate 60 min after its placement in the coastal slope, calculated by different versions of the model: (1) full model, (2) pure ablation model (without sediment transfer and collapse), (3) excluding collapse (ablation + sediment transfer), (4) excluding sediment transfer (ablation + collapse).

The difference between averaged deformations calculated by a pure-ablation model, i.e., not taking into account collapses and sediment transport, and the full model reached 20%. In the simulation over longer periods, the difference increases, as can be seen from Fig. 3, which gives time variations of the averaged deformations, calculated by different variants of the model, and cross-sections of the channel, calculated for the section passing through the center of the plate 60 min after its placing into the bank slope for the same model variants. Clearly, the values of deformations calculated with no account taken of sediment transport and collapses are much higher than the values obtained from the full model.

The effect of flow velocity: Numerical experiments with the model of total deformations were used to assess the contributions of different factors to the process at different approximations of the model. All curves in Figs. 4, 5 and 7, 8 show changes over time in deformations averaged over the area of the plane of the frozen sample in contact with the flow. Figure 4 shows these variations calculated at different Reynolds numbers for the case of a sample of pure ice with dimensions of 0.075 m along the flow and 0.045 m across it inserted into the bank slope.
The transverse size of the sample was the same for all experiments described below. All relationships show a rapid increase in the deformations in the first 6 min, caused by intense thawing and increasing thermal boundary layer, followed by approaching a steady-state value. With the incorporation of heat exchange processes along the flow, sediment transport, and collapses, the steady state will be reached faster. Clearly, all other factors remaining the same, the greater the flow velocity, the greater the deviations from the pure ablation model. At small Re, the main factor of the difference is heat transfer in the direction of flow; as the flow velocity increases, sediment transport is gaining in importance, though its role reduces to decreasing the deformations, as the authors of the ideal laboratory experiment and the model of pure ablation has expected (Aguirre-Puente 1994, Costard 2003, Randriamazaoro 2007). As the flow velocity approaches the erosion threshold, the collapse becomes the main factor and the deformations become close to ablation values and next even exceed them.

![Graph](image)

**Figure 4. Development of deformations calculated for the case with a pure-ice sample with a length of 0.075 m along the flow placed into bank slope at different Reynolds numbers (Re): (a) Re = 6340, (b) Re = 9500, (c) Re = 12700: (1) in the case of ablation alone, (2) with heat transfer along the longitudinal axis of the flow also taken into account, (3) with deformations from sediment transport equation also taken into account, (4) with addition of collapse, i.e., with all model factors taken into account.**

The dependence of deformations on ice content, the size of frozen segment, water temperature and the size of soil particles: The content of ice in the frozen sample (soil iciness \(w\)) has a considerable effect on both the rate of its thawing and its further erosion. However, this effect is ambiguous, depending mostly on the cohesion and size of the sample and its particle size distribution. Studying this dependence was not among the objectives of the model implementation considered here. Numerical experiments with the model of total deformations suggested conclusions which are in agreement with those obtained in (Costard 2003, Randriamazaoro 2007) regarding an increase in erosion at a decrease in iciness. The results of these experiments with samples with a size of 0.15 m along the flow and with different iciness are given in Fig. 5 as curves showing time variations of the deviations of the deformed sample surface from its initial position. For this experiment Re = 9500.
Figure 5. Time variations of deviations of the deformed surface with a sample 0.15 m in length placed into it at different ice content: (1) in the case of ablation alone, (2) with heat transfer along the longitudinal axis of the flow also taken into account, (3) with deformations from sediment transport equation also taken into account, (4) with addition of collapse, i.e., with all model factors taken into account.

Figure 6. Bed reliefs calculated based on numerical calculations (a), (b) not taking into account the heat impact along the flow, sediment transport, and collapse and (c), (d) taking into account these factors, after 15-min impact of water flow (Re = 9500) on samples of pure ice (w = 1) and frozen sand (w = 0.2) 0.15 m in length.

Other factors remaining the same, the lesser the ice content of the inclusion, the greater the deviations of the model from pure ablation. To more vividly demonstrate these deviations, Fig. 6 gives bed reliefs based on numerical calculations after 15-min water flow impact (Re = 9500) onto
pure ice samples \((w = 1)\) and frozen sand \((w = 0.2)\) with a length of 0.15 m. The top part of the figure corresponds to calculations not taking into account the thermal impact along the flow, sediment transport, and collapse, while the bottom part reflects the effect of all these factors.

The difference is obvious, but, as can be seen from Fig. 7 (which gives the same dependences as Fig. 5 but in the case of frozen plates with different ice content, which occupy the entire length of the model segment 1.5 m in length), an increase in sample length leads to an opposite effect, i.e., a decrease in the effect of incorporation of additional factors at decreasing iciness. Moreover, it can be seen that, the longer the sample and the lesser its ice content, the greater the time required for the erosion process to reach stationary regime.

Figure 7. Time variations of deviations of the deformed surface with a sample 1.5 m in length laced into it with different ice content: \((1)\) in the case of ablation alone, \((2)\) with heat transfer along the longitudinal axis of the flow also taken into account, \((3)\) with deformations from sediment transport equation also taken into account, \((4)\) with addition of collapse, i.e., with all model factors taken into account.

However, it is worth noting that laboratory experiments in a hydraulic flume with frozen sand samples with iciness of 20% placed into a bank slope showed no increase in deformation compared with the flow along an ice sample. This might be due to the fact that the sand had a highly heterogeneous particle size distribution and the flow was not stable enough in the initial segment, resulting in an inverse effect, i.e., the deposition of the sediments transported in it onto more solid frozen segment and a decrease in the bank slope below the critical value, which prevented collapse. Similar contradictory field data are given in (Dupeyrat 2011), where it is mentioned that, in some parts of Lena banks, ice-rich beds protrude several decimeters from the nearby sandy permafrost beds, in which ice content is commonly 20%. These banks had suffered the impact of flow with a high discharge \((50000 \text{ m}^3/\text{s} \text{ during flood})\), which facilitated ablation and accelerated erosion at lower ice content, as explained in this study. However, in other places, as is often mentioned in the literature, the largest erosion is observed in permafrost with highest ice content. In many cases, this is attributed to the material being cohesive and the erosion characteristics of incoherent deposits changing considerably in the presence of a small amount of clay.

The model was also used to study the effect of water temperature and the initial temperature
of the frozen sample. The effect of water temperature is considerable as can be clearly seen in Fig. 8a and the results of measurements (Dupeyrat 2008) given in Fig. 8b. The effect of the temperature of the sample is very weak.

The conversion of deformation rates for the conditions of the Lena River, for a frozen inclusion with ice content of 80%, we obtain an estimate of ~40 m, which is in agreement with the result obtained by model calculations in (Costard 2007, Dupeyrat 2011) based on data of field measurements.

![Graph showing variations of deviations of deformed surface at different water temperature](image)

**Figure 8. Variations of deviations of deformed surface at different water temperature ($T$) by data of (a) mathematical and (b) laboratory experiments: (1) 5°C, (2) 7°C, (3) 9°C.**

The Fig. 9 shows the dependence of bed deformations under the impact of thermal erosion and water flow on the size of soil particles other things being equal, obtained from the data of a numerical experiment using the model. It is obvious that the deformations increases with decreasing particle diameter.

![Graph showing variations of deviations of deformed surface at different size of soil particles](image)

**Figure 9. Variations of deviations of deformed surface at different size of soil particles.**

**CONCLUSION**

Numerical experiments carried out with the use of a modified mathematical model of full deformations over long time intervals showed that the difference between averaged deformations
calculated by the model of pure ablation, i.e., with no account taken of collapses and sediment transport, and the full model, can increase. In this case, the values of deformations calculated with sediment transport and collapses not taken into account are far in excess of the values calculated by the full model. This result is in agreement with the data of experiments with a flume.

At the averaging over channel segments longer than the size of ice inclusions, the average values of deformations can slightly differ when calculated by the full deformation model or the model of pure ablation; however, the values of maximal (accumulation) and minimal (erosion) deviations of the channel surface in the calculations by the full model are far in excess of those calculated by the pure-ablation model. This shows that channel deformations over the space are uneven. Experiments in the hydraulic flume were accurate enough to reproduce the effect of delayed collapse, consisting in the nonsimultaneous effects of the thawing of the channel-forming rocks and a freshet.

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Ground Temperature Responses to Climatic Trends in a Range of Surficial Deposits near Kangiqsualujjuaq, Nunavik

Catherine Deslauriers¹; Michel Allard²; and Pascale Roy-Léveillé³

¹Centre d’études nordiques, Dept. of Geography, Université Laval, Quebec City, QC, Canada.
E-mail: catherine.deslauriers.2@ulaval.ca
²Centre d’études nordiques, Dept. of Geography, Université Laval, Quebec City, QC, Canada.
E-mail: michel.allard.3@ulaval.ca
³Centre d’études nordiques, Dept. of Geography, Université Laval, Quebec City, QC, Canada.
E-mail: Pascale.Roy-Leveillee@ggr.ulaval.ca

ABSTRACT

This paper discusses ground temperature responses to climate variations in a range of surficial geology units near Kangiqsualujjuaq, Nunavik. Sixteen thermistor cables, extending to depths from 2.9 to 20 m, were installed in various settings near the community over 35 years. Here we examine ground temperature at three contrasting sites: a lithalsa in ice-rich marine clays near the shoreline, a glaciofluvial delta (sand and gravel), and bedrock (massive gneiss), to compare thermal regimes and responses to climatic variations over the last 20 to 30 years. With climate warming, the site in marine deposits has become isothermal near 0°C and the release of latent heat now impedes ground temperature responses to climate. In sand and gravel, freeze-back is delayed until late in winter with clear development of a zero curtain due to formation of a perched water table in the deepened active layer. Short-term air temperature variations and regional climatic trends are effectively transmitted at depth into highly conductive dry bedrock, where active layer depth also readily responds to air temperature variations.

INTRODUCTION

As ground temperatures in the circumpolar North continue to respond to climatic warming, substantial modelling efforts are invested in predicting permafrost sustainability and thermal response to climatic trends (Karjalainen et al. 2019; Melton et al. 2019; Obu et al. 2019). Long term ground temperature datasets are needed to compare modelled and observed ground thermal regimes (Biskaborn et al. 2019), including temperature records capturing local to regional variations in ground thermal response to climatic trends. This study examined ground temperature responses to climatic variations from the late-1980s to present in a range of geomorphological environments within the community area of Kangiqsualujjuaq (17.5 km²) in Nunavik (Figure 1). Specifically, it aimed to: (1) review recent climatic trends near Kangiqsualujjuaq by comparing them with trends from the eastern and the western Arctic; (2) examine key differences in ground thermal regime in bedrock, glaciofluvial sand and gravel, and glaciomarine silts and clays near Kangiqsualujjuaq; and (3) discuss key controls on ground temperature response to decadal and longer-term climatic trends between different landscape elements.

BACKGROUND

Climatic Trends in the Western and the Eastern Canadian Arctic: Western Arctic air temperatures have been warming more or less consistently since the 1970s (Burn 2004; Smith et al. 2010), as exemplified by the temperature record near Inuvik, in the Northwest Territories (Figure 2a). In the Eastern Arctic, however, air temperatures generally cooled between 1948 and
1993. For instance, Przybylak (2000) reported an annual cooling of 0.32°C for Kuujjuaq in Nunavik and of 0.34°C for Iqaluit in the eastern Arctic between 1950 and 1995, significant at the 0.05 level. This cooling period was felt in permafrost temperatures and was also responsible for the upgrowth of ice-wedges in the permafrost (Allard et al. 1995; Przybylak 2000; Kasper and Allard 2001; Smith et al. 2010; Tremblay et al. 2012; Allard et al. 2020b) (Figure 2a). The climate warmed abruptly starting in 1993 and temperatures generally rose importantly until 2010, the warmest year on record. A dendrochronological study conducted in Kangiqsualujjuaq on the growth rate of *Larix Laricina* corroborates this warming trend since the 1990s in the eastern Arctic (Dufour-Tremblay et al. 2012). More recently, Gagnon and Allard (2019) note that mean annual air temperature (MAAT) in Salluit has been decreasing since 2010, as can also be seen for other eastern Arctic sites in Environment Canada’s records (Figure 2a).

Figure 1. Surficial geology map of Kangiqsualujjuaq, Nunavik (modified from Allard et al. 2020). Red triangles represent thermistor cables, and numbers with an asterisk (*) indicate cables that are currently active. Surficial deposit type is indicated with a letter and colour code on the map, and includes R: bedrock (pink), Tb: till blanket (green), GFb: glaciofluvial sediments (yellow), and glaciomarine sediments (blue), such as GMi: intertidal sediments, and GMn: littoral and nearshore sediments.

**STUDY AREA: KANGIQSUALUJJUAQ**

Kangiqsualujjuaq, formerly known as George River, is one of 14 Inuit communities in Nunavik and was home to 942 residents in 2016 (Statistics Canada 2017). It is located 25 km southeast of Ungava Bay and 160 km northeast of Kuujjuaq, and borders Akilasakallak Cove (Figure 1). The
municipality straddles the treeline while standing partly in the subarctic tundra. The Kangiqsualujjuaq region has a mean annual air temperature of -3.4°C since 1990. Mean annual air temperature was between -5.4°C and -6°C in 1987 (Gahé et al. 1987).

The Kangiqsualujjuaq area rests mainly on gneiss, shaped by glacial action into a series of rocky valleys and elongated hills. Deglaciation began about 7400-7300 BP in the area. The village is located in the extensive discontinuous permafrost zone, at the bottom of a 5 km-long southeast-northeast oriented glacial valley (Allard et al. 2020a). Following deglaciation, a marine transgression occurred up to 100 m a.s.l. and the d’Iberville sea eventually regressed, in the process of which several types of sediments were deposited in a succession of layers (Allard et al. 1989). Glacial deposits and glaciomarine sediments are the two main categories of surficial deposits observed in the area (Allard et al. 2020a) (Figure 1).
METHODS

Analysis of Air Temperature Data: Mean annual air temperatures used to generate the graph for Kuujjuaq, Inuvik, and Iqaluit were extracted from Environment Canada’s Adjusted and Homogenized Canadian Climate Data (AHCCD) dataset (ECCC. 2020). A 10-year moving average was calculated for all 3 locations. As air temperatures are not recorded by Environment Canada in Kangiqsualujjuaq, those from Kuujjuaq, located 160 km southwest of the village, were used as a reference for the area (1948 to 2018) and air temperatures measured ca. 1.5-2 m above ground at two of the thermistor cables sites (Site 3 on bedrock and Site 10 atop a lithalsa) (Figure 1). Air temperatures from 1993 to 2012 were available for Site 3 and from 2008 to 2018 for Site 10. A correlation analysis was done for the mean monthly air temperatures between Kuujjuaq and Site 3 as well as between Kuujjuaq and Site 10.

Installation of the Thermistor Cables: Sixteen thermistor cables, extending to depths ranging from 2.90 to 20 m, were installed and maintained by researchers from the Centre d’études nordiques at different times starting in 1987 for various projects leading to theses, research papers and government reports. These cables were installed in various geomorphological settings over a 17.5 km² area, including bedrock, glaciofluvial sediments, marine sand and gravel, lithalsas in marine silt, and palsas consisting of 1 m-thick peat over marine silt. The 20 m-deep cables in bedrock and coarse soils such as sand and gravel were installed in drill holes with an airtrack drill that was available during the construction of the airport in 1989. The first 3 meters of the hole in glaciomarine silt was drilled with a portable earth-drill (Calmels et al. 2005) and from that depth down, by waterjet drilling until refusal when stones (till?) were met about 20 m deep. In all holes, a PVC casing was installed and filled with silicone oil. The thermistor cables were then inserted and linked to Campbell Scientific™ dataloggers. Although some attempts were made to transfer data by radio, all data were recovered on site during annual visits in September or October. The data were compiled and quality-checked for erroneous values. The data from all cables, including those discussed in detail in this paper, were stored in Centre d’études nordiques’ databases and made publicly available via Nordicana D (Sarrazin and Allard 2020). The three example sites presented below were selected to represent three different types of surficial geological units (glaciomarine, glaciofluvial, and bedrock).

Determination of Yearly Maximum Thaw Depths: The maximum depth of thaw was estimated for each year based on ground temperature data collected in early October. For the purpose of interannual comparison, the 0°C isotherm was assumed to represent the thaw front. Ground temperatures were measured by thermistors at depths spaced several centimeters to meters apart, so a linear interpolation (equation below) was used to calculate maximum thaw front penetration (Dmax, m) for each year based on the depths of the temperature sensors located immediately above (D1, m) and below (D2, m), the maximum depth of the 0°C isotherm and the maximum temperatures measured at D1 (T1, °C) and at D2 (T2, °C).

\[
D_{\text{max}} = -\frac{D_2 - D_1}{T_2 - T_1} T_1 + D_1
\]

RESULTS AND DISCUSSION

Climatic Trends Near Kangiqsualujjuaq: The mean annual air temperature (MAAT) in Kuujjuaq decreased slightly between the 1940s and the 1990s and the warming trend that extended between 1993 and 2005 in Nunavik, as well as the cooling that started in 2010, are clearly visible for Kuujjuaq and Kangiqsualujjuaq (Figure 2a) (Allard et al. 2012; Brown and Lemay 2012). The
air temperatures measured at sites 3 and 10 in Kangiqsualujjuaq were very closely correlated to air temperatures in Kuujjuaq recorded at the Environment Canada weather station \((r = 0.997)\) (Figure 2b and 2c). Mean monthly air temperature was slightly higher during winter and slightly lower during summer in Kangiqsualujjuaq than in Kuujjuaq. This difference is likely due to Kuujjuaq being further from Ungava Bay (50 km), along the Koksoak River, where the maritime influence of the Bay on the local climate is slightly attenuated compared to Kangiqsualujjuaq, which is located nearer to the coast (25 km).

**Site Description:** Kangiqsualujjuaq straddles the latitudinal treeline and is partly in subarctic tundra (Gahé 1987; Allard et al. 2020a). As a result, a range of vegetation conditions are encountered in the area. Trees (mostly *Larix laricina* with some *Picea mariana*) grow on the slopes of the lithalsa formed in glaciomarine silt and clay and sparse shrubs (*Betula glandulosa* Michx.) 50-60 cm tall are scattered over it (Figure 3a). The glaciofluvial deposit, consisting of stratified sands and gravel, is covered by a thin layer of rampant shrubs and herbaceous plants, mosses, and lichens (Figure 3b). The red arrow points to a shallow furrow along a tundra polygon side. The bedrock site (massive gneiss) is also covered by a very thin vegetation cover, mostly mosses and lichens (Figure 3c). Our field observations revealed that sites 1 and 3 are almost devoid of snow cover in winter because they are wind swept, a widespread condition in such environments (Sturm et al. 2005).

**Figure 3.** a) Site 10 (lithalsa in glaciomarine sediments), b) Site 1 (glaciofluvial delta in sand and gravel) and c) Site 3 (bedrock). Photographs by D. Sarrazin.

**Active Layer Depth:** The active layer was deepest at the bedrock site (14 m), where thermal conductivity is high and rock water content extremely low because of low porosity. This allows ground temperatures to cross the 0°C isotherm without trading significant amounts of latent heat. In recent years, active layer depths were near 4 m at the sand and gravel and glaciomarine sites. The relatively deep thaw front penetration observed at these sites is associated with the recent climate regime, as measurements of active layer thickness in the 1980s revealed active layer thicknesses of about 1 m in sand and in silts (Gahé et al. 1987). The large range of active layer depths observed near Kangiqsualujjuaq emphasizes that the heterogeneity of maximum thaw depths (Luo et al. 2016) can be significant at the local scale where surficial deposits vary within short distances.
Figure 4. Ground temperatures in the active layer (Sept. 2011 to Sept. 2013) for a) Site 3, b) Site 10, and c) Site 1 in Kangiqsualujjuaq, and thermal envelopes for the same sites in pannels d), e) and f), respectively.
A long zero-curtain developed at the sand and gravel site (Site 1) during active layer freeze-back which was not completed until early February in 2011-13 (Figure 4c). This is associated with the dissipation of large amounts of latent heat during freeze-back (Burn 2004) and shows the presence of a perched water table at the base of the active layer in these soil materials that are highly permeable when thawed. A zero-curtain is also visible at a depth of 2 m in the coastal lithalsa (Site 10), showing that the freezing front reached 2 m around mid-December, and likely did not reach the bottom of the active layer (near 4 m) until late winter. At this site, ground temperatures at the top of permafrost remained near 0°C during winter, indicating that the heat extracted at the ground surface during the freezing season was nearly all latent heat of fusion, and no sensible heat was extracted from the bottom of the active layer or lower.

Temperature Profiles: The low heat capacity and high thermal conductivity of bedrock allowed for transmission of the surface temperature waves to depth. The annual amplitude of temperature variations was approximately 0.2°C at 20 m below the surface, meaning that the depth of zero annual amplitude (variations of less than 0.1°C) is deeper than the maximum depth of the borehole (20 m) (Figure 4d). In contrast, the depth of zero annual amplitude was reached near 10 and 4 m at the sites with glaciofluvial (Site 1) and glaciomarine (Site 10) deposits, respectively (Figure 4f and 4e). In fact, the entire thermal profile at the glaciomarine site is isothermal near 0°C (Figure 4e), and thaw is likely progressing along the entire profile as unfrozen water content increases as temperatures near 0°C in silt and clay (Romanovsky and Osterkamp 2000).

![Figure 5. Ground temperature in Kangiqsualujjuaq, Nunavik, at a depth of around 20 m (respectively 18 m, 20 m and 19.75 m) for the three case study sites discussed above: sites 1 (glaciofluvial sand and gravel), 3 (bedrock), and 10 (glaciomarine silt and clay).](image)

Ground Temperature Responses to Climate Variations: In bedrock (Site 3), the air temperature signal is transmitted to depth effectively, the temperatures waves taking about 6 months to reach the 10 m depth and 11 months to reach the 20 m depth (not shown here in detail). The Eastern Arctic cooling trend in air temperatures that extended until the 1990s was recorded to
ground temperatures at depth in bedrock as well as the subsequent warming trend and the very recent cooling since 2010 (Figure 5).

Temperature at the 20 m depth in sand and gravel also followed air temperature trends, although with a lag (within a year or two) in its response compared to bedrock temperatures at depth. This is consistent with the lower thermal conductivity of unconsolidated material compared to bedrock. The site with glaciomarine silt and clay (Site 10) was not as responsive to climatic trends at the surface, in part due to the absorption of latent heat of fusion along the entire profile as unfrozen water content increases (Romanovsky and Osterkamp 2000). The temperature profile of Site 10 is isothermal (Figure 4e) and the absorption of latent heat of fusion as the ground warms stabilizes ground temperatures and completely impedes a thermal response to surface temperature trends. At such sites, monitoring permafrost response to climatic warming using temperature sensors is very difficult, as it cannot capture the progression of thawing along the profile in response to warming. Such isothermal profiles will likely become more widespread in the discontinuous zone, for instance in the Hudson Bay Lowlands where fine marine sediments abound. The use of geophysical methods such as nuclear magnetic resonance may help monitor changes in unfrozen water content along the profile and monitor the final decay of permafrost (Kass et al. 2017).

**Figure 6.** Active-layer thickness around October in Kangiqsualujjuaq, Nunavik, at sites 1 (glaciofluvial sand and gravel), 3 (bedrock), and 10 (glaciomarine silt and clay).

**Changes in Thaw Depth:** The early October depth of thaw in bedrock (Site 3) clearly responded to climate (Figure 6), with increasing thaw depths from the early 1990s to 2010, and decreasing thaw depths after 2011 when a cooling trend began. This response is, again, consistent with the high thermal conductivity and low moisture content (low porosity and minimal fracturing) of bedrock (Burn 2004). In glaciofluvial sand and gravel (Site 1) and in glaciomarine sediments (Site 10), the response of early October thaw depths to climatic variations was less pronounced and slightly delayed compared to bedrock (Figure 6). This is consistent with the buffering effects
of latent heat uptake as unfrozen water content increases in fine grained deposits nearing 0°C (Romanovsky and Osterkamp 2000).

SUMMARY AND CONCLUSIONS

This study examined ground temperature responses to climatic trends (late-1980s to present) in a range of geomorphological environments within the community area of Kangiqsualujjuaq (17.5 km²) in Nunavik (Figure 1). Similar to much of the eastern Canadian Arctic, air temperatures in the area cooled until the 1990s and warmed until around 2010 before cooling again. These changes are apparent in ground thermal regimes but the response at individual sites is controlled by the nature of surficial materials among other factors. In snow-free bedrock, where there was a deep active layer and a wide thermal envelope at depth, late fall thaw depths responded to air temperature trends and increased by nearly 5 m over 10 years, while mean air temperature increased from approximately -7.5°C to -3°C. Air temperature trends were effectively and promptly transmitted to 20-m depth in bedrock. In glaciofluvial sands and gravel and in a lithalsa made of glaciomarine silt and clay, thaw depth was more moderately responsive to climatic trends, reflecting the presence of ice-rich ground near the top of permafrost. The silt and clay deposits exhibited an isothermal profile at depths greater than 4 m, and the absorption of latent heat of fusion as the ground warmed and unfrozen water content increases along the profile now impedes the thermal response to surface temperature trends. This study shows that very different thermal regimes and responses to air temperature trends can occur in a restricted area that has a much smaller area than most climate model pixels, highlighting the importance of taking into consideration local to regional variations in terrain conditions, including in surficial deposits, when predicting permafrost response to climatic warming.

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Permafrost Investigations below the Marine Limit at Nain, Nunatsiavut, Canada

Robert G. Way, Ph.D.1; Antoni Lewkowicz, Ph.D.2; Yifeng Wang3; and Paul McCarney, Ph.D.4

1Assistant Professor, Northern Environmental Geoscience Laboratory, Dept. of Geography and Planning, Queen’s Univ., Kingston, ON (corresponding author). E-mail: robert.way@queensu.ca
2Dept. of Geography, Environment and Geomatics, Univ. of Ottawa. E-mail: alewkowi@uottawa.ca
3Northern Environmental Geoscience Laboratory, Dept. of Geography and Planning, Queen’s Univ. E-mail: yifeng.wang@queensu.ca
4Dept. of Lands and Natural Resources, Nunatsiavut Government. E-mail: pcm.mccarney@gmail.com

ABSTRACT

Discontinuous permafrost is a challenge for development in the coastal communities of Nunatsiavut, Labrador, northeast Canada, where local high relief limits suitable terrain for construction. These issues are particularly pronounced in Nain, the largest and northernmost community in Nunatsiavut, which is undergoing rapid population growth and expansion. In this study, DC electrical resistivity tomography was combined with geotechnical borehole records and in situ field data to evaluate the distribution of permafrost at four sites in the lowest parts of the community. Permafrost was identified in at least six of the seven geophysical transects, including beneath culturally critical community infrastructure. A supra-permafrost talik was imaged beneath a convenience store that has experienced extreme differential subsidence, demonstrating that excess ice exists in some of the region’s frozen sediments. The presence of permafrost near the shoreline likely reflects ground cooling due to wind-scouring of snow at the exposed sites and the thermal impact of frost-susceptible sediments. Despite uncertainties in geophysical interpretation due to local site disturbance and coarse near-surface fill at some sites, these results have important implications for future development in this northern coastal community.

INTRODUCTION

Projected warming of northern Canada is expected to affect permafrost bodies throughout the sporadic discontinuous zone over the next century (Woo et al. 1992, Zhang et al. 2008). Baseline information on local permafrost distribution and characteristics, as well as careful planning and adaptation measures, are required to avoid structural damage to existing and future infrastructure in these regions (Nelson et al. 2001, Smith and Riseborough 2010, Hong et al. 2014). In eastern Canada, permafrost-focused studies have been undertaken throughout Nunavik, northern Québec (e.g., Beaulieu and Allard 2003, Payette et al. 2004, Fortier et al. 2008, Allard et al. 2012), but there is a paucity of comparable information for communities in Nunatsiavut, northern Labrador.

In this paper, we present geophysical and local site investigations in Nain (56°32’N, 61°42’W; Figure 1), which examine the contemporary distribution of permafrost in the community and its implications for local infrastructure. Nain, which developed around a Moravian mission established in 1771, is both the northernmost and largest community in Nunatsiavut. Due to rapid population growth, systematic overcrowding in housing, and degrading existing infrastructure, there is an urgent need for community expansion, including the construction of new housing and a new airstrip. Suitable construction sites around Nain are limited because of topographic relief, hydrological conduits, and frost-susceptible marine deposits (Bell et al. 2011).
Nain is located in the sporadic discontinuous permafrost zone and ice content is inferred to be low to medium (Heginbottom et al. 1995, Way and Lewkowicz 2016). The overall impact of climate-induced permafrost degradation is expected to be medium, due to a projected low physical and moderate thermal response of soil to warming, but there is considerable uncertainty at finer spatial scales (Smith and Burgess 2004). Permafrost has been identified as a contributing factor to ground subsidence and structural damage to some buildings in the community (Bell et al. 2011, Smith and Melendy 2015, Way and Lewkowicz 2015), but permafrost distribution and conditions in the region have not been described in detail in recent academic literature.

Figure 1. A) Satellite image of Nain showing the location of (i) a rock excavation site near the community dump (Google Maps) and (ii) the community airstrip; B) Inset map situating Nain relative to Canada; C) Distribution of surficial materials in the centre of the community (Bell et al. 2011) showing the location of (iii) the Puffin Snacks convenience store, (iv) the post office, (v) the Moravian Church, and (vi) the Illusuak Cultural Centre.

STUDY AREA

Nain has a Subarctic climate with a mean annual air temperature of -2.5°C (1981-2010) and cool summers (~9.2°C; Environment Canada 2020). Regional mean annual air temperatures have increased by ~1.5°C since the early 20th Century, and significant warming has occurred in the region from 1987 to 2016 in both winter and summer, albeit with greater winter variability (Barrette et al. 2020). Mean cold-season snow thickness at Nain averages 67 cm (1981-2010; Environment Canada 2020), but a decrease in snow cover duration and a reduced fraction of solid precipitation have been observed in Nunatsiavut between 1980 and 2014 (Barrette et al. 2020).

Nain is situated in a WSW to ENE trending U-shaped valley, bordered to the north and south...
by high ridges and plateaux (200-250 m a.s.l.) of igneous bedrock of Proterozoic age (Wardle et al. 1997). Reworked tills are widespread in the community (Occhietti et al. 2011), while glaciofluvial sands and gravels are common in the valley floor, as are alluvial deposits (Bell et al. 2011). The marine limit at Nain is approximately 40 m a.s.l. (Vacchi et al. 2018), but Holocene marine sediments consisting of sands and muds have been observed only to elevations of ~20 m a.s.l. (Bell et al. 2011).

**METHODOLOGY**

**Field investigations:** Permafrost conditions were investigated using DC electrical resistivity tomography (ERT), frost probing, and instantaneous ground temperature measurements along seven transects below the marine limit in late-July of 2014 and 2018. Vegetation and surficial materials were also recorded. An ABEM Terrameter LS was used with electrodes in a Wenner array (1 or 2 m spacing) on profiles 40 to 200 m long (Table 1). Apparent resistivities were inverted with RES2DINV (Loke et al. 2003) using the robust inversion method until the modelled pseudosection’s errors declined by less than 1% between iterations (maximum of five iterations; Way et al. 2018, Davis et al. 2020). Inverted resistivities were exported as x-y-z values, and model blocks were plotted in R v3.6. Transects were topographically corrected prior to inversion with surface slope measurements collected with a Brunton compass or a clinometer, while starting elevations were determined with a hand-held GPS (± 5 m; Lewkowicz et al. 2011).

<table>
<thead>
<tr>
<th>ID</th>
<th>Site</th>
<th>Elevation (m a.s.l.)</th>
<th>Length (m)</th>
<th>Spacing (m)</th>
<th>Depth (m)</th>
<th>Survey Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Illusuak Cultural Centre (across)</td>
<td>6 to 4</td>
<td>200</td>
<td>2</td>
<td>26</td>
<td>2014-07-20</td>
</tr>
<tr>
<td>S2</td>
<td>Illusuak Cultural Centre (sloping)</td>
<td>9 to 3</td>
<td>120</td>
<td>2</td>
<td>23</td>
<td>2014-07-20</td>
</tr>
<tr>
<td>S3</td>
<td>Northern Shoreline (across)</td>
<td>11 to 7</td>
<td>80</td>
<td>1</td>
<td>15</td>
<td>2018-07-26</td>
</tr>
<tr>
<td>S4</td>
<td>Northern Shoreline (sloping)</td>
<td>12 to 6</td>
<td>106</td>
<td>1</td>
<td>20</td>
<td>2018-07-26</td>
</tr>
<tr>
<td>S5</td>
<td>Puffin Snacks (short)</td>
<td>7 to 4</td>
<td>40</td>
<td>1</td>
<td>7.5</td>
<td>2014-07-19</td>
</tr>
<tr>
<td>S6</td>
<td>Puffin Snacks (long)</td>
<td>8 to 7</td>
<td>80</td>
<td>2</td>
<td>15</td>
<td>2014-07-19</td>
</tr>
<tr>
<td>S7</td>
<td>Southern Shoreline</td>
<td>1 to 6</td>
<td>80</td>
<td>1</td>
<td>15</td>
<td>2014-07-20</td>
</tr>
</tbody>
</table>

Frost tables were measured along three ERT profiles using a 120 cm-long probe and were validated at selected locations with instantaneous ground temperature measurements collected with Onset Hobo UX120-006M 4-Channel Analog Data Loggers (±0.15°C; Way and Lewkowicz 2015, Davis et al. 2020, Holloway and Lewkowicz 2020). Ground temperatures were recorded at the base of the probed hole for 10 to 15 minutes to allow for thermal equilibration.

The probability of frozen ground in marine sediments along ERT transects was estimated with a statistical model developed from modelled resistivities and co-located frost table observations (e.g., Way et al. 2018). ERT model block resistivities, averaged from 0.50 to 1.05 m below the ground surface, were compared to co-located observations of frozen ground presence or absence determined by frost probing (Figure 2A). A maximum likelihood logistic regression model (p<0.01; AIC=37) was used to relate the near-surface model block resistivities to permafrost presence or absence for derivation of frozen ground probabilities in marine sediments in Nain (Figure 2B). This method offered a quantitative means of estimating permafrost distribution and was used to inform qualitative interpretations of ERT profiles.
Figure 2. A) Near-surface modelled resistivities at surveys S1, S3, and S4 grouped by absence or presence of frozen ground (see Table 1); B) Frozen ground probabilities estimated using a logistic regression of near-surface frozen ground absence or presence as a function of modelled resistivities for surveys S1, S3, and S4; C) End of ERT survey S4; D) ERT equipment, frost probe, and instantaneous temperature logger.

Ancillary Data: The Nunatsiavut Government made available geotechnical borehole logs collected at 80 locations throughout the community by Stantec Consulting Ltd. (Stantec 2012; n=9) and Exp Services Inc. (Smith and Melendy 2015; n=71). The Stantec boreholes were drilled during construction of the Illusuak Cultural Centre (Figure 1) in 2012 and described surficial materials to depths of up to 23.5 m. The Exp boreholes were drilled at selected locations within the community in 2014 and characterized surficial materials to depths of up to 10 m.

RESULTS

The seven ERT surveys were completed at four locations (Table 1; Figure 3). Depth of investigation ranged from 7.5 to 26 m (Table 1).

According to the probabilistic relationship shown in Figure 2B, frozen ground was present at all four sites investigated and in all seven profiles, with a minimum thickness of 0.5 m and a maximum thickness of greater than 20 m (Figure 4).

Illusuak Cultural Centre: ERT surveys S1 and S2 were performed across poorly drained medium to coarse sediments along the partially vegetated (grasses and sedges) shoreline zone and across a gravel pad prepared for the construction of the Illusuak Cultural Centre. Survey S1 ran parallel to the shoreline, traversing the gravel pad from 98 to 152 m along the profile (Figure 4A). Frozen ground probabilities were low in the first 70 m of the survey, except for a small body with higher resistivities at depths of 3 to 9 m near the start of the transect. This was interpreted as being a body of warm permafrost with a thick active layer (>120 cm) or possibly a supra-permafrost talik. Permafrost was inferred to occur from the surface or near-surface to a depth of 6 m between 68 and 92 m along the survey, from the surface or near-surface to a depth of 18 m between 92 and 152 m near the centre of the survey, and from depths of 2 to 8 m between 152 and 190 m near the end of the survey. Survey S2 ran perpendicular to the shoreline, beginning in a gravel lot near the community fire hall and traversing the Illusuak Cultural Centre construction site from 58 to 92 m along the survey before ending in the tidal zone (Figure 4B). Frozen ground probabilities were
high in the upper 1 to 2 m but were low at depth from 0 to 35 m along the survey, likely reflecting high resistivities within coarse fill in the near-surface, overlying unfrozen sediments at depth. Frozen ground was inferred to occur at 35 m along the survey and extended from the surface to the base of the profile, except for a body of unfrozen materials extending from the surface or near-surface to depths of 6 m beginning at 40 m along the survey.

Figure 3. ERT transects and boreholes completed in central Nain. Sections of ERT surveys inferred to be frozen (blue) or unfrozen (red) are shown for the Illusuak Cultural Centre S1 across and S2 sloping; Northern Shoreline S3) across and S4) sloping; Puffin Snacks S5) short and S6) long; and S7) Southern Shoreline surveys. S8 shows an ERT survey completed by Way and Lewkowicz (2015). Boreholes (Stantec 2012, Smith and Melendy 2015) are classified by presence of frozen or unfrozen sediments and the finest sediment layer present.

Drill logs from three boreholes near ~102, ~119, and ~135 m along survey S1 and near ~86 m along survey S2 show frozen sediments to a maximum depth of 22 m in August 2012 (Figure 3). An additional borehole drilled in November 2014 at ~23 m along survey S2 confirmed the absence of frozen ground in this section of the survey (Figure 3). These results are consistent with the inferred depth and distribution of permafrost from the ERT surveys performed in July 2014.
Figure 4. Frozen ground probabilities for marine sediments for the Illusuak Cultural Centre A) across and B) sloping; Northern Shoreline C) across and D) sloping; Puffin Snacks E) short and F) long; and G) Southern Shoreline ERT surveys. Frozen ground likelihood was estimated with binomial logistic regression using measured resistivities and thaw depths.

Northern Shoreline: Surveys S3 and S4 were located near the shoreline about 250 m from the end of the airstrip. The ground surface was covered by sedges, grasses, and low shrubs, with several tall patches of willows (Salix spp.) measuring up to 2 m. Survey S3 (Figure 4C) ran parallel to the shoreline through two distinct patches of willows. Frozen ground apparently extended from the surface to the base of the profile along the entire survey, although a short section from 19 to 22 m along the transect was inferred to exhibit ~1 m of thaw from the surface. The latter corresponded to the centre of a 20 m-long patch of willows, measuring about 1 m high. Survey S4 ran perpendicular to the shoreline, starting in a tall willow patch (>1.5 m), running through alternating sections of grass and shrubs, before ending on unvegetated beach sands and gravels (Figure 4D). Frozen ground probabilities were highest in the middle of the survey, while ground was inferred to be unfrozen at the start of the survey, beneath tall willows and wet grasses, and at the end of the survey, on the grassy to unvegetated beach. Frozen ground likely extended from the
near-surface to the base of investigation between 16 and 84 m along the survey, although deeper thaw was inferred past 71 m along the survey. The latter could represent the presence of a talik or may be due to saltwater incursion from the adjacent inlet. A 1 m-thick thawed section between 36 and 42 m along the profile corresponded to the edge of a patch of willows (~1 m) and a winter snowmobile trail. Resistivities at the Northern Shoreline site were higher than those observed at the nearby Illusuak Cultural Centre site, suggesting greater frozen moisture contents. A number of possibilities could account for this difference, including colder ground temperatures, lower salinities, higher ground ice contents, and coarser sediments. The two boreholes nearest to the ERT profiles were both in sands (Figure 3) and were unfrozen to completion depths of 4.9 m.

**Puffin Snacks:** Surveys S5 and S6 were conducted across an open yard and two gravel paths adjacent to the Puffin Snacks convenience store. The building exhibited a ~5° longitudinal tilt due to differential settlement, hypothesized to be due to permafrost thaw. The two surveys shared a mid-point, and the side of the building was located between 7.5 and 19.5 m in survey S5 and 27.5 and 39.5 m in survey S6 (Figure 4E-F). High modelled probabilities of frozen ground were evident in the upper 2 metres along parts of both surveys, including below the side of the building. However, these are likely due to deep seasonal frost and/or coarse unfrozen fill because frozen ground probabilities were much lower below a depth of 2 metres. In the longer and deeper survey S6, frozen material was interpreted at depths of 11 to 15 m at 45 m along the survey and from the surface to the base of the profile from 60 to 65 m along the survey. Thus, permafrost was inferred to exist to the base of the profile (>15 m) between 45 and 65 m, beneath a supra-permafrost talik up to 8 m thick.

A borehole drilled to a depth of 10.1 m in November 2014 about 3 m downslope of the overlapping section of surveys S5 and S6, at ~14 m along survey S5 and ~34 m along survey S6 (Figure 3), found a 3 m-thick body of frozen ground beneath a 3.5 m-thick unfrozen layer. Temperatures at depth, measured in November 2014, were just below 0°C. This borehole shows the presence of degrading thin permafrost, while the results from surveys S5 and S6 indicate an absence of frozen ground immediately next to the building in the same year.

**Southern Shoreline:** Survey S7 was performed across a range of surface and vegetation conditions adjacent to a playground on the community’s southern shoreline (Figure 4G). The transect started in unvegetated sandy foreshore that transitioned upslope to grasses and sedges, then dwarf birch (*Betula glandulosa*) and willow shrubs, to a patch of open forest composed of balsam fir (*Abies balsamea*) and tamarack (*Larix laricina*), before ending on a gravel pad. Frozen ground was inferred to occur at this site to a maximum depth of 0.5 to 1.5 m beneath a section of low shrubs (<40 cm) between 44 and 58 m along the survey. Frost probing and instantaneous temperature measurements confirmed the presence of frozen ground at these depths, but probing resulted in penetration through the frozen layer at some locations, suggesting that the thin body of frozen ground at this site was late-lying seasonal frost rather than permafrost. The two closest boreholes showed unfrozen sediments to completion depths of up to 9.15 m. Consequently, the moderate resistivities and medium frozen ground probabilities at depths exceeding 5 to 7 m are probably not bedrock and are tentatively interpreted as degrading permafrost.

**DISCUSSION**

**Permafrost Distribution in Marine Sediments in Nain:** The borehole logs suggest that permafrost is not widespread in Nain. Unfrozen ground underlies much of the community where sand is usually the finest dominant sediment type (Stantec 2012, Smith and Melendy 2015; Figure 3). Only the boreholes drilled at the Illusuak Cultural Centre construction site and near the Puffin...
Snacks convenience store intersected frozen ground. However, permafrost presence has been reported elsewhere in low-lying marine deposits within the community. For example, construction workers observed ice-rich frozen ground at depths of 3 to 4 m in a 25 m-long section along a water and sewage line ditch near the community’s southern shoreline and to a depth of at least 12 m during drilling and blasting of rock near the community dump (Jamie Ryan, Budgell’s Equipment and Rental Ltd., personal communication, January 26, 2021; Figure 1).

The development of permafrost and its contemporary state adjacent to the shoreline is likely due to lower ground temperatures caused by extensive wind scouring and densification of snow in the open area near the shoreline and by the thermal impact of frost-susceptible soils. Permafrost is present in similar low-lying locales within fine-grained marine sediments in Nunavik (Fortier et al. 2008), including along the coast of Hudson’s Bay (Beaulieu and Allard 2003). There are several indications that permafrost distribution has been impacted by the built environment in the community. Part of the Illusuak Cultural Centre site was occupied by a caribou processing plant constructed in 1988 that suffered severe subsidence and was demolished after only 5 years (Meis Mason et al. 2007). The Puffin Snacks store, which is still in use despite its tilt, has obviously been affected by localized ground warming. Ground subsidence has occurred along the community airstrip (Mitchell 2018), and the current Moravian Church, constructed in 1923, has had to be jacked up by at least 0.75 m to level it (Smith and Melendy 2015).

This local knowledge, combined with recent geophysical and geotechnical investigations, including in nearby forested upland locations (Survey S8 in Figure 3; Way and Lewkowicz 2015), suggests that the combined effects of more than two centuries of permanent structures and widespread surface disturbance, including the clearing of vegetation for built infrastructure, may have degraded formerly present permafrost bodies throughout the community, leaving mostly residual patches near the shoreline. This interpretation is supported by modelling studies by Zhang et al. (2006) and Way and Lewkowicz (2016), which suggest that areas around Nain have experienced widespread permafrost degradation in response to regional warming over the past century. Due to the variable ground thermal and hydrological conditions within the community, the establishment of a permafrost thermal monitoring program would improve our understanding of regional permafrost susceptibility to future climate change (e.g., Smith et al. 2005).

Challenges of Detecting Permafrost in Coastal Labrador: Interpretation of permafrost conditions in Nain was challenging for several reasons. First, additional validation sources were required to inform interpretations of the ERT surveys. The probabilistic statistical model that was developed from modelled resistivities and frost table observations offered a quantitative means of interpreting otherwise qualitative results, but it relied on the assumption of a homogeneous substrate within each profile and between all profiles, which may not be the case. Second, the interpretation of frozen versus unfrozen ground was further complicated by the presence of coarse surficial materials in some surveys, including coarse fill and/or gravel roads or surfaces at the Illusuak Cultural Centre, Puffin Snacks, and Southern Shoreline sites. Unfrozen dry coarse materials exhibit resistivities that can overlap with those obtained in frozen ground. Third, coastal Labrador is characterized by cool summers (Maxwell 1981, Way et al. 2018), which may preserve seasonal frost long into the summer, such as at the Puffin Snacks and Southern Shoreline sites. Discrimination between seasonally and perennially frozen ground in Nain was not always possible using techniques such as frost probing and instantaneous ground temperature measurements. Fourth, ERT profiles and field data indicated the presence of deep active layers (>120 cm) or supra-permafrost taliks at the Illusuak Cultural Centre and Puffin Snacks sites. Without long-term temperature monitoring at depth, it is not possible to conclusively differentiate between the active
layer and a talik, thus complicating our understanding of permafrost stability.

CONCLUSION

In this study, ERT surveys, field observations, geotechnical reports, and local knowledge were combined to characterize permafrost bodies within low-lying marine deposits in Nain. Permafrost was present to depths of at least 20 m at two sites, has partially degraded at a third site, and may be absent or present only beneath a talik at a fourth. The state and persistence of permafrost along the Nain shoreline is attributed to local factors, including frost-susceptible soils and thin layers of high-density snow in response to the low vegetation and the wind-exposed locale. The presence of deep active layers, supra-permafrost taliks, and buildings experiencing differential settlement demonstrates permafrost degradation beneath built infrastructure, indicating that permafrost extent may have been greater in the past. The findings underline that permafrost must be considered during community expansion and site-selection initiatives in Nain, particularly at sites adjacent to the shoreline. Long-term ground thermal monitoring would be an effective means of improving our characterization of regional permafrost risks and hazards.

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REFERENCES


Mercury, Methylmercury, and Microbial Communities in a Degrading Palsa of the Hudson Bay Lowlands, Far North Ontario

Adam H. Kirkwood⁴; Pascale Roy-Léveillé²; Brian A. Branfireun³; and Nathan Basiliko⁴

¹Dept. of Biology, Vale Living with Lakes Centre, Laurentian Univ., Sudbury, ON, Canada. E-mail: akirkwood@laurentian.ca
²Dept. of Geography, Laval Univ., Quebec, QC, Canada. E-mail: pascale.roy-leveillee.1@ulaval.ca
³Dept. of Biology, Western Univ., London, ON, Canada. E-mail: bbranfir@uwo.ca
⁴Dept. of Biology, Vale Living with Lakes Centre, Laurentian Univ., Sudbury, ON, Canada. E-mail: nbasiliko@laurentian.ca

ABSTRACT

The Hudson Bay Lowlands (HBL) may be one of the largest mercury (Hg) pools in the permafrost zone according to recent estimates. However, little is known about the abundance and distribution of organic methylmercury (MeHg), and it is unclear how permafrost degradation relates to the release and potential methylation of Hg. This research characterized total Hg and MeHg distributions in a degrading palsa of the HBL and investigated relations between thawing permafrost, MeHg concentrations, and microbial community structure. Near the study site, palsas lost ~1.0% of their area per year to thermokarst encroachment between 1955 and 2019. The abundance of microbes in families with taxa capable of Hg methylation was not related to MeHg concentrations, which were low (0.13–0.49 ng/g), and strongly related to THg concentrations. Hg concentrations in the top 100 cm of the peat profile were lower in this study than previously estimated for the area, with a storage of approximately 9 mg THg m⁻².

INTRODUCTION

The Hudson Bay Lowlands (HBL) encompasses the world’s second largest northern peatland complex and North America’s lowest latitude continuous permafrost (Figure 1). Permafrost in the HBL has been degrading over the last several decades as can be seen by the continued reduction in area of palsas and peat plateaus (Pironkova, 2017). Extensive permafrost thaw may lead to stores of inorganic mercury (Hg) in frozen soils becoming a more active part of biogeochemical cycling. Schuster et al. (2018) estimate that 81-150 mg Hg m⁻² are stored in the top 100 cm of the peat/permafrost profiles of the HBL. This is of concern to fish consumers due to the potential for this inorganic Hg to be converted to its organic, bioavailable, bioaccumulative and neurotoxic form methylmercury (MeHg) as permafrost in the HBL thaws.

Palsas and peat plateaus are common permafrost features in the HBL, and are mounds of frozen peat that have been raised above the surrounding unfrozen wetland by the formation of segregated ground ice (Gurney, 2001). For palsas and plateaus, a dominant mechanism of permafrost thaw is through thermokarst encroachment. This is where the lateral transfer of heat from the warmer unfrozen wetland to the ice-rich core of permafrost leads to thaw, resulting in subsequent collapse of the palsa/plateau and amalgamation of these peats into a thermokarst wetland (henceforth referred to as ‘thermokarst fen’; Matthews et al., 1997). In non-permafrost wetlands, net Hg methylation is highest in waterlogged, anaerobic, vegetated systems (Haynes et al., 2019). Therefore, permafrost thaw via thermokarst encroachment that leads to wet and anaerobic conditions, sustained warmer temperatures, and changes in vegetation may create environments...
that are more suitable for Hg methylation (Dyke & Sladen, 2010; Laberge & Payette, 1995). Indeed, elevated concentrations of methylmercury have been reported from some permafrost environments affected by thermokarst (Gordon et al., 2016), however data are limited, and none exist for the HBL.

The conversion of inorganic Hg to MeHg is carried out by anaerobic microorganisms that contain the hgcAB gene pairs responsible for encoding proteins that enable Hg methylation (Christensen et al., 2019) including sulfate-reducing bacteria (SRB), iron-reducing bacteria (FeRB), methanogenic archaea and syntrophic bacteria (Tang et al., 2020). The biophysical conditions created by thermokarst encroachment may remove some limitations on methylation for microbial communities (Mackelprang et al., 2011). The abundance and activity of these microbial communities are subject to substrate constraints such as from sulfate (SO$_4^{2-}$) that acts as an electron acceptor in the Hg methylation reaction and organic C that acts as an electron donor (Mitchell et al., 2008) as well as other substrates such as oxidized iron and nitrate (Kerin et al., 2006). As permafrost thaws and ground temperatures warm, base cations (Ca$^{2+}$, K$^+$, Na$^+$, etc.) and other macro and micronutrients (Fe, Mg, S, etc.) previously sequestered in permafrost are released leading to a more alkaline and nutrient rich environments that may become more supportive of microbial communities (Kokelj & Burn, 2003). Changes in hydrology as a result of permafrost thaw can lead to increased connectivity to ground water supplying otherwise limiting nutrients for Hg methylators (Gordon et al., 2016).

Figure 1: Map of the Hudson Bay Lowlands showing permafrost zones (Heginbottom, 2005) and location of study sites in Polar Bear Provincial Park. A) shows an oblique photograph of a palsa of similar size and structure to the case study palsa selected. B) shows digitized area of palsa as interpreted from historical aerial photography from 1955 (grey outline) and 1976 (blue outline).

Wet lowland environments created as a result of thawing permafrost have been identified as an important potential habitat for Hg methylating microorganisms (Podar et al., 2015). However, the community structure of Hg methylating microbes in thawing environments is unknown and its relationship with MeHg concentrations under different field conditions has not been examined. In the HBL, there is little data regarding Hg concentrations in permafrost and permafrost thaw.
features. Similarly, environmental controls to the conversion of inorganic Hg to bioavailable methylmercury by anaerobic microbes have not been explored in this rapidly changing region (Kerin et al., 2006). Therefore, the objectives of this study were to: 1) characterize the Hg pool in palsa fields of the Hudson Bay lowlands in northern Ontario; 2) examine the distribution of MeHg, THg and %MeHg along peat profiles in these palsa fields; and 3) investigate relations between microbial community structure and MeHg concentrations in a palsa and adjacent thermokarst fen.

STUDY AREA

The Hudson Bay Lowlands has an approximate area of 372,000 km$^2$ and is characterized by a continuous extent of peatlands covering ~90% of the area (Riley, 2011). Peatland development in the HBL began ~7 ky BP, following deglaciation of the Laurentide Ice Sheet and the gradual retreat of the Tyrell Sea forming a lowland landscape as the area isostatically rebounded (Glaser et al., 2004). As a result of this lowland development, peatlands in the HBL have accumulated ~30 Pg of carbon in peat (Packalen et al., 2014). Centuries of dry and wet atmospheric Hg deposition and binding to organic matter led to a large accumulation of Hg storage in the extensive peat deposits, however, the concentration of Hg likely remains relatively low. Based on estimations of carbon storage in the HBL, Schuster et al. (2018) estimated that between 81-150 mg Hg m$^{-2}$ may be stored in the top 100 cm of the peat profile, and >150 mg Hg m$^{-2}$ in the top 300 cm, making the HBL one of the largest pools of Hg in permafrost regions. These stores of carbon and Hg were predominantly immobilized as they accumulated due to permafrost aggradation in the HBL. Permafrost aggradation began following deglaciation and isostatic rebound as land emerged from the Tyrell Sea and became exposed to the cool microthermal climate of the area (Boisson et al., 2020; Rouse, 1991). In the HBL today there is a narrow zone of continuous permafrost remaining along the northern coast, which quickly transitions inland to a zone of extensive-discontinuous and sporadic-discontinuous permafrost (Figure 1). In the Ontario portion of the HBL, permafrost predominantly occurs in palsas and peat plateaus.

METHODS

Site selection: In 2017, a 150 km$^2$ watershed in Polar Bear Provincial Park, ON was delineated by the Ontario Ministry of Natural Resources and Forestry (OMNRF), and 10-m resolution SPOT imagery was used to select five palsa fields throughout the area. One palsa-thermokarst pair was selected to represent typical palsa size, shape, and vegetation cover in each field. This paper uses core samples (see below) from the first of the five palsas, which was selected as a case study to examine total and methylmercury distribution in relation to microbial community structure along intact and degraded permafrost and soil profiles.

Air photo analysis: To assess palsa degradation rates at the study site, twenty palsas located in a 25 km$^2$ area centered on the palsa used herein as a case study were manually digitized on historical aerial photographs from 1955 and 1976 (800 dpi) georeferenced in ArcGIS Desktop (Version 10.7.1) and on the recent (2019) high-resolution satellite imagery basemap in ArcGIS Desktop using riverbanks and other features that had not changed over time as reference points.

Field sampling and laboratory analyses: A core of active layer peat (0-35 cm) was collected using a knife, and a core of permafrost (35-91.5 cm) was extracted from a palsa using a portable Earth Drill System (Calmels et al., 2005). A peat core (0-44 cm) was also collected from the adjacent thermokarst fen using a box corer. Sampling took place in late August 2017, when thaw depth was near its maximum, providing an estimate of active layer thickness. Cores were kept frozen and stored at -20°C until they were subsampled in 10 cm depth increments for analysis of
Hg concentrations and microbial communities. Subsamples were cut into ~ 1 cm³ blocks and used to calculate bulk density after drying for 24 hours at 105°C, and used to calculate volumetric contents of total Hg (THg) and MeHg. Subsamples were also assessed for their organic matter content via loss on ignition in a muffle furnace at 550°C for 5 hours (Dean, 1974).

**THg and MeHg Concentrations:** Subsamples of each 10 cm depth were sent to the Biotron Institute for Experimental Climate Change Research (CALA ISO 17025 accredited) trace metal analytical facility for mercury analysis using ultratrace methods on freeze dried samples. A Milestone DMA-80 mercury analyzer was used for total THg (USEPA 7473, 2007), and after distillation of samples, cold vapour atomic fluorescence spectroscopy (CVAFS) on a Tekran 2700 was used to quantify methylmercury MeHg (USEPA 1630, 2001). For Quality Assurance and Quality Control, it was made certain that blank samples had unquantifiable levels of THg and MeHg, and replicate samples had a variation of <20%. To compare our THg concentrations to the estimates from Schuster et al. (2018), we calculated RTHgC, which is the ratio of THg to soil organic carbon (estimated with LOI) and calculated the carbon stores at our site to estimate the storage of THg in the top 100 cm of peat in the HBL.

**DNA Extraction and Sequencing:** Microbial community analysis was performed on palsa (active layer and permafrost) and thermokarst core samples. For microbial community analysis, DNA was extracted from each 10 cm subsection of active layer, permafrost, and thermokarst using a Qiagen DNEasy Power Soil Kit (Qiagen, Germantown, MD, USA) following the manufacturer’s instructions and eluted to a final volume of 100 µL. DNA extracts were assessed with absorbance spectroscopy methods, and sent to Metagenome Bio Inc. (Toronto, Canada) for sequencing of the V3-V4 region on the Illumina MiSeq platform (Illumina Biotechnology CO., San Diego, CA, United States) using SSU rRNA gene primers 515FB and 806RB that captures both bacteria and many archaea (Walters et al., 2016). Following sequencing, data were processed using the DADA2 pipeline, where sequence data were checked for quality, and assigned into amplicon sequence variants (ASVs) to assign taxonomy using the Silva version 132 database (Callahan et al., 2016).

**Analysis:** Following taxonomic assignment, Shannon diversity was calculated using the estimate richness function in phyloseq in R, which assesses the species richness relative to the abundance of total species present (McMurdie & Holmes, 2013; R Core Team, 2018). After calculating Shannon diversity, the dataset was filtered of singleton and doubleton sequences and transformed to relative abundance using the transform samples count function of phyloseq, and filtered by removing any taxa with a mean less than 10⁻⁵ (McMurdie & Holmes, 2013). To examine for relationships between Hg concentrations and categorical variables such as depth and site (palsa vs. thermokarst), one-way ANOVAs were utilized. For assessing the relationship between abundance of microbes considered likely Hg methylators, general linear regressions including Pearson’s correlations were calculated. The fraction of THg that occurs as MeHg (%MeHg) was calculated and used as an indirect and relative indicator of net Hg methylation potential.

**RESULTS**

**Loss of palsa area:** Since 1955, there has been considerable loss in the areal extent of the 20 palsas mapped in the palsa field around the study site, ranging from a loss of 26% to complete degradation (100%). Between 1955 and 1976, palsas lost an average of 21% (1.0% per year) of their extent, and between 1976 and 2019 they lost on average 59% (1.3% per year). This amounts to an average loss of 66% of their extent between 1955 and 2019, amounting to a loss of ~35 m² per year (or 1.0% area per year). As palsas degraded, the thawing of ice-rich permafrost led to surface subsidence and the conversion of palsa into thermokarst fen (Figure 1).
THg and MeHg concentrations and distribution: There was a wide range in THg concentration among palsa and thermokarst peats. Active layer had the highest THg concentration, followed by thermokarst and finally permafrost (Table 1). There was no significant differences in MeHg concentrations among cores of palsa or thermokarst, but similar to THg, absolute concentrations were highest in the active layer, then thermokarst and finally permafrost samples (Table 1). The concentrations of THg and MeHg were strongly correlated, where higher THg concentrations led to larger MeHg concentration (r = 0.80, p = <0.001). The same correlation was observed when regressing inorganic Hg (MeHg subtracted from THg) with MeHg. %MeHg was highest in permafrost, followed by thermokarst and active layer (Table 1). When considering volumetric concentrations of THg and MeHg by factoring in bulk density of the samples, the trends between peats were the same as when presented using gravimetric concentrations. Both THg and MeHg concentration decreased with depth in the soil profiles of both the palsa and thermokarst (p = 0.174 and p = 0.018, respectively; Figure 2). %MeHg decreased with depth in thermokarst but showed no clear trend with increasing depth in the palsa, aside from a very high value at 70 cm depth, just above the interface with the mineral soil (Figure 2).

Table 1: Average concentrations of total mercury (THg) and methylmercury (MeHg), and ratio of THg occurring as MeHg (%MeHg) for active layer, permafrost, and thermokarst samples from our study site.

<table>
<thead>
<tr>
<th>Peat type</th>
<th>THg (ng/g)</th>
<th>MeHg (ng/g)</th>
<th>%MeHg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Palsa (Active Layer)</td>
<td>241.3</td>
<td>0.49</td>
<td>0.20</td>
</tr>
<tr>
<td>Palsa (Permafrost)</td>
<td>16.4</td>
<td>0.13</td>
<td>0.83</td>
</tr>
<tr>
<td>Thermokarst</td>
<td>43.9</td>
<td>0.33</td>
<td>0.75</td>
</tr>
</tbody>
</table>

We found an average RTHgC of 0.21 µg THg g C⁻¹ at our study site. We used this average and applied it to our calculated carbon store of ~43 kg C m⁻², which led to an estimate of 9.5 mg THg m⁻² at our study site. The RTHgC of 0.21 µg THg g C⁻¹ agreed well with the RTHgC from the five other palsas included in the OMNRF monitoring campaign, which was 0.22 µg THg g C⁻¹, indicating that our study site was representative for Hg stored in the top 100 cm of the peat profile in our delineated watershed.

Microbial community structure and diversity: At our study site, Shannon diversity (an integrated measure of both the richness and evenness of microbial taxa) was highest in thermokarst and lowest in permafrost. There was no significant relationship between Shannon diversity and MeHg concentrations (p = 0.27). A general linear regression of abundance of taxa in groups that predominantly contain Hg methylating organisms (as defined in Figure 3) and concentration of %MeHg showed no significant relationship, though this is likely due to the small sample size analyzed as part of this case study (n = 13). Organisms likely capable of mercury methylation were present in all samples with the exception of the surface layers of the palsa (Figure 3). They comprised <10% abundance in the palsa profile, and <15% abundance in the thermokarst profile. Members of the family Syntrophaceae were the most abundant Hg methylating organisms and present in all samples, follow by small abundances of Geobacteraceae and various others. In the palsa, Hg methylating microbes were absent on the 10-20 cm horizon, and occurred in very small abundance in the 20-30 cm horizon. In the thermokarst profile there was a near absence of methylating organisms in the 10-20 horizon, similar to the palsa profile, but followed by a steady increase with greater depth. This trend was the opposite of the MeHg concentrations observed in the thermokarst, which were higher at the surface and decreased with depth (Figure 2 and 3). Hg
methylating organisms increased in abundance deeper in the active layer, and in permafrost, we observed no clear trends in abundance (Figure 3).

![Figure 2: Measurements of methylmercury, total mercury, %MeHg and bulk density along the depth profile of palsa and thermokarst. The dashed black line indicates the depth of thaw at the time of sampling. Contact with mineral sediment on the palsa was made at 80 cm.]

DISCUSSION

**RTHgC and storage of THg:** Following the methods by Schuster et al. (2018) for calculating RTHgC we found an average value at our site (0.21±0.16 µg THg g C⁻¹). This value was considerably lower than the value used by Schuster et al. (1.6±0.9 µg Hg g C⁻¹) in their estimate of circumpolar Hg stocks in the permafrost areas of the HBL. Our mean RTHgC of 0.21 µg THg g C⁻¹ is lower than estimates from other types of environments reported by other studies. For example, in sub-Antarctic soils Peña-Rodríguez et al. (2014) found 1-11.3 µg Hg g C⁻¹, Smith et al. (2005) found 0.26-4.88 µg Hg g C⁻¹ along a latitudinal gradient in Manitoba, and in the global compilation of data presented by Schuster et al. (2018) the range was 0.0-10 µg Hg g C⁻¹. Applying RTHgC to estimated stocks of carbon from our study site led to an estimate of Hg stores (9.5 mg THg m⁻²) that was much lower than the 81-150 mg Hg m⁻² estimated by Schuster et al. RTHgC from the case study site and the average of the five OMNRF palsas monitored are very similar, suggesting that the results presented may be accurately scalable across the 150-km² watershed where we sampled. However the palsa studied here is in the Far North of Ontario (Figure 1) where peat deposits are younger and thinner than more southern portions of the HBL (Packalen et al., 2014). Therefore, Hg stores in the top meter and three meters of the ground may be higher in more southern portions of the HBL.
Figure 3: Families of known Hg methylating organisms shown by depth in the palsa (A) and thermokarst (B) profile. Thaw depth in the palsa at time of sampling was 35 cm (indicated by dashed line in A), and contact with mineral sediment was made at 80 cm.

**THg distribution in the soil profile**: THg concentrations were highest in the active layer and decreased through depth in the palsa profile (Table 2; Figure 2). These results are similar to what is reported elsewhere, where THg concentrations are higher in the surface active layer reflecting post-industrial anthropogenic mercury emission and deposition over the last ~200 years (Bandara et al., 2019). However, the average THg concentrations from active layer peat (241 ng THg g\(^{-1}\)) was higher than reported from other regions. For instance, the Stordalen Palsa Mire in Sweden had concentrations ranging from 100-150 ng THg g\(^{-1}\) (Rydberg et al., 2010), and the Central Yukon had a range of 8-100 ng THg g\(^{-1}\) (Bandara et al., 2019). In permafrost, our average THg concentration was lower than the studies reported above, where we found an average of 16.4 ng THg g\(^{-1}\), while others reported ranges between 20-60 ng THg g\(^{-1}\). THg in the active layer and THg released from permafrost thaw is subject to increased mobility, which results in the movement of THg to thermokarst ponds, or in the case of palsa ecosystems, thermokarst fens (Mu et al., 2020). This trend is reflected in our thermokarst profile which had values falling between those of active layer and permafrost, where THg may move laterally as a result of permafrost thaw and enhanced hydrological connectivity in the active layer (Table 2; Bandara et al., 2019). This increase in THg was seen in the top 20 cm of thermokarst where the concentrations were highest, followed by a steady decrease.

**Distribution of MeHg**: Permafrost thaw and subsequent ground subsidence leads to the transition from palsa to thermokarst fens, causing previously dry environments to become saturated and exposed to a warmer thermal regime (Dyke & Sladen, 2010). Hg methylation potential has been shown to increase in sediments with warmer temperatures (Hudelson et al., 2020), which may therefore cause thermokarst areas to become hotspots for Hg methylation. Here, we found that MeHg concentrations in thermokarst were lower than in the active layer, but were higher than in permafrost (Table 2). However, the significantly higher MeHg in the active layer at the case study site is likely the result of higher THg in comparison to thermokarst. Therefore, using %MeHg (the fraction of THg occurring as MeHg) is an important proxy for determining environments are more likely to methylate Hg, or to accumulate MeHg post-mobilization (Gordon et al., 2016). This is similar to other studies that have found increased MeHg concentrations in pore waters of thermokarst fens and ponds as a result of permafrost thaw (Gordon et al., 2016).
However, despite other studies concluding that thermokarst fens and ponds are more suitable for Hg methylation, we did not find large enough differences in MeHg concentrations or %MeHg between palsa and thermokarst samples to conclude that the thermokarst fen at our study site had a notably high net methylation potential.

**MeHg and microbial communities:** Permafrost thaw has significant implications for microbial communities, altering community structure and predominant metabolic pathways (Mackelprang et al., 2011). While considerable efforts have been focussed on elucidating the role of changing microbial communities in C cycling (Mondav et al., 2014), a critical knowledge gap remains regarding the roles of changing microbial communities in thawing permafrost on Hg methylation (Podar et al., 2015). In this study, we find relatively low abundances of taxonomic groups containing Hg methylating organisms in our samples, which is reflected by low concentrations of MeHg in addition to low values of %MeHg (Table 1). In a similar study, Fahnestock et al. (2019) found that MeHg was higher in thermokarst fens and nearly absent in palsa samples. This corresponded to higher abundances of Hg methylating organisms in thermokarst fen (~7%), and near absence in palsas (<0.5%). This is in contrast to our study, where we found very similar average MeHg and %MeHg across palsa and thermokarst cores (Table 1). This may suggest that the similar abundances of methylating organisms between palsa and thermokarst fen cores (Table 3) may be responsible for the only small differences in MeHg concentrations. However, a limitation to our study and the study cited above was the use of the SSU rRNA gene as a phylogenetic marker; the resolution may not be fine enough to specifically target microbes containing the hgcAB+ gene that encodes the proteins responsible for Hg methylation in microbes. Therefore, the ability to infer the potential for Hg methylation via soil microbes may be limited, as it relies on the previous taxonomic identification of Hg methylating organisms (Christensen et al., 2019). Though the approach used in our study can be used to broadly infer Hg methylating microbial communities, the future direction of work on Hg methylation via soil microbes in thawing permafrost should focus on more targeted approaches to assess changes in community structure and function.

**CONCLUSION**

The purpose of this paper was to contribute new knowledge of Hg distribution in permafrost and thermokarst using a case study from the HBL and to determine if the composition of microbial communities is linked to Hg methylation potential. We found that:

1. THg concentrations at our site were substantially lower than from recent estimates of Hg derived from SOC pools for the circumpolar north;
2. Concentrations of THg and MeHg decrease with depth in active layer and thermokarst profiles, and that MeHg concentrations reflect patterns of THg concentrations;
3. The abundance of taxa purportedly capable of Hg methylating in this study did not relate to MeHg concentrations or %MeHg, and more targeted methods or larger datasets are required to accurately assess Hg methylation potentials via soil community analyses.

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REFERENCES


Impacts of Shrubification on Ground Temperatures and Carbon Cycling in a Sub-Arctic Fen near Churchill, MB.

Chantae Robinson¹; Pascale Roy-Léveillé²; Kevin Turner³; and Nathan Basiliko⁴

¹Laurentian Univ., Dept. of Biology and the Living with Lakes Centre, Sudbury, ON. E-mail: crobinson1@laurentian.ca
²Université Laval, Geography Dept., Québec, QC; Laurentian Univ., Dept. of Biology and the Living with Lakes Centre, Sudbury, ON. E-mail: Pascale.Roy-Leveillee@ggr.ulaval.ca
³Brock Univ., Geography and Tourism Studies, St-Catharines, ON. E-mail: kturner2@brocku.ca
⁴Laurentian Univ., Dept. of Biology and the Living with Lakes Centre, Sudbury, ON. E-mail: nbasiliko@laurentian.ca

ABSTRACT

This paper examines the extent and impact of shrubification on near surface ground temperatures and microbial greenhouse gas production (GHG) in fen environments of the Hudson Bay Lowlands near Churchill, MB. Greening during 1984–2018 was analyzed using Google Earth Engine, temperature sensors were installed just below the ground surface, and active layer samples were incubated to determine potential GHG production. Greening was extensive in the area and, at the site, mean annual ground surface temperature was more than 3°C warmer under shrubs, with twice as many thawing degree-days and half as many freezing degree-days than in sedge dominated fen sites. Methane production was lower in soils from shrub-dominated sites. These preliminary results suggest it is unlikely that permafrost is sustainable where shrubs encroach, yet the effects of permafrost thaw on carbon cycling could be in part offset by lower microbial methane production associated with shrubs in near surface soils.

INTRODUCTION

Mean annual air temperatures in Arctic regions have increased by about 2–3°C over the past 50 years, which is almost twice the rate of the rest of the world (Elmendorf et al., 2012). As a consequence of increased air temperatures, the abundance and maximum height of woody shrub species has increased (shrubification) at sites around the circumpolar Arctic (Myers-Smith et al., 2009; Epstein et al., 2012). Shrubs form thickets that alter the surface energy balance by modifying albedo and shading during summer and snow cover during winter. This impacts the ground thermal regime by either increasing or decreasing soil temperatures according to the season (Myers-Smith et al., 2011). The response of ground thermal regime to shrubification also varies from place to place as some studies report an overall cooling (Loranty et al., 2018) and others report overall warming (Kropp et al., 2020; Loranty et al., 2018; Aalto et al., 2018). Shrub canopy abundance influences active layer depth, and active layer deepening is of concern for the release of greenhouse gas (GHG), such as carbon dioxide and methane, from the microbial decomposition of organic compounds previously stored in permafrost (Schuur et al., 2008; Blok et al., 2010). Understanding the impacts of shrubification on ecosystem functions will help improve projections of future carbon storage and global warming (Bonfils et al., 2012; Loranty et al., 2018). Therefore, this paper examined changes to ground temperatures and microbial greenhouse gas production in Hudson Bay Lowlands fen environments affected by shrubification near Churchill, MB (Figure 1). The objectives were to: (1) assess patterns of shrub encroachment near Churchill, Manitoba, (2) characterize the soil thermal regime for locations affected and unaffected by shrubification;
and (3) assess potential GHG production in areas dominated by shrubs and sedges in this fen.

Figure 1. The study area is located near treeline in the continuous permafrost zone, near Churchill, Manitoba, in the Hudson Bay Lowlands (modified from Kershaw and McCulloch, 2007, and Rouse, 1984).

BACKGROUND

Shrubification has been studied in many Arctic, high-latitude, and tundra ecosystems over past decades. These ecosystems are among the most sensitive to climate warming in terms of climate-induced shifts in vegetation communities (Epstein et al., 2012, Arft et al., 1999; Myers-Smith & Hik, 2018). The impact of shrubification varies in response to the type of environmental conditions in these regions, but a major theme surrounding the effects of shrubification is permafrost thaw. For example, in some areas with continuous permafrost the active layer has become wetter with shrubification due to an increased snow cover and the impermeable frozen strata that prevent infiltration and percolation (Woo & Young, 2006). In discontinuous permafrost zones, however, some areas have become drier due to increased net evapotranspiration and increased drainage associated with shrubification and permafrost thaw (Hansen et al., 2016). How permafrost thaws in response to shrub interactions also varies. For example, Jafarov et al., (2018), have shown that snowpack under tall shrub canopies along the Arctic hillslopes can lead to a transition of the landscape with continuous permafrost to permafrost with through taliks. Their model-based study showed that the shrub-snow interaction can lead to permafrost degradation even in the absence of warming.

Shrubification alters the ground thermal regime in numerous ways. In the winter, snow is trapped by shrub branches, forming an insulative layer and subsequently resulting in warmer winter ground temperatures (Fraser et al., 2014; Lawrence & Swenson, 2011). Recent studies have
demonstrated warmer temperatures in the winter months below shrubs at sites across Alaska (Leger et al., 2019). One modelling study in the southern Seward Peninsula, Alaska, where taller shrubs dominate the midslope landscape, demonstrated how the shrub-snow interaction can lead to warmer soil temperatures below shrub canopies (Jafarov et al., 2018). The density of shrubs influences their ability to intercept snow, which in turn influences the soil temperature. Sparse shrubs do not intercept snow significantly, and shorter shrubs intercept less snow than taller shrubs (Myers-Smith et al., 2011; Loranty et al., 2018; Kropp et al., 2020). In contrast, shrub encroachment can cool soils during summer and may lead to a reduced active layer depth because of the increased canopy shading from solar radiation (Lewkowicz, et al., 2017; Hallinger, M., Manthey & Wilmking, 2010; Lawrence & Swenson, 2011). For instance, in the Indigirka lowlands, the active layer thickness was greater under shorter shrubs (Blok et al., 2010). To date, few studies have addressed the effects of shrubs and shrubification on summer soil temperatures (Blok et al., 2010; Myers-Smith et al., 2011; Loranty et al., 2018).

When other factors are not limiting, warmer conditions are likely to indirectly promote shrub growth by enhancing soil microbial decomposition that promotes nutrient mineralization for shrub uptake (Myers-Smith et al., 2011). As shrubs proliferate, they can cause a positive feedback loop to their own growth by creating a deeper snowpack, which would keep the ground warmer in winter. While shrubs can increase evapotranspiration and contribute to drier conditions, they profit from warmer, moist conditions, which favor microbial decomposition of organic matter, as shrub growth is limited by nitrogen availability (Shaver and Chapin 1980; Myers-Smith et al., 2011). Therefore, shrubification may favor the development of different types of environmental change depending on permafrost distribution and may either accelerate or decelerate as a result (Chapin & Shaver, 1985).

Changes in the thermal and hydrological regimes of the active layer and litter chemistry associated with shrubification may also impact carbon and GHG cycling. For example, drying and deepening of the active layer may increase decomposition and overall C mineralization rates of residual soil organic matter. However, more recalcitrant plant tissues and litters may also reduce overall rates of microbial mineralization by reducing the activity of microbial consortia involved in methane production (Moore & Basiliko, 2006; Godin et al., 2012). This research examined the extent of shrubification and its impact on ground temperatures and microbial carbon dioxide and methane production in fen environments of the Hudson Bay Lowlands, near Churchill, MB.

**STUDY AREA**

The Hudson Bay Lowlands (HBL) form the largest peatland complex in Canada and the second largest cold region peatland in the world, representing approximately 30 Pg of stored biospheric carbon (Packalen et al., 2014). After the collapse of the Laurentian ice sheet, the HBL were depressed by up to 270 m compared to their current levels and were flooded by the Tyrell Sea (Lajeunesse and Allard, 2003). Glacial isostatic adjustment led to emergence of the Lowlands from the sea and continues at very rapid rates up to 13 mm/yr (Packalen et al., 2014). The continued emergence of new land above sea level creates a uniquely dynamic environment for the aggradation of permafrost.

The Churchill area lies at the boundary of boreal forest and southern arctic ecozones, on the Hudson Bay shoreline. From 1981-2010, the mean annual temperature was -7°C and the mean temperature of the warmest and coldest months were 13°C in July and -26°C in January (www.climate.weatheroffice.ec.gc.ca). At our study site (Figure 1), two different vegetation communities were investigated: one dominated by sedges, dominant in the fen, and one dominated
by shrubs, encroaching into the fen. The site is located about 1 km east of the Churchill Northern Studies Centre to the north of Palsa Road (also known as Ramsey Lake road). Detailed site descriptions of sedge meadow areas located in the immediate vicinity have been previously published in Edwards et al., (2006) and Edwards and Jefferies (2010).

METHODS

Assessing greening trends near Churchill, Manitoba: Greening trends during 1984-2018 were analyzed using Google Earth Engine. We used data from Landsat 5 (Multispectral Scanner and Thematic Mapper), Landsat 7 (Enhanced Thematic Mapper), and Landsat 8 (Operational Land Imager and Thermal Infrared Sensor) in our analysis. Only Tier 1 scenes representing top-of-atmosphere reflectance were used. Normalized difference vegetation index (NDVI) values derived from Landsat 5 and 8 scenes which were calibrated to Landsat 7 using a cross-sensor technique to accommodate for spectral differences among sensors (Ju and Masek, 2016; Pironkova et al., 2018). Maximum NDVI during each summer was identified for each pixel and compiled in an image stack for Mann-Kendall trend analysis using the Kendall package in R (Windows version 3.5.2; Team, 2013). Pixels where significant greening has occurred were identified as having positive Kendall tau values and two-sided p values less than 0.05. A Mann-Kendall trend analysis was performed in R, and pixels where the trend was not significant (p > 0.05) were removed from the image.

Ground Temperature Data: Near-surface ground temperatures were monitored and recorded every two hours from October 2018 to October 2019 by Onset HOBO MX2201 Pendant data loggers (range of internal temperature sensor: -20°C to 50°C in wet conditions, accuracy: ±0.5°C from -20°C to 70°C, resolution: ±0.04°C) buried 4 cm beneath the ground surface in sedges and shrubs at the study site.

Measuring microbial GHG production: In vitro microbial GHG production, also referred to as “production potentials” (e.g., Moore & Basiliko, 2006) was measured using jar incubations of soil from both vegetation communities at room temperature (21°C) in anaerobic and aerobic conditions. Near surface soil samples (upper 10 cm) were collected from sedge (9 samples) and shrub dominated environments (8 samples) in August 2019. 20-25 g of soil was placed in 250 mL glass Bernardin® canning jars, accompanied by 40 mL of water for aerobic conditions (to ensure a slurry that would facilitate oxygen movement with turbulent mixing) and 25 mL of water for anaerobic conditions. The anaerobic jars were degassed to remove oxygen, which was replaced with pure N₂ gas (degassed and flushed 5 times). The jars were incubated at room temperature over a period of 3 days for aerobic conditions and 4 weeks for anaerobic conditions. Aerobic jars were shaken on an orbital shaker during the incubation period.

Gasses were sampled by extracting 10 mL of headspace with a syringe every day of the incubation period for aerobic conditions and on day 7, 14 and 28 for the anaerobic conditions. The samples were analyzed for CO₂ and CH₄ through gas chromatography (GC; Greenhouse Gas model, SRI Instruments, Torrance, CA, USA) using a Porapak-Q column (80/100 mesh) maintained at 65°C to separate gases and a flame ionization detector with in-line methanizer (reducing CO₂ to CH₄ using a Ni-catalyst; Godin et al., 2012). A standard gas containing 1000 ppm CO₂ and 10 ppm CH₄ in an air balance (Praxair Inc., Sudbury, ON, Canada), was run before each of the 17 samples for calibration and subsequent calculations of CO₂ and CH₄ concentration (Godin et al., 2012).
RESULTS AND DISCUSSION

Site conditions: The sedge site was dominated by Carex aquatilis, with occasional incidences of Carex chordorrhiza and Carex gynocrates. Other vascular plants present include Andromeda polifolia, Equisetum variegatum, Salix arctophila, and Scirpus caespitosus. Little to no moss was present. There was standing water for most of the summer, which may or may not dry out in the late summer. The shrub site is located between a large pond and the sedge site. It consisted of a large copse of Betula glandulosa surrounded by smaller shrub outposts 1-2 m in diameter. Other low-lying vegetation includes Vaccinium uliginosum, Salix planifolia and high frequencies of moss species such as Dicranum elongatum and Hylocomium splendens. Various sedge and grass species, including Carex aquatilis, and Equisetum variegatum occur at low incidences. The site is typically wet during the spring and most of the summer, and gradually dries out towards mid-late August.

Most shrubification studies to date have been conducted in the Low Arctic (e.g., Hill & Henry, 2011) or in northern alpine systems (e.g., van Wijk et al., 2003). Many experiments have taken place in drier environments such as well-drained, non-forested sites in Northern Quebec (e.g., Ropars & Boudreau, 2012) and coastal lowlands in Ellesmere Island, Nunavut (e.g., Hudson & Henry, 2009). Shrubification in peatlands has also been widely studied in the wetlands of Northern Alaska (Zhang et al., 2013; Euskirchen et al., 2016), though overall few shrubification studies have been conducted in fen sites (Myers-Smith et al., 2011). The study presented herein took place in a fen, offering a discussion of shrubification impacts in a relatively understudied setting for this topic. The study site also presents a unique geomorphological setting, as it is only 4 km from the coast and is affected by the interplay of rapid isostatic uplift, post-emergence permafrost aggradation and anthropogenic climate warming.

Greening trends near Churchill, MB: Significant and widespread greening has taken place in the Hudson Bay Lowlands between 1985 and 2018 (Figure 2). In Northeastern Manitoba this trend is most pronounced towards the coast, including in the areas near the Churchill Northern Study Centre and the study site. It is difficult to disentangle the component of this greening that is associated with vegetation succession in the context of isostatic rebound (at a rate of ~10mm/year (Sella et al., 2007)) from the component that is attributable to warming-induced vegetation changes. Regardless, we expect that large proportional increases in greening reflect shrub vegetation proliferation and ongoing research is evaluating differences in greening among varying land cover classes. In the context of this initial study, both the shrubs and sedges sites are situated within pixels that have experienced relatively moderate greening (tau = 0.29 to 0.45, two-sided p < 0.01) since 1985.

Ground surface temperatures: Mean annual ground surface temperature at the shrub site was 2.6°C while it was -1.0°C at the sedge site (Table 1), with approximately twice as many thawing-degree-days (1452) at the shrubs site compared to the sedge site (715). Clearly, the effects of shading by the shrub foliage during summer was minimal compared to the cooling effects of the wet soil conditions and ponding at the sedge site. Myers-Smith et al. (2011) noted that Betula provides limited shade compared to Salix and Alnus species, but previous work still reports a neutral or cooling effect of shrubification on summer ground temperatures. Some canopy removal experiments have shown significant soil warming and permafrost thaw (Loranty et al., 2018). For instance, in grassy tundra of the Indigirka lowlands in northeast Siberia, the removal of Betula nana increased active layer thickness by 9% (Blok et al., 2010). At a wet sedge tundra site in Alexandra fjord, NU, Hill and Henry (2011) observed relatively stable summer soil temperatures while winter soil temperatures increased at a site affected by increasing above-ground biomass.

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These previous results contrast with the present study where near surface temperatures were substantially warmer under *Betula* than *Carex* during summer. However, the temperatures recorded reflect conditions at the study area, where a slightly elevated ground surface beneath *Betula* was dry while ponding was extensive beneath *Carex*, favoring evaporative cooling at the latter. The discrepancy between this study and previous observations of shrubification impacts on summer ground temperatures confirms that ground thermal responses to shrubification vary depending on regional setting and highlights the importance of considering a range of environments affected by shrubification to improve understanding of shrubification impacts on ground temperatures.

Near surface temperatures were similar at the two sites during fall but diverged in late December (Figure 3), likely when sufficient snow had accumulated in the shrubs to provide insulation while a shallower windblown snow pack in sedges would allow heat to escape more rapidly. There were twice as many freezing degree days at the sedge site (1094) compared to the shrub site (515) (Table 1). The warm ground surface conditions observed in the *Betula* during winter are consistent with a thicker snow pack (Sturm & Benson 1997; Liston *et al.*, 2002), and the high thermal conductivity of ice and frozen wet ground may have favored efficient cooling beneath *Carex* (Burn, 2004).

Based on the limited temperature data presented herein, it is unlikely that permafrost was sustained beneath shrubs at the study area while it was very likely sustained in the fen. If shrubification at the sedge sites and development of areas with higher, better drained soil supporting shrub patches was simply a part of post-emergence vegetation succession, we would expect to see permafrost aggradation at the drier shrub site. Yet what we observed here was the opposite. These results are preliminary, as only limited temperature data were collected, and the

Figure 2. Mann Kendall tau values for a trend analysis in NDVI between 1984 - 2018. Only pixels with a significant trend are included (p < 0.05).
ground thermal regime at these sites will be examined in more detail in coming years.

Table 1. General temperature descriptions comparing the average, maximum and minimum temperatures from October 2018 to September 2019 (1 year) between the sedge and shrub site. The freezing degree days represent the sum of the average daily degrees below freezing (0°C) for the 1-year period. The thawing degree days represent the sum of the average daily degrees above freezing for the 1-year period.

<table>
<thead>
<tr>
<th>Site</th>
<th>Mean daily Temperature (°C)</th>
<th>Max. daily Temperature (°C)</th>
<th>Min. daily Temperature (°C)</th>
<th>Freezing Degree Days</th>
<th>Thawing Degree Days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sedge</td>
<td>-1.0</td>
<td>9.8</td>
<td>-12.8</td>
<td>1094</td>
<td>715</td>
</tr>
<tr>
<td>Shrub</td>
<td>2.6</td>
<td>19.6</td>
<td>-5.1</td>
<td>515</td>
<td>1452</td>
</tr>
</tbody>
</table>

Figure 3. Graph showing the average daily ground temperatures between October 2018 to September 2019 for the shrub and the sedge sites.

Soil microbial CO₂ and CH₄ production: Although CH₄ production was higher (p < 0.01) in the wetter sedge sites, as expected (Myers-Smith et al., 2011; Lawrence & Swenson, 2011), there were no significant differences in CO₂ production under either oxic or anoxic conditions between soils from shrub and sedge sites (Figure 4; p= 0.72). Sedges are almost always found in wetter (moist) environments, while sites with dominant shrubs may show increased evapotranspiration, thereby drying soils and presenting more aerobic conditions. Myers-Smith et al., (2011) suggested that increased evapotranspiration from greater shrub biomass could dry soils leading to higher microbial production of CO₂, which is contrary to the results obtained in this study. Potential explanations include poorer substrate quality in the shrub-dominated sites (Moore and Basiliko 2006) and/or increased acidity constraining microbial enzyme activities (Williams et al., 2000).
CONCLUSIONS

Shrub proliferation has been a widespread occurrence in tundra ecosystems over the past decades, yet few studies have addressed the extent and impacts of shrubification in subarctic fen environments. This study addressed this knowledge gap by assessing shrubification rates and associated changes to ground temperatures and microbial greenhouse gas production in a Hudson Bay Lowlands fen near Churchill, MB. Key results from this preliminary study indicate that (1) widespread greening has taken place in the Hudson Bay Lowlands between 1985 and 2018,
including at the study site near the Churchill northern Studies Centre; (2) shrub proliferation at the study site was associated with increased winter and summer ground temperatures, making it likely that permafrost can only be sustained under the parts of the fen where Carex still dominates; (3) organic soils from shrub dominated sites had a significantly lower microbial methane production than soils from the sedge dominated fen even after 1 month anaerobic incubation at room temperatures, but there were no differences in microbial carbon dioxide production. These preliminary results suggest it is unlikely that permafrost is sustainable where shrubs encroach, yet the effects of permafrost thaw on carbon cycling could be in part offset by lower microbial methane production associated with shrubs in near surface soils.

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Modelled Soil Temperature Sensitivity to Variable Snow and Vegetation Conditions in Low-Relief Coastal Mountains, Nunatsiavut and NunatuKavut, Labrador

Rosamond Tutton¹; Robert G. Way, Ph.D.²; Ryley Beddoe, Ph.D.³; Yu Zhang, Ph.D.⁴; and Andrew Trant, Ph.D.⁵

¹Northern Environmental Geoscience Laboratory, Dept. of Geography and Planning, Queen’s Univ., Kingston, ON (corresponding author). ORCID: https://orcid.org/0000-0002-4843-0451. E-mail: 18rjt4@queensu.ca; rosytutton@gmail.com
²Northern Environmental Geoscience Laboratory, Dept. of Geography and Planning, Queen’s Univ. E-mail: robert.way@queensu.ca
³Dept. of Civil Engineering, Royal Military College of Canada. E-mail: ryley.beddoe@rmc.ca
⁴Canada Centre for Remote Sensing, Canada Centre for Mapping and Earth Observation, Natural Resources Canada. E-mail: yu.zhang@canada.ca
⁵Ecological Legacy Lab, School of Environment, Resources and Sustainability, Univ. Waterloo. E-mail: atrant@uwaterloo.ca

ABSTRACT

Understanding permafrost vulnerability and resilience to climate warming is critical for predicting impacts on northern communities and ecosystems. The thermal characteristics of near-surface permafrost are influenced by effects from overlying vegetation and snow cover, both of which are changing in northern environments. The association between vegetation and snow is important in the coastal mountains of Labrador, northeast Canada, because of high annual snowfall totals and greening tundra biomes. In this study, we present a series of one-dimensional simulations using the Northern Ecosystem Soil Temperature (NEST) model to characterize ground thermal conditions at two field sites (Nain, Nunatsiavut and Pinware, NunatuKavut) along the Labrador coast. NEST simulations covering 1979–2019 were run using ERA5 atmospheric reanalysis for three ecotypes (tundra, shrub, treed) with three different snow accumulation regimes (snow drifting away from site, no snow drift, snow drifting to site). At Nain, perennially frozen ground was present for all three ecotypes when snow cover was kept thin (drifting away) but was largely absent for the ecotypes when snow accumulation was higher. At Pinware, frozen ground was mostly absent except where snow cover was shallow (wind drifting away). For low-snow simulations, frozen bodies (< 20 m) persisted in all ecotypes during cold periods but only remained intact following warmer years for treed ecotypes. These results highlight the importance of spatial and temporal variability in snow cover on ground thermal regimes in coastal Labrador.

INTRODUCTION

Widespread permafrost thaw is projected to occur by the end of this century throughout much of the discontinuous permafrost in Canada (Derksen et al. 2012). Changes to regional permafrost distribution will impact traditional activities (Anderson et al. 2018) and habitat suitability for keystone species (Berteaux et al. 2016). Concurrently, rapid environmental change is expected to alter the distribution of northern vegetation with implications for permafrost and snow cover (Sturm et al. 2001). Sturm et al (2005) found that increased shrub growth led to the accumulation of a thicker, less dense snowpack which better insulates the soil and favours further shrub growth. Variability in snow distribution due to wind redistribution, topography and vegetation interception has also been shown to be a critical factor contributing to permafrost thaw (Wilcox et al. 2019).
Despite its importance to local ecosystems and people, interactions between terrestrial cryosphere components and vegetation are understudied in coastal mountains of the eastern Canadian Subarctic (Way et al. 2018). A series of recent studies (e.g. Way and Lewkowicz 2016, 2018) have highlighted the importance of late-winter snow thickness for modelling permafrost distribution and vulnerability to thaw in coastal Labrador. An enhanced understanding of permafrost-snow-vegetation interactions will support evidence-based infrastructure and development planning in the region.

This study considers two research basins established by an ArcticNet supported initiative in coastal Labrador. Due to the general lack of permafrost information or long-term ground temperature records near these basins, we use ground thermal modelling to provide a first estimate of ground freezing characteristics and to explore variability in ground thermal conditions expected in the region. In this study, we use the Northern Ecosystem Soil Temperature (NEST) model (Zhang et al. 2003) to simulate ground temperatures across ecotypes and snow cover scenarios to evaluate ground thermal dynamics at both snow-permafrost-vegetation research basins.

STUDY AREA

The two study sites (Pinware River Hills [PRH] and Nain Bay Hills [NBH]) were established in 2019 following discussion with the NunatuKavut Community Council and Nunatsiavut Government. Research basins are situated in forest-tundra transitions within the coastal barren ecozones of Nunatsiavut and NunatuKavut (Roberts et al. 2006), at southern (PRH) and northern (NBH) ends of the discontinuous permafrost zone (Way and Lewkowicz 2016) (Figure 1 B). Regional permafrost modelling suggests that NBH is in the sporadic discontinuous permafrost zone while permafrost near PRH is restricted to isolated patches (Way and Lewkowicz 2016).

The regional climate at both sites is influenced by their coastal proximity with cooler, wetter summers and milder winters than areas farther inland (Way et al. 2017). The mean annual air temperatures (MAAT) (1979-2019) are approximately 0.1 °C at PRH (51.7°N, -56.6°E, ~214 m a.s.l.) and -3.0 °C NBH (56.6°N, -62.0°E, ~125 m a.s.l.). The seasonal temperature range at PRH is smaller than at NBH due to colder winters in the Nain area caused by seasonal sea ice cover (winter-spring) in the region (Way et al. 2017). Both study areas are situated on hilltop plateaus in low-relief coastal mountains positioned above the marine limit. Preliminary site evaluations included investigations of near-surface soil freezing (summer 2019) and snow cover (winter 2020). Soil profiles and subsurface conditions were inferred from regional geomorphology, site photos and field studies conducted in coastal Labrador (Majorowicz and Minea 2015; Mmanus et al. 2012; Way et al. 2018). Depth to bedrock varied but average conditions were inferred from soil probing (maximum of 120 cm) and site photos.

PRH is located between the NunatuKavut Community Council’s communities of Red Bay and Pinware along the Trans-Labrador Highway (Route 510). Black spruce (Picea mariana) and white spruce (Picea glauca) krummholz are found in wind sheltered, moist sites while Sphagnum mosses, sedges, cotton grasses (Eriophorum spp.), and other wetland species occupy poorly drained depressions and bogs on lower slopes (Roberts et al. 2006) (Figure 1 D). Bedrock in the region is composed of old and deformed granites (Majorowicz and Minea 2015). NBH is approximately 22 km from the Nunatsiavut community of Nain. Vegetation consists mostly of dense white spruce krummholz, willows (Salix spp.) and other shrubs (Figure 1 C). Elevated headlands have alpine tundra and dwarf and prostrate shrubs, forbs, sedges, grasses and mosses (Roberts et al. 2006, Larking et al. 2021). Bedrock in the region has been mapped as predominantly gabbro (Geological Survey 2015).
METHODS

**Numerical Modelling:** Ground temperatures were simulated for each field site using the NEST model (Zhang et al. 2003). NEST is a one-dimensional process-based model that integrates the effects of climate, vegetation, snow, soil composition and moisture on ground thermal conditions based on energy and water dynamics. NEST determines upper boundary conditions (ground/snow surface) using a surface energy balance while lower boundary conditions are controlled by the geothermal heat flux at 120 m depth. The thickness of snowpack is calculated based on snow water equivalent (SWE) and snow density. SWE is determined by the difference between snowfall and snowmelt, while snow density profiles consider compaction and metamorphism of the snowpack. The model uses an input parameter to consider the effects of snowdrift by modifying the amount of snowfall for the site (Zhang et al. 2012). The soil water dynamics are simulated from precipitation and snow melt, evaporation and transpiration, and water movement. Further information regarding the implementation of NEST model can be found in Zhang et al. (2003, 2012) with recent model updates available via the NEST user manual.

**Site Input:** A simplified soil profile was created for both sites composed of a surficial organic layer (0-0.3 m), silt (0.3-0.5 m), loam (0.5-1 m) and sandy loam (1-2 m), which is broadly consistent with local land cover and soil probing at PRH and NBH non-bedrock locations and with similar sites in the region (Way et al. 2018). The percentage of organic matter decreases from 99% to 20% at 0.2 m with an increasing degree of decomposition (fibric to hemic). Bedrock is presumed...
2 m below the ground surface based on local site visits and soil probing. All model runs assumed a flat slope with no topographic shading and no surface water inflow. Ground water inflow and outflow were set at minimum depths of 20 cm and 10 cm respectively. This parameterization is generally representative of the upland plateau tundra-shrub-tree transitions at the field sites but less so for the small wetlands present in portions of the sites. Geothermal heat flux was set as 0.029 W/m² and 0.054 W/m² for NBH and PRH, respectively, based on Majorowicz and Minea (2015) and previous modelling in the region (Way et al. 2018; Wang 2020). Fraction of quartz in soil was set as 0.1 and 0.0 for PRH and NRH, respectively, following estimates by Way et al. (2018).

**Climate Data:** Input climate data (daily temperature [minimum, mean and maximum] [°C], precipitation [mm/day], total horizontal solar radiation [MJ/m²/day]), daily mean wind speed [m/s] and daily total downward longwave radiation [MJ/m²/day]) required for modelling was derived from ERA5 atmospheric reanalysis (Hersbach et al. 2020) for the period of 1979-01-01 to 2020-01-01. Snow fraction was assumed to be 1 when the daily mean air temperature was less than 0°C. Vapour pressure was calculated based on daily minimum air temperature (Tₘ) using the August-Roche-Magnus equation (Alduchov and Eskridge 1996):

\[
V_{sr_m} = 6.11e^{\left[\frac{17.27T_{m}}{T_{m}+237.3}\right]},
\]

where \(V_{sr_m}\) is the saturated water vapour pressure (mbar) at \(T_{m}\) (daily min air temperature (°C)).

Although ERA5 data has a coarse spatial resolution of 0.5°, the mean elevation for the grid cell overlapping with our sites (Figure 1 B) is similar to that of our field sites (mean absolute error [MAE] of 23 m and 83 m for PRH and NBH, respectively). Although prior generations of some atmospheric reanalysis products showed biases in the eastern Subarctic region (Rapaić et al. 2015), ERA5 trends were similar to studies conducted on the broader region (Barrette et al. 2020). ERA5 air temperature data were verified against shorter, discontinuous records collected from nearby Country Cat Pond and Nain Airport. This comparison between ERA5 and station data showed a daily MAE of 2.09 ± 1.89°C at PRH and 1.39 ± 1.19°C at NBH. The MAAT bias of the climate station data was 0.41 °C (PRH) and -0.32 °C (NBH) relative to the ERA5 reanalysis data. According to ERA5, regional air temperatures have warmed over the past 40 years (0.32 °C/decade [PRH], 0.44 °C/decade [NBH]) but with significant interannual variability (standard deviation of 1.05 °C [PRH] and 1.30 °C [NBH]). Extreme years were observed at both sites in 1992 and 2010 with the former being 1.9 (2.5) °C below the long-term normal at PRH (NBH) and the latter being 2.7 (4.0) °C above the long-term normal at PRH (NBH). Regionally, there are no statistically significant trends in precipitation over the past 40 years in agreement with Rapaić et al (2015).

**Modelling Scenarios:** Nine different modelling scenarios were generated for each site to simulate combinations of three different ecotypes (tundra, shrub and treed) and three different snow redistribution schemes (snow drifting away from site, no snow drifting and snow drifting to site). Vegetation height was set at 0 m for tundra, 0.3 m for shrub and 1.5 m for treed while summer leaf area index (LAI) was set at 0.2 (tundra), 0.5 (shrub) and 3 (treed) (Abuelgasim 2011). Summer LAI and surface albedo were contextualized from region specific papers on LAI across vegetation types in the eastern Subarctic (Abuelgasim 2011; Mmanus et al. 2012). Snow redistribution schemes at individual sites were set by changing NEST’s input snow drift factor to 0.8, 0.0 and -0.8 for scenarios with snow drifting away from site, no snow drifting and snow drifting to site, respectively. Simulated late-winter snow thicknesses were broadly consistent with field observations during snow surveys in winter 2020.

**Validation:** Mean annual ground surface temperatures (MAGST) were collected from tundra, low shrub, high shrub and krummholz sites at the PRH research basin from August 2019 to
September 2020 (Figure 2). Due to Covid-19 restrictions, data could not be retrieved from the NBH basin. Comparison between field observations and NEST simulations extended to Fall 2020 showed MAEs of 1.60 °C (snow drift away) at a tundra site, 1.78 °C (snow drift away) at a low shrub site, 2.08 °C (no snow drift) at a high shrub site, and 1.60 °C (snow drift to site) at a treed site.

**Analysis:** Model outputs were analyzed for permafrost related parameters including cryotic ground thickness, seasonal freeze-thaw layer depth and the mean annual ground temperature (MAGT) at the base of the freeze-thaw layer (FTL). We use the term cryotic ground as opposed to permafrost to reflect the possibility ground below 0°C for at least one full year, but not necessarily the two years required to fall under the definition of permafrost (Harris et al. 1988). Thickness of cryotic ground was determined as the difference between annual maximum freeze and maximum thaw depths while the FTL was determined as the maximum thaw depth if cryotic ground is present, or the maximum freeze depth where it was not. MAGT at the base of the FTL is equivalent to the temperature at the top of permafrost (TTOP) (Smith and Riseborough 2002; Way and Lewkowicz 2018) and is used instead of MAGT at the depth of zero annual amplitude to better reflect climate impacts on the thermal state of thinner permafrost bodies.

![Figure 2: Modelled ground surface temperatures (GST) for the snow drifting away from site (orange), no snow drifting (blue) and snow drifting to site (pink) scenarios compared to field ground surface temperature data (dotted line) for A) the tundra site, B) the shrub sites, where the bolded dotted line is field data from the high shrub site and the regular dotted line is field data from the low shrub site, and C) treed site.](image)

**Results:** A total of 18 simulations were run using NEST spanning 1979 to 2018 (inclusive) with daily ground temperature outputs produced for the upper 10.55 m and maximum cryotic ground thicknesses determined from the whole profile (120 m). Model runs showed considerable variability in snow thicknesses because of the local snow drifting factors used with mean annual average snow depth of 2 cm (5 cm) for drift away from site at PRH (NBH), 59 cm (64 cm) for snow drift to site, and 27 cm (33 cm) for no snow drifting.

At PRH, maximum cryotic ground thickness ranged from 0 m (continuously for most ecotypes with drifting to site & no snow drifting) to 19 m (1999 for the treed ecotype with snow drifting...
away from the site) (Figure 3 A i.-iii.). No cryotic ground was modelled for the snow drifting to site and no snow drifting scenarios (Figure 3 A iv.-ix.). At PRH, mean FTL depth was $1.7 \pm 1.6$ m for snow drifting away from the site, $0.3 \pm 0.2$ m for snow drifting to site and $0.4 \pm 0.2$ m for no snow drifting (Figure 3 A i.-ix.). At NBH, cryotic ground thickness exceeded the bottom of the profile (>120 m) for all years in the snow drifting away from site scenario (Figure 3 B i.-iii.). Maximum continuously frozen ground thickness for the no snow drift scenario ranged from ~15 m (1998, tundra) to ~17 m (1999, treed) (Figure 3 B iv.-vi.) but was rarely present with snow drift to site, though frozen soil layers 0.5 m thick briefly developed in 1993 (Figure 3 B vii.-ix.). The mean FTL thickness at NBH was $0.78 \pm 0.15$ m for snow drifting away from the site, $0.84 \pm 0.38$ m for no snow drift and $0.41 \pm 0.25$ m for snow drifting to site (Figure 3 B i.-ix.).

![Figure 3: Cryotic ground (blue), unfrozen ground (blank) and freeze-thaw layers (orange) for 1979-2018 for Pinware River Hills (PRH) (A i.-ix.) and Nain Bay Hills (NBH) (B i.-ix.)](image)

The average MAGT at TTOP across all scenarios was $1.7 \pm 2.0$ °C at PRH and $-0.5 \pm 2.7$°C at NBH. MAGT for PRH ranged from -2.3°C (1993, shrub, snow drifting away from site) to 4.6°C (2006, tundra, no snow drifting). Only the snow drifting away scenario resulted in MAGTs favorable to permafrost at this site (Figure 4). At NBH, MAGT ranged from -6.5°C (1993, shrub, snow drifting away from site) to 2.9°C (2006, tundra, snow drifting to site). The standard deviation between ecotypes was 0.6°C, 0.8°C, and 0.7°C for PRH and 1.1°C, 0.9°C, and 0.6°C for NBH (snow drift away from site, no snow drift, and snow drift to site respectively). MAGT was more variable when comparing between snow redistributions, with standard deviations ranging from 1.8°C (treed) to 2.1°C (shrub) at PRH and 2.4°C (treed) to 2.9°C (tundra) at NBH.

Due to the similar magnitudes and temporal trends in MAGT across ecotypes for the same snow redistribution scenarios, we aggregated MAGT across ecotypes for each snow drift factor and calculated the rate of MAGT change through time at both sites (Figure 4). Both sites and most scenarios exhibited high variability in MAGT (standard deviation of 2.0°C [PRH] and 2.7°C...
(NBH), but overall warming over the past 40 years. Significant warming (> 90% confidence level) was only evident at PRH for the snow drift to the site scenario \( (R^2 = 0.14, p\text{-value} = 0.02) \) (Figure 4 A) and for NBH under the no snow drift \( (R^2 = 0.08, p\text{-value} = 0.08) \) and snow drift to the site \( (R^2 = 0.24, p\text{-value} < 0.01) \) (Figure 4 B) scenarios. The rates of warming were 0.020°C/year, 0.021°C/year and 0.024°C/year for PRH snow drift to site, NBH no snow drift and snow drift to site, respectively (Figure 4).

![Figure 4: Mean annual ground temperature at the top of permafrost/base of the freeze thaw layer aggregated for the three ecotypes by snow drift factor for A) Pinware River Hills (PRH) and B) Nain Bay Hills (NBH). Least-squares trendlines are organized by snow drift factor and errors are depicted in grey shading.](image)

DISCUSSION

Implications for Permafrost in Coastal Labrador: During preliminary field investigations at Pinware River Hills (PRH), late-lying frozen ground was detected to depths of 59 cm and 61 cm at several locations (typically exposed tundra and peat covered mounds) but was generally absent from the upper 120 cm. At Nain Bay Hills (NBH), frozen ground was detected via probing regularly at depths of 50 cm to 92 cm although more widespread near-surface bedrock made probing difficult to interpret where deeper thaw had occurred. Cryotic ground was simulated for both PRH and NBH with minimal snow cover; however, simulations suggested contemporary near surface cryotic ground would be non-existent at PRH and rarely present at NBH with higher snow accumulation. MAGTs and cryotic ground thicknesses did not vary greatly across ecotypes at PRH nor NBH but during extreme warm years greater thaw was inferred for tundra and low shrub ecotypes compared to treed ecotypes at both sites. Cooler air temperatures in the early 1990s also led to deeper freezing at NBH for the treed ecotype relative to the tundra and shrub ecotypes. These results align with those of Jorgenson et al. (2010) showing that forest cover can elicit strong negative feedbacks which enhance permafrost resilience to climate warming and disturbance.

The presence and persistence of cryotic ground at the southern site (PRH) is more dependent on thin snow cover than MAAT, similar to results from peatland permafrost in southeast Labrador (Way et al. 2018; Way and Lewkowicz 2018). The correlation between annual thawing degree days (TDDA) and MAGT at TTOP varied with snow drift scenarios, showing a strong association for the snow drift to site \( (r = 0.75) \) but a weak association \( (r = 0.27) \) for the snow drifting away
scenario. The magnitude of change across sites linked to snow redistribution suggests that changes in vegetation and associated snow distribution may be more impactful than atmospheric climate warming on MAGT in this region. Zhang et al. (2008) found that a thinning snowpack and shorter snow duration (as a product of climate warming) is anticipated to reduce the rate of permafrost degradation due to climate warming. This decoupling of ground and air temperature could impact permafrost thaw rates in the eastern Subarctic (Zhang et al. 2005, 2008).

Both PRH and NBH showed rapid freeze/thaw transitions and minimal buffering of the climate response due to coarse soils overtop of shallow bedrock (at a depth of 2 m) (Throop et al. 2012). The generalized soil profile (composed of an organic layer, silt, loam and sandy loam) resulted in a relatively high thermal conductivity and low specific heat capacity (Wang et al. 2019). The model assumption of pure, pore free bedrock likely contributed to the volatility of the MAGT. This could be adjusted by altering ground thermal conditions such as the lateral water flow; however, we chose to use observed conditions to maintain consistency and focus on the sensitivity of the sites to snow and vegetation. All modelled scenarios experienced drops in MAGT between 1989 and 1995; however, for the no snow drift scenario continuously frozen ground developed at NBH while at PRH ground remained unfrozen despite the cooler climate conditions. Together these results lead us to suggest that climate-driven, ecosystem-protected permafrost can exist at PRH and climate-driven permafrost can exist at NBH (Shur and Jorgenson 2007).

Although climate warming has occurred in coastal Labrador, trends in MAGT were rarely statistically significant due to large inter-annual fluctuation. For decades, experimental studies have demonstrated the importance of snow cover for permafrost formation and thaw processes (Nicholson 1979; O’Neill and Burn 2017), particularly in discontinuous permafrost and ecologically sensitive environments (Way et al. 2018). At PRH and NBH, scenarios with deeper snow (drift to the site) experienced significant warming of ~0.02 °C/year and MAGT varied by up to 4 °C at PRH and by 6 °C at NBH (Figure 4). These results support studies implicating snow accumulation as a driver of permafrost thaw (Grünberg et al. 2020; Jafarov et al. 2018; O’Neill and Burn 2017) and demonstrates the sensitivity of ground temperatures to snow thickness modifications in coastal Subarctic mountains.

Model limitations: Owing to a lack of detailed field data from the two research basins the model results rely on assumptions that may not universally apply to real-world conditions. For example, soil profile characteristics are kept constant across ecotypes, but this is based on geomorphological interpretation rather than soil profile analysis. Additional simulations with different soil profiles, surface covers and alternative depths to bedrock would reveal broader differences in ground thermal characteristics. As NEST is a one-dimensional model, it only considers vertical heat fluxes and thus does not consider adjacent unfrozen terrain which may affect lateral heat transfer (Wang, 2020). The model configuration used in this study also did not directly associate snow redistribution to specific ecotypes. Prior research in Labrador (Way and Lewkowicz, 2018) has shown that treed ecotypes rarely have a thin snow cover thus some simulations (e.g. snow redistribution away from site at treed ecotype) are considered a sensitivity analysis. Despite regional greening, LAI, snow redistribution factors and site characteristics were kept constant through time, which may not be representative in a changing environment (Mmanus et al. 2012). Further field investigations and model calibration will be required to assess the impacts of variable topography and land cover conditions on permafrost at these two research basins.
CONCLUSION

In this study, we examined snow-vegetation-permafrost interactions in coastal mountains of the eastern Canadian Subarctic. Both sites lie within the discontinuous permafrost zone (Riseborough and Smith 1998), a region susceptible to permafrost thaw in response to a changing climate (Barrette et al. 2020; Throop et al. 2012; Way and Lewkowicz 2016). The model results do not show significant overall warming trends in ground temperature; however, they support previous findings showing that wind scouring of sites may preserve permafrost even at the southern end of the discontinuous permafrost zone (Way et al. 2018).

Permafrost and ground freezing characteristics were not interpreted as being strongly influenced by vegetation types for either site except for the Pinware treed site with minimal snow accumulation scenario which protected permafrost from warm years while other ecotypes did not (tundra, shrub). Cryogenic at both sites responded rapidly to regional cooling and warming suggesting climate driven permafrost, particularly at NBH. However, muted response to warm years under the no snow drift treed scenario demonstrates climate driven – ecosystem protected permafrost at PRH (Shur and Jorgenson 2007). The results presented in this study demonstrate the sensitivity of cryogenic ground thickness and temperature to snow accumulation throughout coastal tundra and forest ecotones. Variability in cryotic conditions across the scenarios we present highlight the need to integrate geomorphological analysis, ecosystem science and snow science together in permafrost vulnerability assessments. This region continues to be largely unexamined and requires further fieldwork to support the modelled results presented in this study.

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Thermal Modelling of Post-Fire Permafrost Change under a Warming Coastal Subarctic Climate, Eastern Canada

Yifeng Wang¹; Antoni G. Lewkowicz, Ph.D.²; Jean E. Holloway, Ph.D.³; and Robert G. Way, Ph.D.⁴

¹Dept. of Geography, Environment and Geomatics, Univ. of Ottawa, Ottawa; Northern Environmental Geoscience Laboratory, Dept. of Geography and Planning, Queen’s Univ., Kingston, ON (corresponding author). E-mail: ywang379@uottawa.ca; yifeng.wang@queensu.ca
²Dept. of Geography, Environment and Geomatics, Univ. of Ottawa. E-mail: alewkowi@uottawa.ca
³Dept. of Geography, Environment and Geomatics, Univ. of Ottawa. E-mail: jhollowa@uottawa.ca
⁴Northern Environmental Geoscience Laboratory, Dept. of Geography and Planning, Queen’s Univ. E-mail: robert.way@queensu.ca

ABSTRACT

Forest fires are known to have lasting thermal impacts on permafrost, but there are no previous studies of such effects along the eastern Canadian coastline. One-dimensional thermal modelling was used to examine the ground thermal regime at a coastal forest fire site in the discontinuous permafrost zone near Nain (56.5°N), Nunatsiavut, eastern Canada. Simulations were undertaken for both the unburned forest and adjacent fire-disturbed area, which were modelled to have an initial permafrost thickness of 15.6 m in 1965. Future scenarios incorporated changes to regional air temperature following Representative Concentration Pathway (RCP) 4.5 and 8.5, as well as variations in surface organic material regeneration. Results varied from permafrost thinning but persisting beyond 2099 under RCP4.5 (unburned) to thawing entirely by 2060 under RCP8.5 (high severity burn, no organic material regeneration). In all burned scenarios, a supra-permafrost talik developed immediately following disturbance, but in most cases, frozen ground re-aggrated after several decades. Our findings are broadly consistent with those from western North America and demonstrate that the main impact of fire is to accelerate permafrost thaw due to climate warming.

INTRODUCTION

Forest fires are a widespread natural disturbance in many permafrost regions. The post-fire loss of organic layer and canopy cover and related changes in snow distribution have long-lasting effects on the underlying frozen ground (Jorgenson et al. 2010). Immediate and short-term increases in ground temperatures have been reported post-fire, resulting in active layer thickening and supra-permafrost talik formation (Gibson et al. 2018). However, there is a paucity of research on long-term post-fire permafrost response, particularly within the context of a warming climate.

Thermal modelling is a useful approach for examining the response of frozen ground to surface disturbance and is essential for predicting change. One-dimensional thermal modelling has been used to study post-fire permafrost response within the North American boreal forest, but almost exclusively for subarctic sites in Alaska and the Northwest Territories (e.g., Nossov et al. 2013, Brown et al. 2015, Zhang et al. 2015). Here, we employ thermal modelling to examine the impacts of forest fire on frozen ground near Nain, Nunatsiavut (Figure 1). This region is wetter than northwest North America, resulting in longer fire recurrence intervals (Foster 1983, Coops et al.
2018), faster surface organic layer growth (Williams and Flanagan 1998), and greater snow accumulation (Way and Lewkowicz 2016), all of which affect the response of permafrost to fire.

STUDY AREA

Coastal Labrador comprises the easternmost part of the Canadian Shield and is mostly characterized by igneous and metamorphic bedrock with thin layers of medium-to fine-grained materials deposited during and following the retreat of the Laurentide Ice Sheet (12-6 k years BP; Dyke 2004, Roberts et al. 2006, Bell et al. 2011). The subarctic climate is influenced by the Labrador Current which carries cold arctic waters southward, resulting in long, cold winters and short, cool summers (Banfield and Jacobs 1998). The mean annual air temperature at Nain is -2.5°C, average total rainfall is 450 mm, and average total snowfall is 475 cm (1981-2010 climate normal; Environment Canada 2020). Nain is located in the sporadic discontinuous permafrost zone (Heginbottom et al. 1995, Way and Lewkowicz 2016), but permafrost has only recently been observed at nearby forested sites where it is thought to be climate-driven, ecosystem-protected, following Shur and Jorgenson (2007).

Forests around Nain are composed of black spruce (Picea mariana), white spruce (Picea glauca), balsam fir (Abies balsamea), and eastern larch (Larix laricina; Roberts et al. 2006). The estimated fire return interval of 1501-5000 years for this region (Coops et al. 2018) greatly exceeds the estimated 75-200 year intervals for the western North American boreal forest (Foster 1983).

Our simulations relate to ground conditions at a burned site located on a north-facing slope along Webb Bay (WB; 56.706°N, 62.209°W), ~25 km northwest of Nain (Figure 1-C). The WB burn was a small (1 km²) low severity fire that occurred in summer 2004 (Brehaut and Brown 2020). The site is located on a series of raised beaches, although erratics are also present. Unburned stands are of mixed composition, with an average tree age of 227 years (Brehaut and Brown 2020).
MODELLING PARAMETERS AND METHODOLOGY

Modelling was undertaken with TEMP/W (Geoslope), a finite element geothermal modelling software. TEMP/W is commonly applied in infrastructure-related studies (e.g., Smith and Riseborough 2010), but this is the first to use it to investigate the effects of fire on frozen ground.

Profile and Material Properties: An idealised material profile was developed for WB (Table 2). Materials were assumed to be saturated (McClymont et al. 2013). The material stratigraphy and boundaries were estimated from field measurements of organic layer thickness, laboratory-based analyses of near-surface sediment samples, and interpretations of sub-surface geophysical investigations (Wang 2020), supplemented by descriptions of regional geomorphology (Natural Resources Canada 1957, Bell et al. 2011). The base of the model was set at a depth of 100 m (Brown et al. 2015), and mesh elements measured 0.01 m (surface-0.2 m), 0.05 m (0.2-3 m), 0.1 m (3-30 m), 0.5 m (30-50 m), 1 m (50-70 m), and 1.5 m (70-100 m). The TEMP/W default unfrozen moisture content (UMC) curve for sand was applied throughout the profile.

Table 1. Stratigraphy and material properties for WB.

<table>
<thead>
<tr>
<th>Material Layer (depth)</th>
<th>Dry Bulk Density (g/mL)</th>
<th>Porosity (%)</th>
<th>Volumetric Content (%)</th>
<th>Thermal Conductivity (W/m/K)c</th>
<th>Heat Capacity (MJ/m³/K)d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Organic (0-0.2)</td>
<td>0.17</td>
<td>0.84</td>
<td>0.84</td>
<td>0.16</td>
<td>1.57</td>
</tr>
<tr>
<td>Organic-Minerala (0.2-0.6)</td>
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<td>0.55</td>
<td>0.55</td>
<td>0.04</td>
<td>2.27</td>
</tr>
<tr>
<td>Upper Minerala (0.6-3)</td>
<td>1.58</td>
<td>0.39</td>
<td>0.39</td>
<td>0.02</td>
<td>2.51</td>
</tr>
<tr>
<td>Lower Minerala (3-10)</td>
<td>1.58</td>
<td>0.39</td>
<td>0.39</td>
<td>-</td>
<td>2.63</td>
</tr>
<tr>
<td>Bedrockb (10-100)</td>
<td>2.98</td>
<td>0.06</td>
<td>0.06</td>
<td>-</td>
<td>2.48</td>
</tr>
</tbody>
</table>

a mineral constituents were 15% silt, 85% sand (Wang 2020).
b bedrock was assumed to be gabbro (Natural Resources Canada 2017).
c thermal conductivities were calculated using a geometric mean (Farouki 1981).
d heat capacities were calculated using a weighted average (Johnston 1981).

Boundary Conditions: Daily air temperatures at WB were aggregated from bihourly measurements recorded from June 1, 2017 to May 31, 2018 using a shielded Hobo Pro v2 U23-001 Data Logger (Onset; ±0.45°C accuracy) positioned in the unburned forest. This mean daily air temperature record was used to linearly scale historical daily air temperatures (1950-2017; R²=0.97, n=315), derived from the Berkeley Earth Surface Temperature Project (BESTP) record for Nain (accessed through climexp.knmi.nl). The BESTP product has been shown to reliably represent air temperature variability in Labrador (Way and Viau 2015). Projected daily air temperatures (2010-2099) were averaged from four statistically downscaled climate scenarios: CanESM2, CESM1-CAM5, HadGEM2-ES, and MIROC-ESM (accessed through climate-scenarios.canada.ca; Hope et al. 2016). Projections were compiled for the current century under
moderate (RCP4.5) and high (RCP8.5) warming scenarios (Figure 2-A). Projected daily temperatures were statistically downscaled to WB using the delta method (Ramirez-Villegas and Jarvis 2010), with anomalies calculated from a 2010-2017 overlap period.

Figure 2. A) Mean annual air temperatures estimated for WB (see text for data sources); B) N-factors for burned scenarios at WB. The minimum freezing n-factor was set to 0.05.

Ground surface boundary temperatures were generated by applying n-factor transfer functions to the air temperature record. The freezing n-factor (n\textsubscript{f}) and thawing n-factor (n\textsubscript{t}) summarize the influences of snow cover and vegetation shading on ground surface temperatures (Karunaratne and Burn 2004). An n\textsubscript{f} of 0.36 and an n\textsubscript{t} of 0.73, measured in the unburned region from 2017 to 2019, were applied throughout the unburned scenarios. An n\textsubscript{f} of 0.12 and an n\textsubscript{t} of 0.84, measured in the burned region for the same period, were used to inform and calibrate post-fire changes in the burned scenarios. N\textsubscript{f} and n\textsubscript{t} were adjusted in five year increments according to a fifty-year linear two-segment model, in which values recovered to 66% of the pre-fire conditions after the first 25 years following fire, before fully returning to pre-fire conditions after another 25 years (Zhang et al. 2015; Figure 2-B). Low geothermal heat fluxes (22 mW/m\textsuperscript{2}) were measured 50 km south of the study sites (Mareschal et al. 2000). To avoid producing unrealistically thick permafrost, the geothermal heat flux was set to 220 mW/m\textsuperscript{2}, which is still below the values used to model permafrost mounds in nearby southern Labrador (Way et al. 2018).

Figure 3. Comparison of modelled (red) and measured (blue) ground surface temperatures in the unburned region at WB (July 20, 2017 to July 19, 2018).

Thermal Modelling with TEMP/W: Six basic simulations were run to evaluate the response of frozen ground under moderate (RCP4.5) and high warming (RCP8.5) with no disturbance and low and high severity fire disturbance. Six additional simulations were run to test the model sensitivity to organic layer accumulation by excluding post-fire organic material regeneration. Each time step (30 minutes) was iterated up to 900 times to allow convergence within 0.005 K. The thermal model was initialized using the 1950-1979 daily average climate normal until
equilibrium was achieved (node variability <0.01 K per 100 years; Zhang et al. 2015). The model was then run forward in time from 1965 using the daily air temperature composite for WB, comprised of BESTP (1965-2017) and RCP4.5 or RCP8.5 projections (2018-2099; Figure 2-A). Fire disturbance was modelled as a decrease in organic layer thickness and $n_f$ and an increase in $n_t$, applied as of August 1, 2004 (Figure 2-B). Low and high burn severities were represented by a modelled reduction in the organic layer by 5 or 15 cm, respectively. Following this loss, the organic layer continued to accumulate in the model at a rate of 3.6 mm/year, based on *in situ* measurements (Wang 2020). Gradual adjustments to $n_f$ and $n_t$ were applied during the post-fire recovery period (Figure 2-B).

<table>
<thead>
<tr>
<th>WB</th>
<th>RCP4.5</th>
<th>RCP8.5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>UNBURNED</strong></td>
<td><img src="fig1" alt="A" /></td>
<td><img src="fig1" alt="B" /></td>
</tr>
<tr>
<td><strong>BURNED</strong></td>
<td><img src="fig1" alt="C" /></td>
<td><img src="fig1" alt="D" /></td>
</tr>
<tr>
<td><strong>LOW SEVERITY</strong></td>
<td><img src="fig1" alt="E" /></td>
<td><img src="fig1" alt="F" /></td>
</tr>
</tbody>
</table>

**Figure 4.** Summary of modelled ground temperatures (1965-2099) for WB under RCP4.5 and RCP8.5: A-B) unburned and C-D-E-F) burned. Organic layer accumulates by 3.6 mm/year from 2004 to 2099. The base of the active layer, approximated by a temperature threshold of -0.025°C or an unfrozen moisture content of 0.11, is shown as a white line.
Model Output Processing and Validation: Model outputs were processed in R (version 3.6.2; R Core Team 2019, Wang 2020) with results portrayed as frozen (dark blue; ≤-0.025°C; <11.39% UMC), partially frozen (light blue; between -0.025°C and 0°C; 11.40-99.99% UMC), or unfrozen ground (red; >0°C; 100% UMC). Model validation used ground surface temperatures, as boreholes were not established due to permitting limitations. Ground surface temperatures were measured at four-hour intervals using Thermochron DS1922L iButtons (Maxim; ±0.5°C accuracy), buried 5 cm below the ground surface in both burned and unburned locations at WB.

RESULTS

Model Validation: Model outputs were compared to ground surface temperatures measured from July 2017 to July 2018. The modelled annual surface temperature averaged 0.41°C higher in the unburned region (daily RMSE: 2.53°C; Figure 3) and 0.13°C higher in the burned region (daily RMSE: 2.20°C; not shown). Discrepancies occurred following the transition from the historical to projected air temperature inputs in January 2018 and in the spring when observations showed a slower melt out than the modelled values due to late-lying snow.

Impacts of Fire and Climate Warming: Model simulations hindcast 15.6 m of permafrost at the WB site in 1965 (Figure 4). In the subsequent 39 years prior to the fire, the temperature at the depth of zero annual amplitude (DZAA) remains near -0.5°C (standard deviation of 0.1°C).

The RCP4.5 unburned simulation shows frozen ground persisting to the end of this century (Figure 4-A) but thinning by half relative to 1965 (Table 2), mainly from the base up. A talik does not develop and there are no notable changes in active layer thickness. In this scenario, the temperature at DZAA increases at a linear rate of ~0.004°C/year from 2018 to 2099. Under RCP8.5, a climate-initiated talik develops in the 2060s (Figure 4-B). After talik initiation, the active layer, now defined by the depth of seasonal freezing, thins. The talik deepens throughout the remainder of the century, but a very thin layer of permafrost persists in 2099 (Table 2).

Fire disturbance triggers talik development in all burned scenarios (Figure 4). Under RCP4.5, the thaw front advances rapidly into the permafrost at first, but gradually slows until the talik reaches a depth of 5-6 m. The active layer, defined by the depth of seasonal freezing, initially thins by an average of 35-46 cm when compared to the unburned scenarios over the same period. This thinning trend reverses once the talik reaches its maximum thickness and begins to refreeze. The temperature at the DZAA decreases and permafrost re-aggrades, causing the talik to close after three decades in the low burn severity simulation and four decades in the high burn severity simulation (Table 2). However, much of the re-aggraded permafrost is only partially refrozen, with temperatures between -0.005 and 0°C. Following talik closure, active layer thicknesses in the burned scenarios average 8-28 cm greater than those in unburned scenarios. By the mid-2090s, the temperature at the DZAA is the same in both burned and unburned scenarios, and 2-4 m of permafrost remains at the end of the century (Figure 4-C and 4-E; Table 2).

Under RCP8.5, low severity fire disturbance results in the development of two taliks and the complete degradation of permafrost by 2080. The first talik is initiated immediately following fire, and the thaw front advances for approximately two decades. This is followed by a 20-year period of stability until refreezing occurs from the active layer downwards (Figure 4-D). The ground layers comprising the first talik only partially refreeze and then thaw out during the development of the second talik. In the high burn severity simulation, the fire-initiated talik does not refreeze, and permafrost degrades completely by 2069 (Figure 4-F; Table 2).
Impact and Importance of Organic Layer Accumulation: Sensitivity analyses with no organic layer accumulation result in thaw from the surface downwards in all unburned scenarios. Under RCP4.5, a climate-initiated talik forms in 2082 and deepens so that only 4.9 m of permafrost remains in 2099 (Table 2). Under RCP8.5, a climate-initiated talik begins to form in 2059, 4 years before the talik in the comparable simulation with organic layer accumulation (Figure 4-B), and permafrost is lost by 2095 (Table 2). In all burned scenarios, omitting surface organic layer regeneration results in the total loss of permafrost by the end of the century. Under RCP4.5, permafrost is lost by 2082 and 2066 for the low and high burn severity simulations, respectively (Table 2). Under RCP8.5, the fire-initiated talik does not refreeze in either low or high burn severity simulation, and permafrost thaws by 2069 or 2060, respectively (Table 2).

Table 2. Summary of modelling simulations performed in TEMP/W. Organic layer accumulation at 3.6 mm/year is indicated with an arrow (↑). Note: all fire-initiated taliks began in 2004.

<table>
<thead>
<tr>
<th>RCP</th>
<th>Surface Boundary (Burn Severity)</th>
<th>Organic Layer Thickness (cm)</th>
<th>Fire-Initiated Talik (1965-2004)</th>
<th>Climate-Initiated Talik (2004-2099)</th>
<th>Permafrost Thickness in 2099 (m) or Year of Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>Unburned</td>
<td>15</td>
<td>15</td>
<td>-</td>
<td>8.1</td>
</tr>
<tr>
<td>4.5</td>
<td>Unburned</td>
<td>15</td>
<td>15</td>
<td>-</td>
<td>2082 (2082)</td>
</tr>
<tr>
<td>8.5</td>
<td>Unburned</td>
<td>15</td>
<td>15</td>
<td>-</td>
<td>2063 (2063)</td>
</tr>
<tr>
<td>8.5</td>
<td>Unburned</td>
<td>15</td>
<td>15</td>
<td>-</td>
<td>2059 (2059)</td>
</tr>
<tr>
<td>4.5</td>
<td>Burned (Low)</td>
<td>15</td>
<td>10</td>
<td>2034</td>
<td>3.6</td>
</tr>
<tr>
<td>4.5</td>
<td>Burned (Low)</td>
<td>15</td>
<td>10</td>
<td>2046</td>
<td>2062 (2082)</td>
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<td>Burned (Low)</td>
<td>15</td>
<td>10</td>
<td>-</td>
<td>65 (2069)</td>
</tr>
<tr>
<td>8.5</td>
<td>Burned (High)</td>
<td>15</td>
<td>0</td>
<td>65</td>
<td>-</td>
</tr>
<tr>
<td>8.5</td>
<td>Burned (High)</td>
<td>15</td>
<td>0</td>
<td>56</td>
<td>-</td>
</tr>
</tbody>
</table>

DISCUSSION

Post-Fire Permafrost Response: In our simulations, permafrost thickness is relatively stable and varies by only 0.5 m (3%) during the first 40 model years. A local maximum in permafrost thickness and a corresponding minimum in the temperature at DZAA occurs in 1995 (Figure 4), reflecting the end of a period of regional climatic cooling (Way and Viau 2015; Figure 2-A).

Our simulations present a shallower active layer for the first two decades immediately following fire. This is because a supra-permafrost talik forms in the first year and the base of the active layer is then defined by the maximum depth of seasonal freezing rather than the maximum depth of seasonal thaw. Most field studies describe thickening of the active layer following fire (e.g., Zhang et al. 2015, Gibson et al. 2018, Holloway et al. 2020), but late-summer probing cannot be used to differentiate between the active layer and a talik. Our results do not contradict previous studies, but they emphasize the need for ground temperature monitoring in the field to distinguish between these two layers. The overall understanding of post-fire permafrost response would also benefit from a clear differentiation between an active layer and a talik in modelling studies.

Fire disturbance can trigger thawing of permafrost from the base, as has been modelled for
Permafrost in the Northwest Territories (Zhang et al. 2015). Despite the relatively high geothermal heat flux that was applied to the base of our model domain, the post-fire persistence of permafrost at the WB site speaks to the resiliency of perennially frozen ground in the region. Omission of surface organic layer accumulation caused rapid permafrost degradation due to the impacts of rising air temperatures. The organic layer is largely responsible for the thermal offset, which enables permafrost persistence following disturbance or in unfavourable climates (Smith and Riseborough 2002). Our results support previous findings of the importance of the organic layer in post-fire permafrost stability (Fisher et al. 2016) and of including organic layer regeneration in post-fire permafrost modelling (Zhang et al. 2015).

**Permafrost Resiliency:** The resiliency of the permafrost at WB can be assessed in relation to the climate-ecosystem-permafrost types defined by Shur and Jorgenson (2007). Prior to fire disturbance, permafrost is interpreted to be climate-driven, ecosystem-protected. Following fire, permafrost re-aggrades via ecosystem-driven processes and is re-classified as ecosystem-protected permafrost. Under RCP8.5, climate warming overcomes the ecosystem’s ability to protect the permafrost, resulting in the development of a climate-initiated talik and the loss of permafrost.

Our simulations agree with previous studies that have demonstrated that fire acts to accelerate the loss of permafrost (e.g., Zhang et al. 2015, Gibson et al. 2018). Under the same climate scenario, burned simulations result in thinner and warmer permafrost that thaws out more quickly than in unburned simulations. Under RCP4.5, by 2100, low and high severity fire disturbances at WB result in respective permafrost thicknesses that are 56% and 73% thinner than the ~8 m of permafrost remaining in the undisturbed simulation. Loss of permafrost due to fire disturbance in the Northwest Territories is modelled to occur an average of 8 years earlier when compared to undisturbed sites (Zhang et al. 2015). At WB, loss of permafrost following fire is modelled to occur at least 24 years earlier when compared to undisturbed simulations, at a rate that is three to four times faster than that reported for the West. This accelerated degradation is likely due to the warmer, less ice-rich permafrost (i.e., with lower latent heat) at our site compared to conditions in the central Northwest Territories (Smith et al. 2015) and may also reflect the slower post-fire forest regeneration processes reported for WB (Brehaut and Brown 2020) compared to the pulse recruitment that has been documented following fire in the West (Johnstone et al. 2004).

The development of climate-initiated taliks in the latter half of this century under RCP8.5 shows the sensitivity of permafrost at WB to the impacts of climate warming. Taliks produced by warming deepen more linearly than those initiated by fire disturbance. Fire-initiated taliks develop in the immediate post-fire period due to the sudden effects of increased heat penetration in summer and decreased heat loss in winter (Gibson et al. 2018). However, post-fire talik development eventually slows as the ecosystem recovers from disturbance (Gibson et al. 2018). By contrast, permafrost degradation due to climate warming occurs more gradually, and climate-initiated taliks do not re-freeze (O’Neill et al. 2020).

**Modelling Limitations:** The modelling involves several generalizations that could have affected the individual predictions but are not thought to have altered the overall results. First, n-factors were held constant for 5-year periods (Figure 2-B), but in the field, they can vary over short distances and time periods (Karunaratne and Burn 2004). Nf is sensitive to inter-annual differences in snow cover (Karunaratne and Burn 2004) and can be impacted by permafrost presence or absence (Riseborough and Smith 1998). A reduction in nf following talik development would delay talik recovery and possibly accelerate permafrost loss. No adjustment method is currently available for this purpose. Second, post-fire variations in soil moisture, which typically relate to the remaining organic layer thickness (Kasischke and Johnstone 2005), were not considered due to
limited field-based validation. Post-fire soil moisture is generally expected to increase due to thickening of the supra-permafrost layer, poor drainage, and reduced evapotranspiration (Nossov et al. 2013). Soil moisture is eventually expected to return to near pre-fire conditions as the site recovers from the fire disturbance (Smith et al. 2015, Gibson et al. 2018). Future simulations incorporating dynamic soil moisture and the lateral and/or vertical flow of water would provide insight into how changes to the post-fire hydrological regime might influence soil freezing properties and characteristics. Finally, equal-weighted averaging was performed to assemble the composite record for projected air temperatures (2018-2099) under RCP4.5 and RCP8.5 (Figure 2-A). The resulting composite records mask inter-model variability. Individual climate scenarios would have produced more variability in the timing and rate of post-fire permafrost change.

CONCLUSION

The Webb Bay study site near Nain, Nunatsiavut is wetter, accumulates a deeper winter snowpack, and experiences slower and more continuous post-fire forest regeneration processes compared to sites in western North America where most research on forest fire and permafrost has been carried out. Despite these differences, the results of our thermal modelling agree with the prevailing view that fire disturbance accelerates permafrost loss due to climate warming. Following fire and within the context of a warming climate, permafrost at this coastal boreal site is modelled to persist to at least 2060 and, in many cases, through to the end of this century. Simulations highlight the role of the organic layer in permafrost persistence under otherwise unfavourable climatic conditions and the importance of differentiating between the active layer and a talik, if present, in the supra-permafrost layer. The results extend geographic coverage and improve our understanding of active layer change and talik formation as part of the overall long-term post-fire permafrost response.

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Engage the Public in Science and Embrace Future Change with Human-Centric Stories, Art, and Imaginings

Stacey A. Fritz, Ph.D.¹; and Patricia Romero-Lankao, Ph.D.²

¹Cold Climate Housing Research Center–National Renewable Energy Laboratory, Fairbanks, AK. E-mail: Stacey.Fritz@nrel.gov
²Center for Integrated Mobility Science, National Renewable Energy Laboratory, CO. E-mail: Patricia.RomeroLankao@nrel.gov

ABSTRACT

Human-centric stories that weave in real scientific data may be able to engage the public in environmental issues that do not yet directly affect them. Science fiction and artistically rendered futuristic scenarios can unleash the imagination and act as a lens to envision technological, social, and cultural aspects of transitioning to clean energy. Cultures with strong oral traditions use stories to record history, develop a shared identity, pass on environmental lessons, and prepare for future change. Through the lenses of our non-physical science disciplines (cultural anthropology and environmental sociology), we discuss challenges to communicating science, the use of narratives to illustrate ecosystem processes, and we report on a collaborative, interdisciplinary workshop and book project that is creating narratives of hope and visions for the future through inspiring art, short stories, and essays. We explore the surprising potential of ‘cli-fi,’ humor, games, and the research behind emotion and imagination-driven engagement. We describe methods that help people visualize their future well-being and we explore opportunities to spread those methods.

INTRODUCTION

Chatting with an elder Iñupiaq friend from the village of Wainwright on the Chukchi Sea a few years ago, the subject of warming permafrost came up. Charles knew about all the village ice cellars and the issues people were having with them. Problems keeping the harvest frozen were important, but they were not as dramatic as all the ponds and lakes that were disappearing in sudden drainage events in the surrounding area. Those had him truly surprised – and he had seen a substantial amount of change in his life. For now, he explained, people could adapt to the warming ice cellars by being extra careful to leave them uncovered as little as possible. Also, Charles confided that there was sometimes a silver lining. If cellars are just a little warmer – just the right temperature to allow things to age slowly but still safely – the meat can be extra delicious (C. Ekak, personal communication, 2017). If you enjoy this vignette, it may be because you know about permafrost, ice cellars, the region, or the widespread arctic tradition of fermenting meat.

Engaging the general public in science of arctic change, by contrast, is challenging at any time and particularly difficult during the current era. Ever increasing amounts of dire warnings, overlapping immediate crises, and growing distrust of the media and science are leading to widespread disconnected helplessness. Many people do not have the time or luxury to worry about arctic change because they face more immediate problems. In addition, many who do pay attention to it suffer from ‘climate despair,’ a sense that climate change is an unstoppable force that will render humanity extinct and renders life in the meantime futile,” (Pearl 2019).

Communicating climate change in general presents great challenges because it has been both temporally and spatially remote from most individuals (O’Neill and Nicholson-Cole 2009). The Arctic is particularly remote and communication about change there to people that live in lower
latitudes is furthermore confounded by a lack of baseline understanding about the Arctic: it is difficult to discuss permafrost degradation with people who have no experience with permafrost.

Furthermore, those trying to share arctic change information with the public are competing in what is known as the attention economy, a large and growing segment of the modern economy which “increasingly revolves around the human attention span and how products capture that attention,” (Kane 2019). Acknowledging this context – that attention is a scarce economic resource and one of the most valuable resources of the digital age – is critical for effective communication.

For most of human history, access to information was limited. [...] Now we are presented with a wealth of information, but we have the same amount of mental processing power as we have always had. The number of minutes has also stayed exactly the same in every day. Today attention, not information, is the limiting factor. (Kane 2019).

This ruthlessly competitive attention economy combines with our current media landscape to create additional challenges for communicating arctic change effectively. A 2019 study that tracked the digital footprint of several hundred climate contrarians with the same number of expert scientists found that contrarians were featured in 49% more media articles (from all sources) than scientists (Petersen et al. 2019). When the comparison was limited to mainstream media sources, researchers found only a 1% excess visibility for contrarians. This shows that the proliferation of new media sources, many of which disseminate climate change disinformation, is crowding out professional mainstream sources (Ibid.).

If there is any arctic change information that can push past these high barriers to connect with people, it may be good, human-centric stories and visualizations that stick in people’s minds. We review some interesting research and popular non-academic media on stories and provide a few possible tales of arctic change, principally to illustrate various strategies and pitfalls and the psychology behind the power of narrative. We encourage scientists to use narrative but caution them against the dangerous potential of stories for their science before they embrace them. Aligned with the vision of this conference session, we advocate for engaging anthropologists, sociologists, behavioral specialists, artists, comedians, gamers, and others to work together on efforts to communicate effectively.

WHY STORIES WORK

Experts have largely agreed that stories, art, and emotional connection are effective ways to engage people and help them remember things. Behavioral psychologist Susan Wienschenk (2014) explains that stories are, in fact, our normal mode of processing information and that when we hear stories, more areas of our brains are engaged than when we take in and process facts and figures. In fact, our brains are hard pressed to incorporate any facts and figures until we understand the context through a story (Grankvist 2019a). Stories can activate our olfactory sensory areas, our motor cortex, and, perhaps most importantly, the empathy areas of the brain (Wienschenk 2014). People take in information if it is, as filmmaker Cheryl Miller House (2020) explains, “in service to a story that touches their hearts.” A more diverse brain event is more pleasant, causes people to better understand information, and helps them retain it longer (Weinschenk 2014). A critical point regarding arctic change is that more pleasant brain events do not depend on pleasant stories. For example, trying to teach government officials rules and ethical standards is much less effective than telling them shocking true stories of what has actually happened when officials have broken the rules (DoD 2009).
Based on our research and review of the literature, we can assert that some stories, when conveyed correctly, can make us relate to and feel empathy for individuals we do not know. Psychologists have demonstrated that people have a harder time empathizing with larger groups of people – even if group members all share a similar situation. The identifiable victim effect is the idea that a single individual’s story is more compelling than a group of people with the same need (Small and Loewenstein, 2005). Identifying a victim and relating to them builds empathy and it also helps people make up their minds. Research has found, for example, that jurors’ verdicts in trials rely heavily on the stories they are able to build and identify with in their own minds from the evidence they hear (Pennington and Hastie 1988).

Our work on climate change and sustainability with U.S., Latin American, and Asian cities has also taught us that visions and narratives about the future affect actions in the present (Lente 1993, Borup et al 2006, Romero-Lankao et al 2019 and 2019, Geels and Verhees 2011, Miller et al 2015). Visions entail images, assertions, stories, and ideographs. They shape actors’ agendas and a kind of division of social roles, with some playing the advocate, or the supporter, and others playing the adversary. When urban communities form political groups or coalitions, a range of visions or narratives coexist in a contested arena. Some narratives are full of hope for a promising future; others are full of anxiety about the risks that lie ahead; yet others are ambiguous. Visions are fundamental to a climate future as they shape actors’ expectations and actions. For example, some transnational organizations, countries, and cities have announced ambitious mitigation goals. At the same time other actors hold dystopian visions, or they just oppose the necessary actions to come.

Many in Alaska are fortunate to have regular evidence of how well people remember stories when we talk with indigenous people who have been passing on stories of arctic change and rules for surviving in the Arctic for thousands of years. Indigenous peoples’ orally transmitted stories are not always as straightforward as some people would like (e.g., Schneider 2002). Indigenous scholar Linda Tuhiiwai Smith describes that, especially when with indigenous researchers, “[] indigenous elders can do wonderful things with an interview. They tell stories, tease, question, think, observe, tell riddles, and give trick answers,” (Smith 2001). Non-indigenous researchers are commonly puzzled when the answer to a relatively straightforward question comes in the form of a seemingly unrelated story (Schneider 2002). Yet these long-lasting tales can hold the greatest accounts of environmental change over time. A linguist and a geographer who collected and compared traditional stories of Aboriginal Australians found that the seemingly unrelated and supernatural tales accurately describe sea level changes around 10,000 years ago with detailed information passed along over 300 to 400 generations (Nunn and Reid 2015). Along those lines, permafrost scientists in Alaska may be familiar with a site on the North Slope’s largest river, the Colville or Kuukpik. The site is about 50 miles upriver from the Beaufort Sea coast, the village of Nuiqsut, and the Alpine oil field. It is called Itiglak - known in English as Ocean Point, and everyone there understands that from long ago their people used to say it was the edge of the sea. This highlights one of the reasons many anthropologists value place names research. Place names tell stories, convey lessons from the past, and commonly involve guidance on how to survive in harmony with a specific environment.

**SCIENTIFIC DATA IN STORIES**

These basic understandings of the science of stories should encourage scientists hoping to disseminate information about arctic change to avoid, for example, summarizing the findings of the U.S. Army Corps of Engineers 2018 report on erosion control options and the status of
permafrost underlaying Utqiaġvik’s infrastructure. They could focus instead on one relatable Iñupiaq father’s struggle to keep his family home from sinking, his family’s winter food stores from rotting in a melting ice cellar, and his inability to use traditional trails to access his hunting sites when he needs to or the now-unstable shore-fast ice to reach the open ocean. Once the audience can relate to and is invested in this one man’s struggle to provide for his family, the story could start weaving in data about the dramatic erosion rate or the implications of the fact that the coastal permafrost under the town is actually ice rich and “unbonded” (saline) permafrost that thaws at lower temperatures than regular permafrost (Ibid). Perhaps the man is aware that one moderate storm from a specific direction could inundate the town’s only freshwater lake or flood the underground utilidors on which the city depends (Oliver 2016). It is likely that the man’s father worked for the military when it established outposts just north of the village in the 1950s: many residents got heavy equipment jobs dredging mountains of gravel from beaches that protected the coastline for that industrial development. The man knows that other Alaskan villages are relocating or trying to, so his mind naturally calculates where the closest higher ground is for his community, and he immediately realizes that it is hundreds of miles away. Because his entire society has always been organized around whaling crews and his culture is fundamentally connected to the sea, it is hard for him to envision his children living as Iñupiat in that future.

The story outline above might be emotive and individualized, but it fails to share a compelling vision of the future, it is not upbeat, and there are other reasons to exercise caution before embracing a strategy of interspersing data into human-centric narratives to illustrate ecosystem processes. Interrupting a story with data while telling it might make both the story and the data less memorable. Scientists trying to use stories to convey data risk breaking the all-important flow of a story by diverting resources and the reader or viewer’s attention. An even greater risk is that people may reject the science if they perceive they are being emotionally manipulated by scientists with an agenda.

For those and other reasons, storytelling can be dangerous. Emotive storytelling is an age-old practice of persuasion, thus scientists embracing human-centric narratives may simply be embracing well-accepted tools of propaganda. The current rise of populism can be attributed to widely told and alluring stories of how we will make the future look like a mythical past (Grankvist 2019b). Scientists may not want to risk more confusion, more resistance, or greater controversy by inserting data into narrative. They may find it unethical to do so, and they may think the risk to the credibility of the science is too great.

Scientists may be averse to using stories to convey data because they know that the availability of information is not a problem. The public has access to more information than at any time in the past, but people are resistant to information that appears to inform them they should change how they live or that seems to threaten their autonomy. Because scientists cannot control how the media portrays arctic change data, and the media model is to magnify alarm to win our attention, scientists must use care. The media knows that fear is the most attention grabbing emotion. It is stronger than hope. Moreover, it should be simple to convey fear while describing people living on ground that is rapidly disintegrating underneath them. The Yup’ik even have a specific word that conveys the kind of change we are seeing in the Arctic: устек. Устек is the compounding effects of permafrost thaw, erosion, and flooding that threaten catastrophic land collapse.

Despite the fact that fear is a particularly powerful emotion, the evidence indicates that it does not encourage people to take productive action. When researchers presented climate-related data visualizations and urged test subjects to take action using fear-based terms, people usually responded with “denial, apathy, avoidance, and negative associations,” (Pearl 2019, O’Neill and
Fearful climate change images may evoke powerful feelings about the issue but may also reduce the participants’ convictions that they can do anything about it (O’Neill and Nicholson-Cole 2009). This means that “dramatic representations must be partnered with those that enable a person to establish a sense of connection with the causes and consequences of climate change in a positive manner – so that they can see the relevance of climate change for their locality and life, and that there are ways in which they (and others) can positively respond,” (Ibid).

Besides urgency, narratives should provide meaningful options for action. Fear is unproductive, but scientists should also avoid narratives that merely evoke guilt or lead people to blame themselves, and credit is due to climate scientists Michael Mann and Kathleen Hayhoe for making this case (Joselow 2019). Mann and Hayhoe naturally want individuals to reduce their personal carbon footprint, but they also want us to recognize that many climate stories are promoted to distract us – they are designed to make the relatively elite people who have the luxury to do so feel individually culpable, to worry about shopping totes and buying carbon offsets to mitigate our airline flights. Mann contends that these stories are convenient for corporations and governments because they keep us trapped in thinking we are to blame and that our individual actions make a meaningful difference (Ibid). This focus can sow division and strife among people who share concerns about climate change, resulting in people spending their energies flight- and hamburger-shaming each other. Our individual actions help, but the statistic that should occupy our bandwidth is that about 100 companies are responsible for about 70 percent of fossil fuel emissions. Furthermore, research suggests that when people feel they are making an impact on climate change in their personal life, they are significantly less likely to support a carbon tax or other climate policy at the government level (Ibid).

Some might appreciate the increasing media attention on the spikes of methane gas release and collapsing pingos creating massive craters in Siberia. Perhaps these events present opportunities for conveying understandings of arctic change, especially because the human brain is not designed to respond to slow-moving crises but is better designed to respond to obvious, immediate threats. U.S. arctic change scientists might begin strategizing about how they will most effectively promote arctic change information if a crater appears in North America: be ready to identify an individual victim, portray it as a natural disaster, and weave in data about small temperature changes in sediments producing huge amounts of methane gas. The Siberian version of the story is that one summer day in 2017 on the Yamal Peninsula in the Siberian Arctic, an indigenous Nenets reindeer herder heard a loud blast, felt the tundra rumble, and saw smoke rising from the ground (Bressan 2020). Let’s imagine that he secured his herd and children safely before going to investigate. He was dumbfounded to see that a massive crater had opened up in the earth close to his home where his reindeer normally graze, but he had already heard rumors about the first such explosion, recorded in 2013, that was heard over a distance of 62 miles. Other indigenous reindeer herders have reported seeing flaring flames and a column of smoke from areas where craters formed (Ibid).

That might be memorable, yet scientists want narrative that helps people understand arctic change. Unfortunately, the most memorable stories are about anomalies and it is easy for people to accept anomalous stories as the norm. The permafrost craters in Siberia provide an excellent example – it is extremely rare to be personally affected by one, but they provide a memorable story that might lead people to assume that, eventually, the tundra will be exploding to form craters everywhere. Moreover, the ongoing research into the craters is not always made clear to the audience. What has been promoted is that indigenous Nenets and Khanty have been keeping reindeer herds in the area for at least 1,000 years, and the area is now home to Russia’s largest natural gas fields. While some scientists have worried that either warming temperatures or
infrastructure and drilling associated with gas could be exacerbating the conditions that lead to the creation of the funnels, researchers associated with the gas industry have a competing narrative: they blame over-grazing of reindeer for reducing the lichen and grasses on the tundra, reducing its albedo (Nilsen 2016).

If shocking anomalies do not produce the education desired, perhaps humor is effective. One sunny summer day, a group of people sat on the bluff of the beach on the Chukchi Sea. The community was still celebrating the unprecedented ease and safety with which their crews had harvested bowhead whales the previous year. For the first time in anyone’s memory, there was no shorefast ice in front of town during the whales’ migration. What this meant was that people were able to sit at home in the comfort of their living rooms and keep an eye out the window for whales, instead of laboriously cutting a path through ice blocks and drifts out to the edge of the ice, hauling out boats and gear, and setting up camps to wait for the whales while also keeping an eye on the wind, weather, and ice conditions to make sure the ice they were on did not break off and carry them out to sea. The subject of climate change came up and my acquaintance expressed ambivalence. He seemed scornful of all the hubbub and he rejected the alarmism. As he said he did not see what the big deal was, he gestured out to the wide blue ocean: “The only difference I see is that there is no more ice!” Another example of dark irony regarding arctic change is the support the indigenous-led municipal government of the North Slope has not just for onshore and nearshore oil development, but for almost any development, including roads and pipelines, that are oriented along the coast. They believe that the only chance they have to protect the eroding coastline is if industry and the government are constantly fortifying it.

These true stories display one quality of narrative that could be explored: humor. However, they both display a rather lazy and common dark humor, or gallows humor, the kind that George Carlin employed to mock humans decades ago for their arrogance in thinking they could either save or destroy the planet when they haven’t even learned to take care of themselves. Since then, research has indicated that is actually good-natured comedy, rather than cynical satire, that may be most effective at engaging the public. A survey of 30 students who were previously depressed by the subject of climate change but were required to put on comedy acts found they were dramatically more hopeful and believed their commitment was more sustainable (Osnes et al. 2019). Comedian and environmental economist Matt Winning believes a mixture of highs and lows, humorous and dark points, is the meaningful way to engage.

IMAGINING OUR FUTURE SELVES

As mentioned above, stories are most effective when we are able to imagine ourselves as the identifiable victim. Yet people have a hard time relating to the person they will be in decades to come. Some data suggest that, because we are disconnected from our future selves, we have trouble visualizing our future well-being. Here, futuristic science fiction – and even dystopian climate fiction, known as ‘cli-fi,’ in which humans are somehow still muddling along – can help us relate to ourselves as either protagonists or at least surviving victims in a future world.

One survey found that stories compel “readers to imagine potential futures and consider the fragility of human societies and vulnerable ecosystems,” (Schneider-Mayerson 2018). That’s why we engaged in a collaborative, interdisciplinary workshop and book project (NREL and Arizona State University). At this unique workshop, science fiction authors, graphic artists and NREL experts used science fiction as a lens to explore the technological, social, and cultural aspects of a transition to a clean energy future. We produced a book about imagining the future of the post-carbon city.
There is agreement that, in the next few decades, humanity will need to decarbonize its energy systems and adapt to address climate change. Cities are where people use and consume about 75% of the world’s energy. Cities, also, are where a large proportion of the world’s energy systems are managed even if they are brought from more distant places. Internationally, thousands of city officials and business have pledged to become carbon neutral.

The book combines stories and essays around the cities of the future, to help unleash the human imagination to the many options to organize the solar-powered city. Because the future is not fixed, the book explores the many possible sociotechnical paths cities might follow. It also glanced at implications of these futures for people’s hopes and dreams, lives, and livelihoods.

The large Swedish government funded innovation program called Viable Cities, which aims for climate neutral cities in Sweden by 2030, embraces the power of narrative to achieve their goals. They recognize that “[s]tories can nudge people in the right direction and engage us in a way pure facts cannot,” (Viable Cities 2019). To this end, Viable Cities has engaged a “Chief Storyteller,” Per Grankvist, who is establishing a playbook that supports citizen co-creation of a wide collection of stories that will help make people embrace the future instead of fear it. The stories will lead to better understandings of various perspectives that will help engage everyone affected and solutions that satisfy as many as possible (Grankvist 2019b), and all aspects of this storytelling effort will be made available to everyone via open source.

Grankvist (2019a) compares the perspective that providing empirical facts and figures will lead people to make the right choice to the myth of the rational economic man. Both are mythical because humans are not logical but are instead more governed by emotions. Accordingly, appealing to audiences who are primarily concerned about their financial interests means that communication about arctic change could be linked to specific economic data and financial forecasts. By this we do not mean a focus on how much transitioning off fossil fuels will cost, but about how savvy people will get rich by embracing the future. Put simply, expound on examples of those profiting off sustainable solutions (Grankvist 2019a) to help members of the public identify themselves in eco-success stories as they narcissistically visualize a wealthy future version of themselves. Instead of being restricted, innovative entrepreneurs can thrive off the standards set by sustainability goals in their competitions. The authors of Big World Small Planet contend that the mind-shift that will come from accepting our planetary boundaries will help unleash a new wave of creative, profitable, and sustainable technological inventions and trigger “an abundance of ideas and solutions for human prosperity and planetary stability,” (Rockström and Klum 2015). Moreover, playing the tech stock market can be engaging and is a thrilling game for many, which leads to our final suggestion for communicating arctic change: gamify it.

A 2015 analysis of the genre of climate change games, which references the 5 decades worth of research supporting the efficacy of game-based learning, found a dramatic rise in digital climate games and considered strengths and weaknesses of various types (Wu and Lee 2015). The best games encompassed every effective communication strategy we have discussed here and more. These games:

- Provide “designed experiences” where people learn through doing and being, triggering emotional pathways.
- Allow players to build empathy by taking on various roles and perspectives.
- Allow for visioning - for example, being able to envision oneself in the future - and seeing consequences of actions at different points in time.
- Are highly engaging and tap into a range of human emotions, from fear and aggression to joy and wonder.
• Target affective outcomes, such as players’ motivations, attitudes, and values.
• Promote a winner’s mentality, also described as ‘urgent optimism’ and the belief that an ‘epic win’ is always possible. (Epic win’ refers to finding solutions to difficult problems, which is particularly apt for addressing climate change.)
• Make it easier and less intimidating to identify new strategies through inconsequential trial and error. (Wu and Lee 2015).

The research also notes the potential scale of games: “60% of Americans play videogames, or an estimated 185 million people. As a result, gamers represent a large potential audience for raising awareness,” (Ibid.). Research on the long-lasting benefits of games is still underway, but the authors conclude that the emerging trend of “pervasive games,” which blend digital and physical mediums, may be especially suited for promoting concrete action (Ibid.).

In the game called EcoChains: Arctic Life, cards with gorgeous art let players build food chains and protect sea ice. It’s a “family-friendly game of strategy and survival” that is “aligned to next generation science standards,” (Lee et al. 2019). Carbon pollution event cards result in each player melting two of their sea ice cards, but you can migrate your animal to keep it alive and the player who keeps the most animals alive wins (Ibid.). We were first introduced to this game by the Alaska Native educator and Tribal Liaison at the Alaska Climate Adaptation Science Center, who loves this game and, with her colleagues, used it extensively in the Reaching Arctic Communities Facing Climate Change project (M. Chase, personal communication 2020).

CONCLUSIONS

Despite the many challenges, we hope this review of research relevant to communicating arctic change encourages scientists to experiment based on strong evidence of what makes effective storytelling. It is liberating to know that there is no one way: we need numerous different stories to appeal to numerous different audiences. Scientists and the public cannot afford to let sensationalist media attempt to communicate arctic change because the media relies on alarmist and largely negative narrative and images to garner attention while broad segments of the population do not trust the media. Their goal is gaining the public’s attention, but not necessarily engaging the public in a productive way. Collaborating with gamers, comedians, artists, and social and behavioral specialists in truly transdisciplinary efforts helps ensure communication is relevant.

Because the Arctic is so remote to most people, communicating arctic change successfully may depend on making obvious connections with other, real-time, and even small changes in other regions - things that are real and close to people in time and space. There are now plenty of examples. A review of recent extreme climate change-induced events is outside the scope of this paper but suffice to say: effects are being experienced in places far from the Arctic. Much of the research we referenced here is at least 2 years old, and some of it is over a decade old. Research in 2006 (Lowe et al.) and 2008 (O’Neill) indicated that individuals were likely to feel that dangerous climate change would not affect them for many years, if at all. Individuals had difficulty imagining 15 to 20 years into the future. We anticipate that research being conducted now, after the disastrous fire seasons in Australia, California, Oregon, and Washington, and more hurricanes stacking up across the Atlantic, may find that many more individuals can visualize it as something real and close to their everyday lives.

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An Estimation of Past and Present Air Temperature Conditions, Water Equivalent, and Surface Velocity of Rock Glaciers in Cordillera Volcanica, Peru

Edwin Badillo-Rivera1; Edwin Loarte2; Katy Medina3; Xavier Bodin4; Guillermo Azócar5; and Diego Cusicanqui6

1National Institute for Research on Glaciers and Mountain Ecosystems–INAIGEM, Huaraz; Faculty of Environmental Engineering and Natural Resources, National Univ. of Callao, Bellavista-Callao, Peru. E-mail: enbadillor@unac.edu.pe
2National Institute for Research on Glaciers and Mountain Ecosystems–INAIGEM, Huaraz, Peru
3National Institute for Research on Glaciers and Mountain Ecosystems–INAIGEM, Huaraz, Peru
4Laboratoire EDYTEM, Université de Savoie Mont Blanc, CNRS, Le Bourget-du-Lac, France. E-mail: xavier.bodin@univ-smb.fr
5Atacama Ambiente Consultores, Santiago, Chile. E-mail: gazocar@atacamamb.com
6Laboratoire EDYTEM, Université de Savoie Mont Blanc, CNRS, Le Bourget-du-Lac; Univ. Grenoble Alpes, CNRS, IRD, Institut de Géosciences de l’Environnement (IGE, UMR 5001), Grenoble, France. E-mail: diego.cusicanqui@univ-grenoble-alpes.fr

ABSTRACT

Rock glaciers (RG) are one of the most important geomorphological features in the Peruvian Andes. However, the local characteristics of RG have barely been studied or remain unknown. The aim of this research was to characterize past and present conditions of the RG located in Cordillera Volcanica in the southern of Peru. For this purpose, an inventory of RG was carried out and modern and past regional mean air annual temperatures (MAATs) were calculated. We estimate the water equivalent of RG to assess their importance as possible storage of frozen water for past and present conditions using an empirical rule. In addition, the local surface velocity of RG was obtained from Landsat 8 imagery. Within the study area, 187 RG were identified (surface area of 8.3 km2). Of these, 63 were classified as inactive, 39 as active and 85 as relict forms. The altitudinal distribution of RG ranges between 4616 to 5551 m a.s.l. (meter above sea level) where modern MAAT is 0.9°C. In the current conditions, relict RG are located in positive MAAT levels around 1.4°C, however, for the past conditions, relict RG were located in negative MAAT levels around -5°C. The amount of water stored in intact RG range between 28 and 64 million m3. Meanwhile, for past conditions (paleo-WVE), we estimated that volume stored within rock relict RG was between 16 and 35 million m3 (we assume an ice-rich layer of RG permafrost has between 20–45%). On the other hand, the average surface velocities of the active RG have been estimated between 1 to 10 cm/month. The finding of this research contributes to increasing knowledge about RG in the Peruvian Andes, however, further research is needed to understand the importance of RG as stores of frozen water during the past and present conditions.

Keywords: Relict rock glacier, Intact rock glacier, water equivalent, surface velocity, MAAT

INTRODUCTION

The high mountain regions are experiencing accelerated changes due to the increase in global temperature in the last century that triggers multiple impacts on human habits and environment systems (IPCC, 2019). The Andes Mountain is considered one of the most sensitive environment to climate change (Schneider et al., 2014) due to a temperature increase of 0.13°C/decade between the years 1936 and 2006 (Vuille et al., 2008) and 0.17°C above 5000 m a.s.l.,. Future temperature
projection indicate that by the end of the XXI century, the temperature in the Andes will increase by between 2 to 5.0 °C (Pastick et al., 2015; Cabré et al., 2016 PTT). Peru has to 71% of the world’s tropical glaciers (Kaser & Osmaston, 2002; ANA, 2014), and in the last 50 years about 54% of its glacier surfaces have disappeared (INAIGEM, 2018). To date, the characteristics of RG in the Peruvian Andes remain unknown and they have been barely studied in the past years. (Ahumada et al., 2014; Azócar et al., 2017; Haeberli et al., 2006).

RG are commonly defined as geomorphological features that consist of mixture of rock with variable ice content (0.1 to 60%, Janke, Ng, & Bellisario, 2017). They usually have ridges, furrows and sometimes lobes on their surface and steep front at the angle repose (Potter, 1972). RG have been used to infer the lower limit of permafrost distribution (Angillieri, 2017).

RG, according to their dynamics, can be classified as active forms (They still contain ice and move downslope), inactive forms (They could still contain ice and stopped move downslope) and relict forms (They don’t display any movements and the ice has completely melted (Brenning, 2005; Azócar, 2013; Jones et al., 2019). Normally, active and inactive RG are commonly grouped intact forms (Brenning, 2005b; Schmid et al., 2015). Relict RG are also considered as paleoclimatic indicator of past permafrost conditions. (Colucci et al., 2016; Kinworthy, 2016; Seligman, 2009).

In the past decades, several studies have been carried out to understand the characteristics and dynamics of RG in the European Alps and Himalayas mountains (Boccali et al., 2019; Frauenfelder & Kááb, 2000; Haq & Baral, 2019; Jones et al., 2017; Marcer et al., 2017; Necsoiu et al., 2016; Roer et al., 2005; Schmid et al., 2015; Scotti et al., 2013). On the other hand, in South America, most studies have been conducted in the Argentine, Chilean and Bolivian Andes (Angillieri, 2017; Azócar, 2013; Brenning & Azócar, 2010; Perucca and Esper Angillieri, 2011; Rangecroft et al., 2014; 2015) and more recently in Peruvian Andes where RG have been inventoried and local studies have been carried out in Chila and Huanzo cordillera (Medina et al., 2020).

The aim of this research is to characterize the current and past conditions of RG in the Volcanic Cordillera in southern Peru. For this purpose, RG were classified and mapped in term of dynamic status. In addition, the current MAAT and paleo-MAAT of the RG were estimated. Moreover, water volume for present and past conditions of RG were estimated. In addition, displacement movements of RG were inferred using optical image correlation.

MATERIAL AND METHODS

**Study area:** The Cordillera Volcanica is one of the two extinct glaciers mountain and is part of the Western Cordillera of the Andes and its surface drainage mainly to the Pacific. The Cordillera Volcanica cover an area of 5805 km2 between 15°36’ to 16°51’ south latitude that extends along to Arequipa and Moquegua departments (INAIGEM, 2018). In term of weather conditions, the region has a dry period between October and November where precipitation are scarce in comparison to the wet period, where snow is mainly present above 3800 m a.s.l. between December and September.

**Mapping, activity and altitude of rock glaciers:** RG cannot be easily digitized from remote sensing techniques because they are spectrally similar to their environment (Rangecroft et al., 2014), therefore, the optimal approach for mapping and classifying RG is to manually identify particular visual features of them such as forms, ridge and furrow on their surface, appearance of the RG front, among other geomorphological indicators (Roer & Nyenhuis, 2007; Rangecroft et al., 2014; 2015; Azócar, 2013; Brenning & Azócar, 2010; Angillieri, 2017; Sattler et al., 2016 and
Marcer et al., (2017). For this purpose, high resolution spatial satellite images such as Bing Maps and Google Satellite were used. An identifier was assigned to each RG and dynamic status was determined based on criteria of recognition proposed by several authors (Azocar, 2013; Jones et al., 2017; Martini et al., 2013; Rangecroft et al., 2014). The altitude characteristics of the RG was extracted from a 12.5 m. spatial resolution Alos Palsar digital elevation model.

![Fig. 1. Location map of the Cordillera Volcanica](image)

**Estimation of the MAAT and paleo-MAAT:** To estimate the MAAT and paleo-MAAT of each RG, WorldClim 2.1 global average temperature products from 1970-2000 were used (www.worldclim.org; 30 seconds; 1 km² resolution). This climate product has been used in several studies in the Andes (Angillieri, 2017; Drewes et al., 2018; Rangecroft et al., 2014) and other mountains around the world (Haq and Baral, 2019; Sattler et al., 2016). In general, WorldClim data show a good correlation with information collected from local weather stations ($R^2>97\%$; Anderson-Teixeira et al. 2015). Therefore, it is considered a good source of climate data for regional studies. The MAAT will be extracted from each RG. To estimate the paleo-MAAT of the relict RG, the methodology adopted by Millar and Westfall (2008) and Seligman and Brown (2009) was used, in which they estimated the paleo-MAAT using the altitude of the foot of the relict RG.

$$Paleo-MAAT = (H_{RGr} - H_{iso}) \times GTV$$

Equation 1

Where $H_{RGr}$ is the altitude of relict RG foot, $H_{iso}$, is the altitude of the current isotherm -2°C and GTV is the vertical temperature gradient, 0.0065°C/m for the tropics (Kaser and Osmaston, 2002). It is explained that the methodology used to estimate the paleo-MAAT only estimates the temperature when relict RG were formed, but not the date (Kinworthy, 2016), however, relict RG are commonly thought formed in the late Pleistocene (Barsch, 1996).

**Estimation water volume equivalent (WVE) and paleo-WVE of the rock glacier:** To estimate the WVE and p-WVE of the RG, the empirical formula proposed by Brenning (2005) was
used which has been used in several studies (Azócar & Brenning, 2010; Boccali et al., 2019; Janke et al., 2017; Jones et al., 2017; Perucca and Esper Angillieri, 2011; Rangecroft et al., 2015), where:

\[
\text{Mean rock glacier thickness} = 50 \times (S[km^2])^{0.2} 
\]

Equation 2

S represents the area of each RG, the value obtained is then multiplied by the percentage of ice contained in each RG (IC, %). For this study we assumed an ice content between 20 to 45% for intact RG and ice density (\(\rho\)) of 900 kg/m\(^3\). To estimate the p-WVE, the same procedure was applied to the relict RG.

\[
\text{WVE} = \text{Mean rock glacier thickness (m)} \times \text{IC} \times \rho 
\]

Equation 3

Estimation of horizontal velocities of the rock glacier: To estimate the horizontal velocities of RG, the co-registration of optically sensed images and correlation (Cosi-Corr) was used (Leprince et al., 2007). For this purpose, Landsat 8 imagery (panchromatic band of 15 m resolution) was acquired 5 images during the dry season between 2015 to 2019.

Optical image correlation was applied using Fourier correlation, with a window size of 32x16 pixels and a step of 4 pixels in X and Y. Finally, a mask threshold of 0.9 was applied. Correlation consists of a process that calculates the displacement in the X (EW) and Y (NS) direction. In order to obtain the horizontal velocities of the RG (Vs), the resultant of the displacement vectors was applied in X and Y divided by the time between these acquisitions.

\[
Vs = \frac{\sqrt{(EW)^2 + (NS)^2}}{n} 
\]

Equation 4

Where ‘n’ represents the temporal space expressed in months of the image before and after the event, Vs is the velocity in m/month. The signal to noise ratio (SNR) values indicate the quality of the measurement and are in a numerical value from 0 to 1, while the SNR is closer to 1 indicates a better correlation (Moragues et al., 2018), in this study, SNR<0.7 were eliminated and a mean of the four evaluated periods was obtained.

RESULTS

Mapping, activity and altitude of rock glaciers: The results of local inventory of RG indicate that 187 RG are found in the study area. Of these, 45% are relict (85), ~34% inactive (63) and the ~21% remaining active (39) forms. They cover an area of 8.3 km\(^2\) (active 2.2 km\(^2\); inactive 3.07 km\(^2\) and relict 3.03 km\(^2\)). In term of size, most of them are classified as medium size (< 0.1 m\(^2\)). Most of RG are located between 4616 and 5551 m a.s.l, with an average altitude of 5081 m for active RG. Meanwhile, inactive and relict forms tend to be located in lower elevation (4961 m and 4851 m; Figure 2). In term of the altitudinal distribution, 69% of the RG are located between ~4600 – 5000 m a.s.l., while 25% are between 5000 – 5200 m a.s.l., the remaining RG are between 5200 – 5600 m a.s.l.

MAAT and paleo-MAAT of the rock glacier. The total RG are located in areas where the average MAAT is 0.9°C (between -1.7 to 2.9 °C). Meanwhile, active and inactive RG are closer to 0°C, 0.1° and 0.7° C respectively. On the other hand, relict RG glaciers can be present where MAAT is 1.4°C, value indicating that the temperature is farther away from the freezing zone. Of the total RG inventoried, 32 (18%) are located below the 0°C isotherm. Paleo-MAAT estimates of relict RG indicate that in past conditions RG were located in areas where the MAAT had values under the 0°C isotherm, it was -5°C on average (Figure 3).
WVE and paleo-WVE of the rock glacier: RG have thickness estimate ranges between 16 - 39 m. In the Cordillera Volcanica the WVE of the intact RG considering the minimum ice content (20%) is 28 million m$^3$, while considering the maximum ice content in RG (45%) we have a WVE of 64 million m$^3$ considering an area of 5.3 km$^2$ of RG.

The paleo-WVE was estimated based on relict RG, obtaining a paleo-WVE of 16 million m$^3$ and 35 million m$^3$ considering a paleo ice content of 20% and 45% respectively.

Horizontal velocities of the RG: The results of the horizontal velocities of active RG using optical images correlation indicate that the average velocity ($V_s$) is 4 cm/month, in addition, the horizontal velocities range is between 1 cm/month to 10 cm/month, it should be noted that the averages value correspond to the period of analysis during dry season, from 2015 to 2019.
Table 1. Area and WVE (million m$^3$) and paleo-WVE for RG in the Cordillera Volcanica

<table>
<thead>
<tr>
<th>Type rock glacier</th>
<th>Cordillera</th>
<th>Area (km$^2$)</th>
<th>WVE (million m$^3$) (20% ice)</th>
<th>WVE (million m$^3$) (45% ice)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact Volcanica</td>
<td>5.3</td>
<td>28</td>
<td>64</td>
<td></td>
</tr>
<tr>
<td>Relict Volcanica</td>
<td>3.0</td>
<td>16</td>
<td>35</td>
<td></td>
</tr>
</tbody>
</table>

The following figure shows the velocity rates found in active RG.

**Figure 4.** a) Horizontal velocities for active RG in Cordillera Chila. b) Horizontal velocities distribution and SNR in four RG. c) Velocity profile of the AB profile (red line on figure b).

**DISCUSSION**

In general, RG of the study area tend to be located above 4,600 m where MAAT are near 0°C and they cover a surface of 0.15% (8.3 km$^2$) of the basin. Similar surface values were reported in the Bolivian Cordillera (11 km$^2$; Rangecroft et al., 2014) and in Argentine Andes (9.66 km$^2$, Angillieri, 2009; 5.86 km$^2$ Perucca and Esper Angillieri, 2011). In comparison to other mountain regions, RG in the study are located in higher elevation than the RG located for example in the Bolivian, Chilean and Argentine Andes. (Brenning, 2005a; Janke et al., 2017; Perucca and Angillieri, 2011; Rangecroft et al., 2014). The minimum altitude at the front (MAF) limit of the
intact RG has been related to the area where the mountain permafrost begins or a good approximation of the lower limit of mountain permafrost (Barsch, 1996; Haeberli, 1985; Scotti et al., 2013), this limit can be set at 4693 m a.s.l., the altitude where the intact RG in the study area begin, this value is consistent with the value found in the Bolivian Andes 4700 m a.s.l. (Rangecroft et al., 2014), and much higher than that found in Chile and Argentina, for example, in the Central Chilean Andes, >3000 m a.s.l. (Brenning, 2005a), in the Aconcagua River Basin Chile, 3187 m a.s.l. (Janke et al., 2017) and in the Dry Andes of Argentina 4200 m a.s.l. (Perucca & Esper Angillieri, 2011), this indicates that the periglacial belt in the South American Andes increases when away from the Equator.

The largest number of reported RG correspond to the relict RG (45%) which occur over 4616 m a.s.l. with an altitude difference of 77 m in relation to the intact, which indicates that in the past environmental conditions were more favorable for their formation and shows a change in the elevation of the freezing line, overall, the average paleo-MAAT of the relict RG was found to be 6°C lower than the current MAAT of the relict RG. RG develop in conditions where the MAAT is lower than 0°C, however, under current conditions ~70% of intact RG prevail even under positive temperature conditions, this is according to what was found by Azócar and Brenning, (2010) in Chile and by Kinworthy (2016) in New Mexico USA, this shows that RG due to their detritic cover usually have a much slower response time to temperature increase (Martini et al., 2013) compared to that of white glaciers and that these geofoms are resilient to current climatic conditions and can subsist in the form of permafrost in these conditions for periods of up to a century (Gruber & Haeberli, 2007), however, it can be hypothesized that with the projected temperature increase of 2 - 5 °C at the end of the XXI century (Pastick et al., 2015) in the Andes, would decrease the number of active and inactive RG and increase the number of relict RG due to accelerated temperature change in a period of time.

It is worth noting that these approaches have been made on the basis of empirical formulas, without direct measurements of the thickness of the RG or the ice content in the RG, however, it is prudent to show these results as a first approximation. The WVE of intact RG was estimated at 28 - 64 million m³ per 5.3 km² considering 25% and 45% ice content respectively in these landforms, the content of WVE is less than that reported by Rangecroft et al. (2015) in the Bolivian Andes with a WVE range of 9 - 140 million m³ by 6.93 km², considering 40% and 60% ice content, is also lower than that reported in the Dry Andes Argentinos (Perucca and Esper Angillieri, 2011) which estimated 120 million m³ by 6.0 km² considering 50% ice content and much less than that found in the Andes Chilenos (Azócar & Brenning, 2010), 2370 million m³ by 147.5 km². In addition to this, the paleo-WVE of the relict RG was estimated, finding a maximum value (considering 45% ice content) of 35 million m³ by 3.0 km², this was estimated by the past active state of these landforms, a similar value 109 million m³ of paleo-WVE per 3.4 km² was estimated by Boccali et al., (2019) in the southeastern Alps.

As indicated in the previous chapter, the horizontal velocities found in this study have a mean of 4 cm/month, with a minimum and maximum of 1 - 10 cm/month respectively, in general, it has been observed that the highest velocities of active RG occur in the rooting zone of RG, area where debris and snow are brought in, therefore, it is an area where more dynamics are expected. Similar values to those found in this research were reported in Austria (Kellerer-Pirklbauer and Kaufmann, 2012) from 1 - 140 cm/month, Strel (2017) found velocities ranging from 1 - 6 cm/month with a maximum of 25 cm/month in northern Tien Shan in Central Asia, Wirz et al. (2016), reported velocities ranging from 2 - 54 cm/month in the Swiss Alps, extreme cases were reported by Corte which reports velocities of up to 100 m/year (Necsoiu et al., 2016). Figure 5 shows that most of
the active GR with the highest velocities (> 4 cm/month) are in areas where the MAAT is above 0.0°C. In situ monitoring of these landforms is also necessary to identify the real impact of temperature increase on these landforms.

Special care should be taken with intact RG (where areas of potential mountain permafrost are inferred) whose MAAT in which they are found is close to 0°C or higher, as a temperature increase could generate accelerated melting of ice contained in them and cause acceleration in permafrost flows, causing destabilization and collapse of the frozen soil in high mountain areas.

The authors recognize that more research is needed (particularly in situ measurements) to better understand the uncertainties in estimating ice content, the thickness of the RG and surface velocities; also, it is necessary to understand better the RG dynamics using different in situ techniques such as control with geodesic points and extensometers, also applying radar interferometry techniques to avoid loss of information in satellite images due to cloud coverage, finally, it is necessary to complement the paleo-MAAT estimation data of relict RG in order to carry out more accurate paleo-climatic studies.

CONCLUSIONS

In this research, 187 RG were identified covering an area of 8.3 km² between the 15.92° to 16.55° south latitude, where intact RG are the predominant geomorphological features. The minimum altitude of permafrost derived from the currently intact RG distribution (MAAT 0.46°C) was found above 4937 m a.s.l. and in the past was over 4616 m a.s.l., also, it was found that 70% of the intact RG prevail in conditions where the MAAT is positive. An estimation of WVE and paleo-WVE of RG in a part of Peru is presented for the first time, finding 28 – 64 million m³ per 5.3 km² RG area, in the past the relict RG had 35 million m³ per 3.0 km² of paleo-ice, these values not be considered as values to establish relationships between the hydrological contribution of the RG in the study area, however, it is important to accompany these results with fieldwork to establish relationships of probable water supply of RG in this mountain of extinct glaciers. Finally, it was found that RG has velocities from 1 – 10 cm/month and presents a direct relationship with the MAAT of the area where RG is located, this could indicate that these landforms are in an acceleration by the temperature increase.
ACKNOWLEDGMENTS

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Mountain Permafrost in the Tropical Andes of Peru: The 0°C Isotherm as a Potential Indicator

Hairo León1; Katy Medina2; Edwin Loarte3; Guillermo Azócar4; Pablo Iribarren5; and Christian Huggel6

1Faculty of Environmental Sciences, Santiago Antúnez de Mayolo National Univ.; National Institute for Research on Glaciers and Mountain Ecosystems, Huaraz, Peru (corresponding author). E-mail: hleond@unasam.edu.pe
2National Institute for Research on Glaciers and Mountain Ecosystems, Huaraz, Peru. E-mail: kmedina@inaigem.gob.pe
3National Institute for Research on Glaciers and Mountain Ecosystems, Huaraz, Peru. E-mail: eloarte@inaigem.gob.pe
4Atacama Ambiente Consultores, Chile. E-mail: gazocar@atacamamb.com
5Instituto de Ciencias de la Tierra, Faculty of Sciences, Valdivia, Chile. E-mail: pablo.iribarren@uach.cl
6Dept. of Geography, Univ. of Zurich, Zurich, Switzerland. E-mail: christian.huggel@geo.uzh.ch

ABSTRACT

In the tropical Andes of Peru very little is known about the occurrence and extent of mountain permafrost. Only recently systematic studies have been carried out on the high elevation sites of the mountain ranges (cordilleras). In the framework of the first pioneering studies, and with the objective to improve the understanding of characteristics of mountain permafrost and rock glaciers, we analyze how mountain permafrost in the Peruvian Andes is correlated with the altitude of the 0°C isotherm (ZIA). Climate change has generated an increase in air temperature and in the ZIA in the past decades. These temperature changes could lead to impact the state of the mountain permafrost. In this research, we focus on two mountain regions: The Cordillera Central (CC) and the Cordillera Volcánica (CV), the first located in the central zone and the second in the south zone of Peru. The study used air temperature data from 20 weather stations (2002–2016) to calculate the mean annual air temperature (MAAT), interpolated using a multiple linear regression model (MLRM) and digital elevation model (MERIT DEM). Occurrence and extent of 46 intact rock glaciers (IRG) and the global model of permafrost (Permafrost Zonation Index) were used to validate the results. The MAAT of CC has a minimum value around -4.1°C ($R^2 = 0.8$) and a ZIA average of ~5152 m a.s.l. None of the IRGs are located above the ZIA. The MAAT of CV has a minimum value around of -5.5°C ($R^2 = 0.8$), a ZIA average of ~4861 m a.s.l., and 60% of the IRGs are located above of the ZIA. The results show a greater variation of the position of the ZIA in CC in comparison to CV, which could indicate a possible degradation of mountain permafrost in these mountain ranges.

1. INTRODUCCIÓN

Mountain regions create conditions for the existence of permafrost (Arenson et al., 2011). Dominant factors such as elevation and its derivatives (aspect and slope), incidence of solar radiation, snow cover and the ground conditions influence on the ground temperatures (Gruber and Haeberli, 2009). In the Peruvian Tropical Andes, permafrost is expected to be found on all slopes above the ZIA and some studies (for example, Gorbunov, 1978; Guodong and Dramis, 1992; Haeberli et al., 1993) state that the mountain permafrost is at a minimum altitude of
approximately 5000 m a.s.l. In these mountain ranges, periglacial features such as intact rock glaciers (IRG) are geomorphological indicators of the occurrence of permafrost and therefore they can be used to approximate the regional lower limit of the mountain permafrost belt (Delaloye and Echelard, 2020). Based on this, it was estimated that the occurrence of isolated permafrost in a small sector of the cordilleras of southern Peru (Chachani Volcano, Arequipa) is above 5050 m a.s.l., between 5250 to 5420 m a.s.l. there is discontinuous permafrost, and at higher elevations permafrost tends to be continuous (Andrés et al., 2011; Yoshikawa et al., 2020). However, these geomorphological approaches are usually complex and require fieldwork. The ZIA is an indicator that has been used to estimate areas that are at negative temperatures levels and may host mountain permafrost (Carey et al., 2012; Haeberli et al., 2017), as well as, the changes that may occur in the permafrost due to its location in or outside the ZIA (Azócar et al., 2017). According Bradley et al. (2009) the ZIA is located within 4800 m a.s.l. ± 300 m for regions ranging from 20°N to 20°S that corresponds also to the lower limit of the permafrost estimated for the mountainous regions of Peru (~5050 m a.s.l), therefore, ZIA could indicate the possible distribution of mountain permafrost in Peru. The variation of ZIA would also be used to evaluate the existing changes in these areas that potentially contain permafrost, since currently and as a result of climate change, there has been an increase of ~0.2°C/decade in the global average temperature during the last 30 years (IPCC, 2019).

This increase is also evident in mountainous areas. In the Alps the increase is of 0.5°C/decade (EEA, 2009) and in the Andes it is 0.11°C/decade (Vuille et al., 2015). According to these trends, the ZIA will increase its altitude as time passes, in some projections it is estimated that the current altitude of the ZIA for tropical mountain ranges would experience an increase of 230 m by the end of the 21st century (Schauwecker et al., 2017). The relationship between the ZIA and the lower limit of the permafrost (delimited using intact rock glaciers) will help us to estimate how it behaves as a function of time.

Finally, this research used ZIA data from two pilot mountain ranges in the Peruvian Andes to infer mountain permafrost. As well as studying its variation to identify areas where the permafrost is possibly changing as a result of the increase in temperature.

2. DATA AND METHODS

2.1. Study area

The study area comprises the central and southern sector of the tropical Andes of Peru, covering the areas of the Cordillera Central (CC) in the center and the Cordillera Volcánica (CV) in the southern part of Peru. The CC cover an area of 11,574 km² and is located between the parallels 13.0° and 11.4° S between the meridians 76.7° and 75.2° W. CV has an area of influence of 5805 km² and is located between 15.6° and 16.9° S and 70.5° and 71.8° W. In terms of altitudinal distribution, CC has an average altitude of 4,162 m a.s.l. with the highest point located at Pariacaca glacier (5750 m a.s.l.) Meanwhile, CV has an average altitude of 4368 m a.s.l., with the highest site located at Chachani Volcano (6057 m a.s.l; Figure 1).

Rainfall is concentrated in the northeastern area of the CC, where there is an average annual rainfall greater than 800 mm during the austral summer (Aybar et al., 2019; INAIIGEM, 2018). A semi-frigid climate predominates, with an average annual temperature around 6 °C above 4500 m a.s.l. (Vicente-serrano et al., 2017). The climatic conditions in the CV are characterized by having precipitation that is concentrated in the eastern flank of the mountain range with annual mean values of 400 mm, with temperatures below 5 °C above 4500 m a.s.l. which favors the presence
of moderate frosts during the dry autumn (INAIGEM, 2018; Vicente-serrano et al., 2017). The climatic characteristics, high altitude, and increase in latitude allows greater possibilities of finding permafrost in the CV compared to the CC.

Figure 1. Location map of the study area using the MERIT digital elevation model (90 m). Elevation ranges are shown in the legend; yellow dots indicate the location of intact rock glaciers.

2.2. Data

2.2.1. Air temperature data

The maximum and minimum monthly mean temperature data (Tmax and Tmin) were provided by the National Service of Meteorology and Hydrology of Peru (SENAMHI-Peru). The Tmax and Tmin are calculated by the average of the maximum and minimum daily air temperatures records in a given month. Then, Mean Annual Air Temperatures (MAATs) were calculated for 20 weather stations located above 3000 m a.s.l. for a fifteen-year meteorological period (2002 to 2016; Table 1).

2.2.2. Rock glacier inventory

The regional inventory of rock glaciers was prepared using Microsoft’s Bing Maps imagery accessible through QGIS 3.12. Rock glaciers were identified following the criteria of classification proposed by Humlum (1982), Barsch (1996) and Pereyra et al. (2010). Frequently, active and inactive rock glaciers (grouped together as intact forms in this study) have a steep front with visual
unstable rocks with ridges and furrows on their surface that are indicative of their present deformation (Azócar et al., 2017). In contrast, an irregular and collapsed surface due to thawing of ice commonly indicates that rock glacier is in its fossil or relict condition (Colucci et al., 2016; Rangecroft et al., 2015).

Table 1. Location of the weather stations and number of years between 2002 and 2016 with observations.

<table>
<thead>
<tr>
<th>Weather station name</th>
<th>Cordillera</th>
<th>Region</th>
<th>Record n years</th>
<th>Longitude</th>
<th>Latitude</th>
<th>Elevation (m)</th>
<th>MAAT (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Huancalpi</td>
<td>CC</td>
<td>Huancavelica</td>
<td>15</td>
<td>-75.237</td>
<td>-12.539</td>
<td>3450</td>
<td>9.0</td>
</tr>
<tr>
<td>Jauja</td>
<td>CC</td>
<td>Junin</td>
<td>15</td>
<td>-75.487</td>
<td>-11.787</td>
<td>3378</td>
<td>11.3</td>
</tr>
<tr>
<td>San Juan de Jarpa</td>
<td>CC</td>
<td>Junin</td>
<td>15</td>
<td>-75.432</td>
<td>-12.125</td>
<td>3600</td>
<td>9.2</td>
</tr>
<tr>
<td>La Oroya</td>
<td>CC</td>
<td>Junin</td>
<td>15</td>
<td>-75.959</td>
<td>-11.569</td>
<td>3910</td>
<td>8.5</td>
</tr>
<tr>
<td>Huayao</td>
<td>CC</td>
<td>Junin</td>
<td>15</td>
<td>-75.338</td>
<td>-12.038</td>
<td>3360</td>
<td>12.2</td>
</tr>
<tr>
<td>Laive</td>
<td>CC</td>
<td>Junin</td>
<td>15</td>
<td>-75.355</td>
<td>-12.252</td>
<td>3860</td>
<td>6.5</td>
</tr>
<tr>
<td>Huarochiri</td>
<td>CC</td>
<td>Lima</td>
<td>15</td>
<td>-76.234</td>
<td>-12.139</td>
<td>3120</td>
<td>9.8</td>
</tr>
<tr>
<td>San Lázaro de Escomarca</td>
<td>CC</td>
<td>Lima</td>
<td>12</td>
<td>-76.352</td>
<td>-12.181</td>
<td>3758</td>
<td>7.2</td>
</tr>
<tr>
<td>Carania</td>
<td>CC</td>
<td>Lima</td>
<td>14</td>
<td>-75.872</td>
<td>-12.344</td>
<td>3820</td>
<td>7.6</td>
</tr>
<tr>
<td>Vilca</td>
<td>CC</td>
<td>Lima</td>
<td>14</td>
<td>-75.826</td>
<td>-12.115</td>
<td>3832</td>
<td>7.2</td>
</tr>
<tr>
<td>Patahuasi</td>
<td>CV</td>
<td>Arequipa</td>
<td>7</td>
<td>-71.415</td>
<td>-16.055</td>
<td>4035</td>
<td>6.4</td>
</tr>
<tr>
<td>Pampa de Arrieros</td>
<td>CV</td>
<td>Arequipa</td>
<td>15</td>
<td>-71.589</td>
<td>-16.063</td>
<td>3715</td>
<td>8.6</td>
</tr>
<tr>
<td>Pillones</td>
<td>CV</td>
<td>Arequipa</td>
<td>14</td>
<td>-71.213</td>
<td>-15.979</td>
<td>4455</td>
<td>3.5</td>
</tr>
<tr>
<td>Las Salinas</td>
<td>CV</td>
<td>Arequipa</td>
<td>14</td>
<td>-71.148</td>
<td>-16.318</td>
<td>4378</td>
<td>4.5</td>
</tr>
<tr>
<td>Imata</td>
<td>CV</td>
<td>Arequipa</td>
<td>15</td>
<td>-71.091</td>
<td>-15.843</td>
<td>4475</td>
<td>3.7</td>
</tr>
<tr>
<td>El Frayle</td>
<td>CV</td>
<td>Arequipa</td>
<td>15</td>
<td>-71.189</td>
<td>-16.155</td>
<td>4131</td>
<td>8.1</td>
</tr>
<tr>
<td>Huanca</td>
<td>CV</td>
<td>Arequipa</td>
<td>15</td>
<td>-71.881</td>
<td>-16.035</td>
<td>3065</td>
<td>12.7</td>
</tr>
<tr>
<td>Puquina</td>
<td>CV</td>
<td>Moquegua</td>
<td>15</td>
<td>-71.169</td>
<td>-16.627</td>
<td>3284</td>
<td>12.7</td>
</tr>
<tr>
<td>Ubinas</td>
<td>CV</td>
<td>Moquegua</td>
<td>15</td>
<td>-70.854</td>
<td>-16.372</td>
<td>3380</td>
<td>10.5</td>
</tr>
<tr>
<td>Crucero Alto</td>
<td>CV</td>
<td>Puno</td>
<td>15</td>
<td>-70.912</td>
<td>-15.764</td>
<td>4521</td>
<td>4.1</td>
</tr>
</tbody>
</table>

For this research, we grouped active and inactive rock glaciers as intact forms, due to uncertainties in their classification (Azócar et al., 2017). Rock glaciers were digitized as point marks located at the end of the rock glacier front using an on-screen map scale of 1:5000.

2.2.3. Digital elevation model

The MERIT (Multiple Error Elimination Enhanced Terrain) digital elevation model was developed by removing multiple error components (absolute bias, stripe noise, speckle noise, and tree height bias) from the existing spaceborne DEMs (SRTM3 v2.1 and AW3D-30m v1). It represents the terrain elevations at a 3 arc sec resolution (~90m at the equator), and covers land areas between 90N-60S, referenced to EGM96 geoid (Yamazaki et al., 2017).

2.3. Methodology

2.3.1. Regionalization of Mean Annual Air Temperature (MAAT)

To estimate the ZIA, the MAAT was first calculated using data from meteorological stations over a 15-year time period (2002-2016) and additional data was obtained from the PISCOt grid.
(Peruvian Interpolation data of the SENAMHI’s Climatological and hydrological Observations) for different points within the whole study area (Aybar et al., 2019). The MAAT for a particular year was calculated as the arithmetic average of the monthly mean temperatures for that year. To have a better representation of the mountain climate, all meteorological stations are located above 3000 m a.s.l. and are far away from the coast to avoid the moderating effect of the ocean on the air temperature (Hiebl et al., 2009, Azócar, 2013).

The temperature data underwent quality control based on the standard deviation of the data, in order to eliminate errors in the time series caused by changes in the temperature sensor, station location or poor data capture (Hunziker et al., 2018). The elevation (meters), latitude (coordinate in degrees) and longitude (coordinate in degrees) were used, taken from the MERIT Digital Elevation Model (MERITDEM) with a resolution of 90 m and a vertical precision of around 4 m (Yamazaki et al., 2017). Multiple Linear Regression models (MLRM), unlike ordinary linear regression models, can explain the dependency between observations and estimated values (Alzate et al., 2018; Azócar et al., 2017).

To predict the spatial variability of MAAT, the annual residual variation was also considered. To calculate the MAAT at any location X with known elevation, latitude and longitude within the study region, the following model equation was used:

$$\text{MAAT}(x) = a_1 \text{elevation}(x) + a_2 \text{latitude}(x) + a_3 \text{longitude}(x) + \text{residual}$$  \hspace{1cm} (10)

The overall fit of the MM was evaluated by examining the mean absolute error (MAE) in °C, coefficient of determination ($R^2$) and root-mean-square error (RMSE). This model was applied in the R programming language, using the Regnie package (Rauthe et al., 2013).

2.3.2. Estimation and validation of ZIA

We used the 0 °C MAAT threshold to illustrate the extent of conditions suitable for permafrost under present day conditions (2002-2016) similar to Rangecroft et al. (2016). For this purpose, we extracted the MAAT pixel values that are at a temperature ≤ 0 °C. In this way new value were assigned to each pixel: 0 being a pixel with a temperature higher than 0 °C and 1 pixels with lower temperature. Thus, images with binary values were obtained, and all those areas that had temperatures higher than 0 °C were eliminated. Finally, the remaining areas were displayed in a GIS, to obtain the altitude of the ZIA.

This estimate was discussed and compared with the lower limit of permafrost assuming that above this threshold discontinuous permafrost could be found. This limit is given by the minimum altitude of active and inactive rock glaciers (Barsch, 1996). In addition, the lower limit of fossil rock glaciers is used as an indicator of past permafrost conditions (Kerschner, 1978). It is assumed that changes in the altitudinal distribution reflect changes in climatic conditions, especially mean annual air temperatures (Lambiel and Reynard, 2001). Also, a qualitative analysis was performed between the ZIA and the global permafrost zonation developed by Gruber (2012).

3. RESULTS AND DISCUSSION

3.1. Mean Annual Air Temperature

The temperature distribution model indicated a current ZIA (2002-2016 mean) at ~5152 m a.s.l. in CC, and at ~4861 m a.s.l. in CV. The result cannot be directly compared with other studies, because they were carried out in places close to our study settings and on a different time scale (SENAMHI, 2016; ZIA~5100 m at Cordillera Huaytapallana near CC, Schauwecker et al., 2017;
The rate of temperature decrease with altitude obtained in this study (~0.55 °C per 100 m) is reasonably close to the average temperature decrease in the free atmosphere (~0.6 °C per 100 m; Beniston, 2006). The minimum temperatures identified in CC reach values of -4.1 °C with a standard deviation of 3.2 °C, while in CV the lowest temperature corresponds to -5.5 °C with a standard deviation of 2.2 °C (Figure 2).

Figure 2. The maps illustrate current annual mean air temperature (MAAT) areas for CC and CV using a suitability threshold of 0 °C and -2 °C as an approximation of the extent of mountain permafrost. Rock glaciers distribution are shown as red and green dots.

The precision (MAE) of the MAAT distribution model is between 1.1-1.4 °C for CC and 0.9-1.3 °C for CV highlighting uncertainties in this key predictor of permafrost preference in this region.

The analysis carried out to the values of the MAAT model yielded R² values close to 1 (0.8) for both mountain ranges, showing a good correlation between the observed and the estimated values. However, the values obtained in the RMSE (1.7 °C and 1.2 °C, for CC and CV respectively) indicate that there may be uncertainties in these predictions.

The estimated MAAT values agree with MAAT models used as explanatory variable to estimate the distribution of permafrost in the Chilean Andes (RMSE <.0.96 °C in Azócar et al., 2017) and on a global scale (RMSE≈1 °C in Gruber, 2012). These comparisons can only provide a very general guidance. The lack of air temperature records and the spatial distribution of weather station could increase the residual standard errors and affect possible interpretation on permafrost.
distribution. Therefore, MAAT obtained in the study could be warmer than reality and permafrost distribution may be underestimated at higher elevations.

3.2. Rock Glacier Inventory

85 rock glaciers were inventoried in the study area (Figure 2). Of these, 44 were classified as intact and 41 as fossils or relicts forms. The number of rock glaciers differs by 74 between CC and CV. Furthermore, the distribution of rock glaciers by dynamic status within each mountain range was uneven, ranging from 44% intact forms in CV to 8% intact forms in CC.

Figure 3. The figure shows the aspect as a function of the minimum altitude where intact rock glaciers are located. This is the lower limit of current permafrost based on an inventory of rock glaciers in CC and CV.

Rock glaciers have predominant orientations towards the south, similar finding have been indicated by Loarte et al., (2019) that indicate that rock glaciers located above 4,700 tend to have west-southwest aspects and rock glacier located above 5200 m a.s.l have mainly eastern orientations (Figure 3). Intact rock glaciers with north aspects were not identified possibly due to higher radiation rates on slopes with this orientation. On the other hand, we observed an uneven distribution of rock glaciers in each zone due to climate conditions. In CC, rock glaciers are scarce in comparison to white glaciers and wet conditions are more predominant (Marcer et al., 2017). Meanwhile, in CV, climate conditions are drier and rock glaciers are abundant thanks to the absence of white glaciers, a lithology prone to rock erosion, and the production of talus (Matsuoka & Ikeda, 2001). The existence of abundant fossil or relict forms in both mountain ranges suggests that favorable conditions existed for generalized permafrost in late glacial cold events. These findings were also found by previous studies (Guodong & Dramis, 1992; Lambiel & Reynard, 2001; Deluigi et al., 2017). Most intact rock glaciers occur in a MAAT below the ZIA, 60% (24) are below this threshold in CV, while in CC all the intact rock glaciers are below the ZIA limit (Figure 4). Brenning (2005) and Rangecroft et al., (2015) also reported active rock glaciers with positive MAAT in mountain ranges such as CC.
Figure 4. Proportion of intact and fossil rock glaciers located below (+MAAT) and above (−MAAT) the isothermal altitude of 0 °C MAAT (ZIA) for CC and CV.

Figure 5. Annual ZIA (from 2002-2016) for CC and CV estimated with data from SENAMHI meteorological stations using the MAAT model: the x-shaped symbols represent the median, the boxes represent the 25th and 75th percentiles, the whiskers represent standard deviations and outliers are represented by points.

3.3. Calculated and Variation of Zero Isotherm Altitude (ZIA)

Figure 5 shows the mean elevation of the ZIA over a 15-year period. The CV shows a decline in the ZIA during the years 2002-2008, also reported by SENAMHI (2016), influenced by local effects that occurred in the climate of this area. However, as of 2010, an altitude peak is shown that follows a positive trend during the rest of the years for both CC and CV (Harris et al., 2000). The higher altitude found in CC with respect to CV is explained by the climatic difference between the two zones, CV is further from the equator, where drier and colder climates predominate.
Schauwecker et al., 2014). The highest ZIA in CC is in agreement with that observed by Bradley et al. (2009) for tropical areas.

In general, the air temperature and estimation of the ZIA is affected by several factors (for example, katabatic winds, aspect and insolation) that can influence the local climate conditions. Therefore, the regional estimate of the ZIA presented here is associated with a bias that increases significantly when temperature extrapolation is carried out for periods with few weather stations available (Schauwecker et al., 2017). However, because the stations used in this study have similar information periods in the 15 years evaluated, it allows us to identify the variations experienced by the ZIA during this period. In CC there have been variations in the ZIA between the 5000 at 5500 m asl, while in CV there are changes between 4800 and 5000 m a.s.l. These changes evidenced in the ZIA could be related to past events of rock falls and slope collapses registered in the Peruvian Andes (Evans & Clague, 1994). For example, in Arequipa (area of the CV) areas potentially exposed to sliding slopes and rock falls have been identified due to some triggering factor, such as temperature. Rock collapses could impact and travel considerable distances in the valleys (Vela, 2018).

3.4. ZIA and Lower Limit of Permafrost

The results indicate that the lower limit of mountain permafrost based on rock glacier distribution could be located in CV at 4655 m and at 4767 m in CC (Figure 3). Meanwhile, the estimated position of ZIA is at 4861 m a.s.l. and 5152 m a.s.l. for CV and CC respectively (±4m, error inherited from DEM). The difference between the two altitudes is because intact rock glaciers can exist below the ZIA due to local favorable conditions (i.e. lower radiation, geomorphological position and sources of debris material).

Similar findings have been found for the European Alps (Marcer et al., 2017) and for Chilean, Argentine and Bolivian Andes (Azócar et al., 2017; Esper Angillieri, 2017; Rangecroft et al., 2015). When we comparison the results of global Permafrost Zonation Index (PZI) values (Gruber, 2012) with the ZIA obtained in this study the PZI values greater than 0.4 are found below the ZIA or in colder conditions.

As found in CV, the highest probability of finding permafrost is above 4712 m a.s.l. and near the ZIA (4861 m a.s.l.), values that are close to the one obtained by Andrés et al., (2011), which shows us a lower limit of 4900 m a.s.l. for CV. Gorbunov (1978) also estimated that the permafrost could be above 5000 m a.s.l. in tropical mountain ranges.

4. CONCLUSION

The results show that statistical models can be used as first approaches to estimate MAAT and ZIA position, however, the lack of measurements over 5000 m a.s.l. and the effects of some local factors (temperature circulation and complex topography) could affect the prediction of temperature in higher elevations. In general, the linear multiple regression model is useful for general approaches to the estimation of MAAT and ZIA. However, for more suitable statistical approaches, it is highly recommended to use Mixed Effect Models due to these models account for the non-independence of data.

The aspect of rock glaciers affects the position of the lower limit of permafrost, which generates the altitudinal differences with the ZIA. However, it is observed that the majority of IRG are within the ZIA limit in CV, while the opposite occurs in CC. Therefore, the ZIA can be used as an indirect indicator of the initial estimated distribution of mountain permafrost. In addition, the
altitudinal difference between the ZIA and the lower limit of the permafrost, can generate changes in the local landscape, a problem that will increase as time progresses, since a positive trend was observed in the ZIA, which means that the ZIA will retreat to higher ground, exposing new areas to landslides.

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Consensus-Based Rock Glacier Inventorying in the Torngat Mountains, Northern Labrador

Robert G. Way, Ph.D.1; Yifeng Wang2; Alexandre R. B eveingto n3; Philip P. Bonnaventure, Ph.D.4; Jake R. Burton5; Emma Davis, Ph.D.6; Madeleine C. Garibaldi7; Caitlin M. Lapalme8; Rosamond Tutton9; and Mishèle A. E. Wehbe10

1Asst. Prof., Northern Environmental Geoscience Laboratory, Dept. of Geography and Planning, Queen’s Univ., Kingston, ON (corresponding author). E-mail: robert.way@queensu.ca
2Northern Environmental Geoscience Laboratory, Dept. of Geography and Planning, Queen’s Univ. E-mail: yifeng wang@queensu.ca
3British Columbia Ministry of Forests, Lands, Natural Resource Operations and Rural Development. E-mail: alexandre.bevington@gov.bc.ca
4Bonnaventure Lab for Permafrost Science, Dept. of Geography and Environment, Univ. of Lethbridge. E-mail: philip.bonnaventure@uleth.ca
5Parks Canada. E-mail: jake.burton@canada.ca
6School of Environment, Resources and Sustainability, Environment, Univ. of Waterloo. E-mail: emma.davis@uwaterloo.ca
7Bonnaventure Lab for Permafrost Science, Dept. of Geography and Environment, Univ. of Lethbridge. E-mail: madeleine.garibaldi@uleth.ca
8Independent Researcher. E-mail: caitlin.lapalme@gmail.com
9Northern Environmental Geoscience Laboratory, Dept. of Geography and Planning, Queen’s Univ. E-mail: 18rjt4@queensu.ca
10Bonnaventure Lab for Permafrost Science, Dept. of Geography and Environment, Univ. of Lethbridge. E-mail: mishelle.wehbe@uleth.ca

ABSTRACT

The Torngat Mountains of northern Labrador are an Arctic cordilleran mountain range located at the southern limit of the Canadian Arctic. Sparse observations of periglacial landforms including rock glaciers and ice-cored moraines imply that permafrost may be widespread but limited in situ information is available for the region. In this study, we provide the first comprehensive feature inventory of intact rock glaciers in the Torngat Mountains of northeast Canada. Prospective features were identified by a team of eight independent mappers using high-resolution satellite imagery. The initial inventory was re-assessed via consensus-building and review stages, resulting in a final inventory of 608 rock glaciers. Rock glaciers were distributed from ~58°N to ~60°N and were primarily concentrated in the northern end of the Torngat Mountains National Park with southern clusters located in high relief coastal mountains near 59°N. The use of a large mapping team and the multistage consensus-based approach maximized feature inclusion and reduced misinterpretation of other features (e.g., debris-covered glaciers, ice-cored moraines, and talus couloirs) for rock glaciers. Our results show the efficacy of consensus-based landform identification for geomorphological mapping in the heterogeneous environment of the Torngat Mountains.

INTRODUCTION

Rock glaciers are key indicators of contemporary and past permafrost conditions (Humlum 1998) and are amongst the most recognizable permafrost landforms in mountain environments.
Recent studies show that rock glaciers exhibit complicated kinematic responses to climate change (Eriksen et al. 2018; Scotti et al. 2017; Sorg et al. 2015) but can be associated with geocryological hazards (Marcer et al. 2020) and can distribute large quantities of freshwater into local ecosystems (Jones et al. 2019b). Further, rock glaciers are of significant ecological importance as they provide long-term cold habitat for local biodiversity (Brighenti et al. 2020). Numerous studies have used rock glaciers to inform interpretations of past or present permafrost distributions in areas where in situ observations of permafrost conditions are limited (Fernandes et al. 2017; Lilleøren et al. 2013; Marcer et al. 2017; Sattler et al. 2016; Schmid et al. 2015).

Debate remains over the utility of rock glaciers as permafrost indicators because they can extend well-beyond the lower elevation limits of discontinuous permafrost (Jones et al. 2019b). Interpreting geocryologically significant landforms like rock glaciers is also challenged by their similar morphology to other landforms found in periglacial environments, including protalus ramparts and periglacial creep phenomena. Inherent uncertainty in the classification of rock glaciers is also relevant for glacier-connected rock glaciers which are difficult to differentiate from debris-covered glacier termini, ice-cored moraines, and ice-cored debris fields (Jones et al. 2019a; Monnier and Kinnard 2015). There are presently no definitive visual criteria for identifying rock glaciers from imagery, though standards are in development (IPA Action Group: Rock glacier inventories and kinematics 2020). Remote sensing studies by Paul et al. (2013) and Brardinoni et al. (2019) demonstrated that mapped outlines of debris-covered glaciers and rock glaciers in mountainous environments could vary considerably between mappers. These studies highlight that in addition to kinematic information (e.g., interferometry, image matching; Liu et al. 2013; Villarroel et al. 2018), the inclusion of multiple operators may increase the rate of geomorphological landform detection in complex terrain.

In the Torngat Mountains of northern Labrador, modelling studies suggest that permafrost may be widespread (Way and Lewkowicz 2016, 2018), but field observations of local permafrost conditions are lacking. Remote sensing and field-based glacier inventorying studies identified ice-cored moraines and rock glaciers at several locations within the boundary of Torngat Mountains National Park (Figure 1; Way et al., 2014, 2015). A preliminary, unpublished rock glacier inventory created by a small team of mappers using SPOT5 satellite imagery identified 201 features in the region (Way 2017). These observations, albeit limited in geographic scope, emphasize the need for a better understanding of the distribution of permafrost landforms in the Torngat Mountains. In this study, we aim to generate a rock glacier feature inventory for the entire Torngat Mountains region; and to evaluate the efficacy of a multistage, consensus-based rock glacier identification process in a remote topographically complex glacial-periglacial environment.

STUDY AREA

The Torngat Mountains of northern Labrador form the southern limit of Canadian Arctic ecosystems (Ponomarenko and McLennan 2010) and include the tallest mountains in eastern mainland Canada (~1650 m a.s.l. central Selamiut Range). The regional climate is classified as polar tundra (Kottek et al. 2006; Way et al. 2017) but has experienced rapid warming over the past several decades (Barrette et al. 2020; Davis et al. 2020), causing widespread expansion of upright vegetation (Davis et al. 2020; Fraser et al. 2011, 2012) and rapid melt of local glaciers (Barrand et al. 2017; Barrette et al. 2020). Regional geomorphology has been influenced by differential erosion caused by contrasting thermal regimes of basal ice (Staiger et al. 2005). Post-glacial depositional environments include extensive deposits of marine sediments below the marine limit (~50 m a.s.l.)
but variable). Distance from the Labrador Sea is a key driver of glacier elevation and distribution due to local cooling and widespread high relief in the coastal mountains (Way et al. 2014).

Little information exists on permafrost conditions in the Torngat Mountains aside from archaeological investigations along the coastline and outer islands that revealed near-surface permafrost in marine deposits (Butler 2011; Jordan 1980). Prior excavation of low-elevation gelification lobes and soil sampling on mountain summits provided inconsistent evidence of permafrost presence or absence (Evans and Rogerson 1988; Hendershot 1985; de Vernal et al. 1983). Permafrost was encountered during the establishment of a Parks Canada monitoring station in the central Torngat Mountains in 2010, but the installed data loggers have yet to refreeze following installation (Dr. Darroch Whitaker, Parks Canada, personal communication). Deep active layers in coarse surficial deposits and exposed bedrock found throughout the Torngat Mountains make permafrost detection difficult in the absence of geomorphic indicators. An unpublished rock glacier inventory derived from SPOT5 satellite imagery (see Way 2017) estimated that 201 rock glaciers were present in the Torngat Mountains, though these data were not sufficiently quality-controlled to form a full inventory. Davis et al. (2020) present the first field-based observations of permafrost in the Torngat Mountains and suggest that near-surface permafrost may not be as widespread as previously thought.

METHODS

Data Sources: All feature identification activities used Maxar (Vivid) optical satellite imagery accessed via the ArcGIS Online platform (1.2 m ground sampling distance; ~8.5 m absolute spatial accuracy). Images ranged from summer 2010 to summer 2019 and minimized seasonal snow and cloud cover. Topographic data for the Torngat Mountains were extracted from the Canadian Digital Surface Model (CDSM) produced by Natural Resources Canada (~20 m spatial resolution) and were used to determine the elevational distribution of the identified rock glacier features. The CDSM was derived from reprocessed radar interferometry data collected by the Shuttle Radar Topography Mission (Endeavour mission in 2000). Gridded air temperature data covering the Torngat Mountains (30 m resolution) were generated from the CDSM following Way et al. (2017; Figure 1a) and were used to determine the 2013-2016 mean annual air temperature at the central position of each of the identified rock glaciers in the region.

Feature Identification: The workflow followed a multistage, consensus-based feature identification process that included eight independent mappers and two independent review team members (Figure 2). Mappers ranged in academic background but most had northern field experience and held bachelor’s degrees or above (two BAs or BScs, four MScs, two PhDs). Two mappers held specific expertise in satellite remote sensing, four had previously studied permafrost science (graduate level or above), and one member had visited the Torngat Mountains for permafrost investigations in 2016. The review team was composed of two members with graduate-level permafrost science field experience in Labrador. One review team member (lead author) had completed five field seasons in the Torngat Mountains and completed regional remote sensing-based glacier and rock glacier inventories (e.g., Way et al. 2014, 2015; Way 2017).
Figure 1. (a) Spatial distribution of mean annual air temperatures (2013-2016) across northern Nunatsiavut and eastern Nunavik. Bounding box (black outline) shows the extent of rock glacier inventorying activities in this study. (b) Photograph of a formerly glacier-connected rock glacier. (c) Photograph of a rock glacier tongue. (d) Photograph of a talus-connected rock glacier.

Figure 2. Conceptual diagram of the multistage consensus-building rock glacier inventorying approach used in this study.

Prior to feature identification, each mapper was provided rock glacier identification training materials, video tutorials, and relevant publications. This included the International Permafrost Association Rock Glacier working group documentation on rock glacier inventorying, in which rock glaciers were defined as ‘debris landforms generated by a former or current creep of frozen ground, detectable in the landscape with the following morphologies: front, lateral margins and optionally ridge-and-furrow surface topography’ (IPA Action Group Rock glacier inventories and
kinematics 2020). Mappers were instructed to identify prospective rock glaciers following the
geomorphological approach. The prospective rock glacier inventory sought to only include
features that were intact and non-relict, with geomorphological evidence of recent or former flow
required for each feature (e.g., steep front, lateral margins, minimal vegetation).

**Stage 1**: The eight mappers were first asked to identify intact rock glaciers throughout the
Torngat Mountains and adjacent regions (bounding box in Figure 1a). Prospective features were
assigned a unique ID and a ranking from 1 (low confidence) to 3 (high confidence) to describe the
mapper’s confidence in their interpretation of each feature. Following the feature identification
phase of stage 1, the review team compiled all eight feature inventories into a single aggregated
inventory using the centre of each geomorphological feature identified by individual mappers. The
mapper identification count and the confidence levels were summed for each unique feature, and
the features then underwent an initial quality-control check by the review team to identify those
that clearly did not meet the geomorphological criteria (e.g., folded bedrock, outcrops). The
aggregated, quality-controlled stage 1 inventory was then compared by the review team to the
earlier unpublished SPOT5 inventory of rock glaciers. Any features that had been previously
identified in the SPOT5 inventory but had not been identified by the mapping team in stage 1 were
included with the final stage 1 inventory for further review in stage 2.

**Stage 2**: In stage 2, the aggregated database of stage 1 features was sent back to the mapping
team for reanalysis. In this consensus-building stage, mappers were asked to indicate whether each
feature was a rock glacier and to again provide accompanying confidence levels. To minimize
bias, mappers were only provided a group-level confidence metric calculated as: confidence (%) =
\((\text{sum of indicated confidence levels} / \text{maximum confidence level based on the number of}
\text{identifying mappers}) \times 100\). Thus, 1/8 members identifying a feature as a rock glacier with a high
confidence \((3 / 3 = 100\%)\) would have a higher confidence than 3/8 members identifying it with
low confidence \((3 / 9 = 33\%)\). This information was provided to guide mappers towards consensus-
building without applying significant pressure on the mapper to conform. Upon completion of the
stage 2 reanalysis by the mapping team, results were compiled by the review team.

**Stage 3**: Stage 3 was completed by the two-person review team. Phase 1 of this process was
an initial quality-control of all features with 100% mapper agreement (i.e., 8/8 mappers rating a
feature as a non-rock glacier or rock glacier). Other prospective rock glacier features that were
missed in the initial inventorying process but were identified during review of nearby features were
flagged by review team members and added to phase 2 for consideration. Phase 2 was a reanalysis
of all remaining features (i.e., 1/8 to 7/8 mappers rating a feature as a rock glacier). Review team
members independently evaluated each stage 2 feature using ArcGIS Online and ArcScene Online,
while also taking mapper agreement and confidence into account. Features with conflicting
classifications by the review team members were subset and subject to another independent round
of evaluation. Features with persistent disagreement were assessed on a feature-by-feature basis
by review team members in a joint review session using ArcScene Online. The resulting feature
inventory was the final Torngat Mountains rock glacier inventory.

**RESULTS**

**Evaluation of Feature Identification**: In stage 1, mapping team members identified a total of
1648 features, of which 932 were unique. The number of features identified by individual mappers
varied from 52 to 367 with a mean of 206 features. During the quality control, 57 features that
corresponded to clearly folded bedrock or outcrops were removed, resulting in an aggregated
inventory of 875 unique features. All the removed features had been identified by only 1/8
mappers, likely reflecting mapper error. Only four of the 875 features were identified by 8/8 mappers, though 100 features were identified by at least 4/8 mappers. A total of 552 features were found by only one mapper. Comparison with the unpublished SPOT5 inventory (Way 2017) showed that 164 of the 201 features (81%) in the earlier inventory were identified by at least 1/8 mappers in stage 1. The remaining 37 features were combined with the stage 1 aggregated inventory, resulting in a combined inventory of 912 features. In stage 2, 8/8 mappers identified 168 features as rock glaciers and 116 features as non-rock glaciers. At least 4/8 mappers identified 574 features as rock glaciers, with 294 identified by at least 7/8 mappers.

During the stage 3 review process, review team member agreement was high (88%), with only 85 features evaluated differently by the two independent reviewers. The phase 2 reanalysis resulted in 35/85 features with remaining disagreement between the reviewers, but this disagreement was rectified through a consensus-building joint review meeting. Following the stage 3 review, 548 of the original 912 features from the stage 2 database were classified as rock glaciers, and an additional 60 out of 63 prospective rock glaciers identified by the review team were added to the final inventory. The cumulative number of mappers identifying a feature as a rock glacier in stage 2 was strongly associated with it being categorized as a rock glacier at stage 3 (Figure 3). For example, the likelihood of a feature being correctly identified as a rock glacier (stage 3 output) in stage 2 increased from ~6% to 99% for 0/8 to 8/8 mapper agreement, respectively. Comparison with the unpublished SPOT5 inventory (Way 2017) showed inclusion of 88% (176/201) of SPOT5 rock glaciers as rock glaciers in the final inventory.

![Figure 3](image.jpg)

**Figure 3.** The number of features identified as non-rock glaciers (grey bars) and as rock glaciers (blue bars) by review team members in stage 3, stratified by the number of mappers identifying a feature as a rock glacier in stage 2. Also shown is the probability of a feature being classified as a rock glacier in the final inventory according to the number of mappers classifying it as such in stage 2 (dashed line).

**Rock Glacier Inventory Results:** Six-hundred and eight rock glaciers were identified in the
Torngat Mountains out of 975 prospective features (932 unique features identified by the mapping team and 37 from the SPOT5 inventory). Of the 608 rock glaciers, 90% originated from the stage 1 and 2 inventories whereas 60 features (10%) were identified by the review team in stage 3. Rock glaciers were identified as far south as 58.1°N and as far north as 60.3°N, with more features in the northern half of the study area (mean latitude of 59.3°N; Figure 4a). Large clusters of rock glaciers in the southern half of the study area were typically located in fretted coastal mountains or in the central Selamiut Range (Figure 4a). The mean elevation of rock glacier centres was ~430 m a.s.l. though elevations ranged from 0 m a.s.l. to 1115 m a.s.l.. The elevation of rock glaciers was inversely related to latitude (-158 m per °N), but the association was weak (r = 0.28, p > 0.01). Mean annual air temperatures (2013-2016) estimated for rock glacier positions ranged from -9.7°C to -3.6°C with a mean value of -6.2°C. Similar to elevation, the distribution of features according to mean annual air temperature was skewed, though to a lesser degree.

**DISCUSSION**

**Rock Glacier Inventorying Process:** The multi-stage rock glacier inventory resulted in a significantly larger feature database than those originally compiled by individual mappers (stage 1). Individual mapper inventories ranged from 52 to 367 prospective features in stage 1, whereas the final stage 3 dataset contained 608 rock glaciers. Underestimation of features in stage 1 highlights the challenges of mapping rock glaciers using optical imagery but also reflects individual sampling practices (e.g., mapper fatigue, no gridding). Notably, the earlier SPOT5 inventory was fairly accurate (176 of 201 landforms correctly identified), but it also underestimated the number of detectable features in the region by 432. This may be a trade-off between SPOT5’s lower spatial resolution but higher spectral resolution which favoured detection of larger, more developed features over smaller features. For this study, the use of a large mapping team facilitated the effective detection of a large number of features and likely reduced the potential rate of omitted features from the final inventory. However, 10% of the final rock glacier inventory was not identified until the final review stage, indicating the possibility that other features could be missing from the inventory. The multi-stage feature identification process also increased consensus amongst mappers from n = 4 rock glaciers with 100% agreement in stage 1 to n = 168 in stage 2. Further, the number of mappers identifying a feature as a rock glacier (or a non-rock glacier) at stage 2 was a strong predictor of feature inclusion at later stages (Figure 3). These results suggest that crowd-sourced landform identification and interpretation can improve geomorphological mapping and provide a means of assessing inventory errors in remote areas.

The review team seldomly (n=8) rejected features that were identified with high agreement (> 85%) amongst mapping team members. The limited cases of disagreement occurred when debris-covered glaciers or ice-cored moraines were misinterpreted by mappers as glacier-derived rock glaciers due to their obvious physical similarities and flow structures. These environments were challenging to interpret as they reflect the transition from glacial processes to the possible initiation of periglacial processes (e.g., creep and rock glacier initiation). Interpretation of these features was informed by past field experience working on debris-covered glaciers in the region, including field visits to some of the misclassified features (e.g., Way et al. 2014, 2015). This also highlights the value of including individuals with local knowledge/expertise on the review team. The second largest challenge was differentiating between talus-derived rock glaciers, talus couloirs, and protalus ramparts, particularly given that talus couloirs often have a lobate appearance with indications of downslope creep. Many of these features may in fact be rock glaciers, but corresponding kinematic data would be required to confirm this hypothesis. For similar reasons, it
was deemed implausible to include relict rock glaciers in the inventory because of difficulty distinguishing these features from historical moraines and protalus ramparts found throughout the Torngat Mountains (André 1986).

Figure 4. (a) Spatial distribution of inventoried features including rock glaciers (dark blue) and non-rock glaciers (light blue). Inset shows a kernel density map (unit: rock glaciers per km²). (b) Example of glacier-connected rock glacier unit; (c) Example of talus-connected rock glacier complex; (d) Avalanche debris chute mistaken for rock glacier in stage 1. Images visualized with ArcGIS Online.

**Rock Glaciers and Permafrost in the Torngat Mountains:** The inventory presented in this study suggests that rock glaciers are widespread throughout the Torngat Mountains, with increasing frequency towards the north and regional southern clusters along high relief coastal mountains. The weak decreasing trend in rock glacier elevation with latitude can be attributed to widespread permafrost distribution but could also be a result of terrain factors including changes to slope suitability for rock glacier formation. However, abundant talus-connected rock glaciers, and the lower overall elevation in the northern end of the Torngat Mountains could be indicative of more widespread near-permafrost conditions compared to regions farther south where features were often proximal to glaciers or former glacier cirques. Strictly speaking, the presence of
prominent Little Ice Age moraines in many of the foregrounds of the small glaciers of the Torngat Mountains may also be indicative of the preservation of interior glacial ice (Way et al. 2015). Together, these observations suggest that mountain permafrost is widespread in the Torngat Mountains, but given the relatively low mean elevation of rock glaciers (~430 m a.s.l.), it is difficult to determine whether the regional limits of permafrost distribution follow clear elevational boundaries.

CONCLUSION

In this study, we presented the first comprehensive feature inventory of rock glaciers in the Torngat Mountains of northern Labrador. Using a multistage consensus-based inventorying approach, we identified a total of 608 rock glaciers distributed across the Torngat Mountains (from 58.1°N to 60.3°N and a mean latitude of 59.3°N). The mean elevation of rock glacier centres was ~430 m a.s.l., ranging from 0 m a.s.l. to 1115 m a.s.l. Consensus-building steps were shown to dramatically improve the overall quality of the feature inventory while multiple (n = 8) mappers ensured that most of the prospective features (~90%) were identified. The results of this study highlight the value in involving larger teams in multistage, crowd-sourced remote sensing feature identification in complex environments like the Torngat Mountains. Detailed field investigations and kinematic information would be required to refine the existing Torngat Mountain rock glacier inventory.

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Preliminary Interpretations from a Landslide Inventory in Interior Alaska

Jaimy A. Schwarber¹, Margaret M. Darrow, Ph.D., P.E., M.ASCE², Ronald P. Daanen, Ph.D.³; and De Anne S. P. Stevens⁴

¹Dept. of Civil, Geological, and Environmental Engineering, Univ. of Alaska Fairbanks, Fairbanks, AK, USA. E-mail: jaschwarber2@alaska.edu
²Dept. of Civil, Geological, and Environmental Engineering, Univ. of Alaska Fairbanks, Fairbanks, AK, USA. E-mail: mmdarrow@alaska.edu
³Alaska Division of Geological & Geophysical Surveys, Fairbanks, AK, USA. E-mail: ronald.daanen@alaska.gov
⁴Alaska Division of Geological & Geophysical Surveys, Fairbanks, AK, USA. E-mail: deanne.stevens@alaska.gov

ABSTRACT

Landslides are geologic hazards that threaten human life, infrastructure, and property. To mitigate these threats, a landslide inventory map must first be developed. We present preliminary interpretations of the first comprehensive landslide inventory in the Fairbanks North Star Borough (FNSB), Interior Alaska. The inventory was developed using light detection and ranging (LiDAR) digital elevation models (DEMs), and validated with field checks of landslides accessible on public lands along the road system. The inventory provides a landslide spatial distribution that can be correlated to types of soil and/or bedrock, slope, aspect, and permafrost distribution. We can determine relative age of landslides using morphology, vegetation, and cross-cutting relationships with infrastructure. We provide landslide examples of: 1) different ages, 2) mechanisms of movement, and 3) morphology; and explore potential triggers of the prehistoric landslides.

INTRODUCTION

Landslides are geologic hazards that pose threats to human life, infrastructure, and property. The first step in mitigating these threats is to develop a landslide inventory map for landslide-susceptible areas. Until this study, no published landslide inventories existed for Alaska. Our preliminary interpretations of this first landslide inventory in the Fairbanks North Star Borough (FNSB) within Interior Alaska (Figure 1) were developed with particular attention applied to permafrost-related characteristics.

Recently acquired and published Light Detection and Ranging (LiDAR) data (FNSB 2019) of part of the FNSB made comprehensive mapping possible for the first time since most of these landslide features are difficult to discern in optical imagery due to vegetation cover. The LiDAR dataset includes the communities of Fairbanks, North Pole, Fort Wainwright, and Eielson Air Force Base, as well as several important transportation corridors.

METHODS

The LiDAR data used in this study has two resolutions, with the smaller, higher-resolution area centered on Fairbanks (Table 1). The landslide inventory was developed using LiDAR digital elevation models (DEMs) at both resolutions, and validated with field checks of landslides accessible on public lands along the road system.
Figure 1. Location of the project area; legend applies to all sub-figures. The irregular hillshade polygon shown in (b) is the available high-quality LiDAR of part of FNSB (FNSB 2019), and the hillshade in (c) is a combined image that adds a background hillshade from ArcticDEM data (Porter et al., 2018) with a gray “no data” section.

Table 1. Resolution quality and area of extent for each type of LiDAR data (OCM Partners 2020).

<table>
<thead>
<tr>
<th>Type</th>
<th>Resolution</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quality Level (QL) 1</td>
<td>1.5 (0.46)</td>
<td>248,555 (1,006)</td>
</tr>
<tr>
<td>QL2</td>
<td>3.0 (0.91)</td>
<td>1,186,822 (4,803)</td>
</tr>
</tbody>
</table>
We generally followed the landslide mapping protocol of Slaughter et al. (2017) for Washington State, making modifications as necessary to account for permafrost and periglacial conditions. We developed DEM-derived products such as hillshades with different azimuth and altitude angles, slope maps, and contour lines to use in the mapping process, which relied on visual interpretation of the data layers and manual digitization of landslide features in a geographic information system (GIS) environment. Following the GIS-based mapping, we field verified 48 landslides and landslide complexes within FNSB during summer 2020 (see Figure 1 for distribution). The goals of the field checks were to evaluate the landslide extents (scars, flanks, and toe deposits), internal morphology (e.g., ponds, closed depressions, cracks, fresh earth), landslide material type (composition), movement type and age, and vegetation indicators (e.g., cracked or leaning trees, types of vegetation).

RESULTS AND DISCUSSION

Landslide Distribution and Ages: We identified nearly 1,400 landslides within the FNSB LiDAR data coverage area. Landslides of varying ages occur in both rock and soil (Figure 2). Most landslides are prehistoric (i.e., predating the first significant anthropomorphic surface changes that occurred approximately 100 years ago in Interior Alaska) based on geomorphic expression (Figures 2a and 2b). There are also historic (Figures 2c and 2d) and currently active landslides (Figures 2e and 2f), as verified by relative age with infrastructure and field observations. The mapped landslides occur on slopes that vary from 5 to 50 degrees, with an average slope angle of 19 degrees.

Landslide geomorphology varies due to both age and material. The surfaces of younger landslides tend to have a greater roughness than older ones. Features like internal scarps and cracks may no longer be apparent on older slides. Much of the area within the FNSB is blanketed by Quaternary loess deposits, ranging in thicknesses from 30 to 150 m (Péwé 1975). Many of the landslides in loess have either faint or no discernable headscarps or flanks and little surface roughness, but the toe deposits are still preserved in the valley bottoms (Figure 2a). In contrast, the landslides in bedrock typically have clear and steep headscarps, and obvious extents and toe deposits. Many of these slides occur in quartz muscovite schist, the predominant bedrock type in the greater Fairbanks area (Newberry et al. 1996). Mechanisms of movement vary depending on the material type. Landslides that occurred primarily within loess moved as flows (Figure 2a). By contrast, those occurring in bedrock exhibit rotational (Figure 2b) or translational movement (Figure 2d).

Based on morphologic expression and vegetation indicators, we conclude that most identified landslides are prehistoric. A few of the field-checked landslides are historic, and others exhibit on-going movement as evidenced by leaning and split trees, and fresh cracks and scarps. One active landslide (Figure 2f) has morphology and surface features similar to frozen debris lobes in the Brooks Range of Alaska, although it differs in internal composition and origin. During our field check of this feature, we observed schist bedrock within the headscarp, and that the body and toe consisted of silty and rocky debris. Indicators of on-going movement included cracked and leaning trees, surface cracks, and active burial of vegetation at the toe. Black spruce and moss covered the distal portion of the landslide, suggesting the presence of permafrost. Its on-going movement despite frozen subsurface conditions suggests movement may be tied to temperature and water pressure, as was demonstrated for one of the frozen debris lobes (Darrow et al. 2017).
Figure 2. Mapped extents of field verified landslides of various ages and material types: prehistoric in (a) soil and (b) rock; historic in (c) soil and (d) rock; modern in (e) soil and (f) rock. Background is LiDAR-derived slope map (FNSB 2019).
Figure 3. Comparison of landslides identified using LiDAR-derived slope maps (FNSB, 2019) in (a), (c), and (e), versus optical imagery (GeoNorth 2019) in (b), (d), and (f).
Most landslides are difficult to discern using only optical images, but become more easily identified with LiDAR (Figure 3). Once a landslide is identified, certain features may be highlighted by vegetation type (Figures 3b, 3d). For example, headscarp delineation of the landslide shown in Figure 3b is aided by the sharp break in vegetation. The body and toe extent in Figure 3d can be inferred as there is a vegetation type change that outlines the toe deposit. The headscarp area is also slightly discernable. In some cases, the landslide remains obscured and is not easily distinguished from areas covered with similar vegetation (Figure 3f).

Figure 4. Example of a landslide complex along the Richardson Highway, MP 296. The ages include (1) prehistoric, (2) historic, and (3) currently moving. Background is LiDAR-derived slope map (FNSB 2019).

**Landslide Triggers:** There are several potential triggers for landslides within the FNSB. This region is currently within the discontinuous permafrost zone, where 50-90% of the ground contains permafrost (Jorgenson et al. 2008). Depending on ice content, thawing permafrost contributes water to the soil, reducing the effective stress and potentially resulting in landslides (McRoberts and Morgenstern, 1974). Climate proxies indicated periods of higher than present temperatures in Alaska during the early Holocene, particularly the Holocene Thermal Maximum (HTM) (Kaufman...
et al. 2004). If climatic conditions were favorable for permafrost thaw, this may have been a trigger for landslides in permafrost-affected slopes.

Triggers independent of permafrost include heavy rain events, earthquakes, and river erosion of landslide toes. For example, rockslides were observed between Fairbanks and Nenana along the Tanana River after the 1947 M7.3 earthquake, which had an epicenter near the entrance of the Nenana River into the Tanana Valley (St. Amand 1948). Landslides also were noted along the Richardson Highway and the Alaska Railroad as a result of the 1948 event (St. Amand 1948). Fairbanks has a history of notable seismic events, such as the M7.9 earthquake along the Denali Fault in 2002 and the M5.2 event near Minto Flats in 2014 (AEC 2020a, b). Older but significant earthquake events in the region include a 7.4M event in 1912, 6.3M and 6.5M events in 1929, and a 7.3M event in 1937 (Péwé 1982). Based on these examples, it is likely that future earthquake events of similar magnitude within Interior Alaska study area could trigger additional landslides.

Along the Richardson Highway, several bedrock landslides were caused by the Tanana River eroding into the base of the slope (Figure 4). These areas demonstrate multiple periods of movement, resulting in a landslide complex. For example, the historic landslide at Richardson Highway Milepost 296 was a reactivated section of an ancient landslide (Figure 4) (Landslide Technology 2000). This particular slide was successfully mitigated and its displacement of the Richardson Highway was stopped; however, on-going movement is still occurring in the lower portion as the Tanana River continues to erode the toe of the slope.

CONCLUSION

Our preliminary mapping and analysis using LiDAR data within the FNSB have identified nearly 1,400 landslides that vary widely in age, composition, and mechanisms of movement. Future work will include analysis of landslide spatial distribution as related to types of soil and/or bedrock, slope angle and aspect, drainage, and permafrost distribution, as well as radiometric dating of samples from selected landslides to determine absolute age.

The final published map will directly benefit the FNSB, as community planners can use the landslide inventory to make informed land management decisions, including safer road alignments and parcel developments. Since seismic events may trigger landslides, emergency managers can incorporate landslides as additional components of multiple-disaster scenarios for better planning. We also hope to raise public awareness of this geologic hazard. The final map is expected to be published by the end of 2021.

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Performance of Bridges in Cold Regions with Sliding Seismic Isolation Bearings

Jesika Rahman¹; A. H. M. Muntasir Billah, Ph.D., M.ASCE²; and Asif Iqbal, Ph.D., M.ASCE³

¹Graduate Research Assistant, Dept. of Civil Engineering, Lakehead Univ., Thunder Bay, ON, Canada. E-mail: rahmanj@lakeheadu.ca
²Assistant Professor, Dept. of Civil Engineering, Lakehead Univ., Thunder Bay, ON, Canada (corresponding author). E-mail: muntasir.billah@lakeheadu.ca
³Assistant Professor, Dept. of Civil Engineering, Univ. of Northern British Columbia, Prince George, BC, Canada. E-mail: asif.iqbal@unbc.ca

ABSTRACT

Effects of extreme temperature on highway bridges in cold regions seismically isolated with sliding type bearings are investigated. The critical factor in consideration is the change in the performance of isolation bearings with significant variation in temperature between seasons. The sliding bearing behavior is characterized by the friction coefficient of the sliding surfaces. The friction coefficient during a seismic motion varies with the sliding velocity and temperature at the sliding surface. Tests associated with past applications have indicated a marked increase in the value of friction coefficient resulting in higher stiffness of bearings at very cold temperatures. The effects of change in bearing stiffness on the seismic performance of the bridge in general and the substructure in particular are demonstrated here. This study aims to capture the change in bearing response and subsequently the overall structural response considering a temperature variation between −40°C and +40°C. Response parameters considered for this study are the base shear in the piers, the acceleration of the bridge deck, maximum and residual displacement of the isolation bearings, as well as the energy dissipation capacity. The response parameters are compared for individual ground motions as well as the mean and coefficient of variation (COV). It is observed that the higher bearing stiffness at extreme cold temperature leads to additional forces on the substructure which reduces the margin of safety and hence should be considered carefully in design.

INTRODUCTION

The significance of improving the structural performance that holds the prime responsibility for the safety of the occupants during hazards is not a new concept. Following a series of natural events namely the Northridge earthquake in 1994, Kobe earthquake in 1995 and Chi-Chi earthquake in 1999, severe concerns were raised among the engineers and researchers about the integrity of structures especially bridges during a major seismic event. Among several techniques, seismic isolation is by far the most effective device for protection against such natural disasters. The main function of the seismic isolators is to separate the substructure from its superstructure and shift the natural period of the structure away from the predominant earthquake period. Researchers have developed and extensively studied different isolation devices such as steel reinforced elastomeric bearings (SREB), fiber reinforced elastomeric bearings (FREB), lead rubber bearings (LRB), and the friction pendulum system (FPS) (Robinson 1982, Tsopelas et al. 1996, Warn and Whittaker 2004, Alam et al. 2012, Van Engelen et al. 2016) for the prevention of severe damage to bridges during an earthquake.

The benefits of using base isolation techniques are notably appreciated in the structural engineering community and the FPS, in particular, has gained attention due to its consistency in
design properties over a broad range of temperatures (Zayas et al. 1990, Zayas and Low 1999). The FPS is a sliding bearing system that restores the structure’s serviceability upon being subjected to earthquake forces through its concave frictional surface (Wang et al. 1998). Based on the principle of a pendulum, the FPS restores the structure by dissipating energy through relative movement between the housing plate and the concave surface. The effectiveness of FPS in bridges has been studied by many researchers (Wang et al. 1998). Eröz and DesRoches (2013) investigated the effect of FPS and LRB as isolation bearings in a multi-span continuous concrete girder bridge under vertical ground motions. They compared the results of both the isolators and reported their satisfactory performance.

Despite the advantages of these base isolation devices in bridges, several concerns among the researchers are prevalent regarding the performance of the isolators when subjected to natural phenomena such as seismic events having a long duration (Hassan and Billah 2020) and, as for this paper, the effect of temperatures. It is well documented that the stiffness and energy dissipation capacity of the isolators tend to increase at low temperatures. For instance, the performance of fiber reinforced elastomeric bearing (FREB) in bridges at low temperature was studied by Toopchi-Nezhad et al. (2019) where they found that both damping and stiffness increased at low temperatures. On the other hand, Wang et al. (2019) reported that low temperatures play an important role in the maximum isolator displacement in the bridge system installed with lead rubber bearings. Further studies (Zhipping 2016, Mendez-Galindo et al. 2017) done on the performance of LRB isolated bridges revealed that the energy dissipation capacity of the isolator decreased and both the bridge pier and isolator showed an increased probability of exceeding damage states during subfreezing temperatures. The authors also indicated the importance of considering the elastomer crystallization during the design of such bridges.

Another study by Zheng et al. (2019) investigated the seismic response of an isolated bridge equipped with shape memory alloy based FPS isolator under low temperature. They found that the isolation system can increase the energy dissipation capacity at 20°C temperature while exhibited limited energy dissipation at low temperatures. They reported temperature sensitivity of the sliding friction coefficient as well as the recentering ability of the system. Previous studies revealed that low temperatures increased the sliding friction coefficient significantly (Dolce et al. 2005). However, the effect of low temperature on seismic performance of bridges isolated with FPS bearings has not been studied thoroughly. The objective of this study, therefore, is to investigate the change in bearing response and subsequently the overall structural response of a base-isolated bridge considering a temperature variation between −40°C (winter) and +40°C (summer). Non-linear finite element analysis is carried out to evaluate the behavior of the isolated bridge. Response parameters considered for this study are the base shear in the piers, the acceleration of the bridge deck, maximum and residual displacement of the isolation bearings as well as the energy dissipation capacity. Comparison of response parameters under individual ground motions as well as the mean and coefficient of variation (COV) in response under summer and winter conditions show the effect of cold temperature on base-isolated bridges.

DESCRIPTION OF THE BRIDGE

The bridge considered in this study is a three-span continuous concrete I-girder bridge located in Montreal, QC, Canada and is classified as a major route bridge according to the Canadian Highway Bridge Design Code (CHBDC) (CSA 2019). The bridge is equipped with FPS bearings at the pier and abutment locations. The bridge superstructure is supported on two reinforced concrete two-column bents and conventional seat type abutments. The FPS bearings allow bridge
movement under service loading (temperature) and earthquake loading. The details of the bridge are shown in Figure 1. The two-column bents have unequal heights of 15m and 11.5m. Each column bent has two circular piers connected by a rectangular concrete pier cap. All columns in one bent are considered to have the same height. The column spacing within a bent is 7.5m. The 1500mm circular reinforced concrete piers are reinforced with 34-25M (diameter 25.2mm) longitudinal rebars and 20M (diameter 19.5mm) bars as spiral reinforcement at a spacing of 75mm in the plastic hinge region and 150mm outside the plastic hinge. The pier columns are supported on a rectangular concrete pile cap which is connected to two rows of circular concrete piles. Conventional seat type abutments are supported on steel HP piles at bridge ends.

FINITE ELEMENT MODELING

The fiber-based non-linear finite element software SeismoStruct 2020 (Seismosoft 2020) is used to develop the detailed 3D model of the bridge considered in this study. The I-girders are modeled using elastic beam element assuming that both deck and girder remain elastic when subjected to seismic forces. The bent cap and columns are modeled using force-based fiber elements. The unconfined and confined concrete is modeled using the constitutive relationship developed by Mander et al. (1988) while the steel reinforcement is modeled following the Menegotto-Pinto (1973) stress-strain relationship. The response of the seat type abutments in transverse and longitudinal directions are defined as bilinear springs following the recommendations by CHBDC (CSA 2019) and Caltrans (2013). According to CHBDC (CSA 2019), pile foundation can be modeled either using six uncoupled compliance springs at the pile cap level or a series of springs distributed along the length of the pile for representing soil-pile interaction. In this study, six uncoupled compliance springs were used for modeling the pile foundations. The properties of these springs were calculated following the pile model developed by Novak (1974). However, the effect of depth of frozen soil layer on the soil-structure interaction has not been considered in this study. Zero-length non-linear spring is used to model the gap between the abutment and deck as recommended by Muthukumar and DesRoches (2006).

When subjected to extreme cold temperature, the concrete and steel material properties also change as reported in past research (Sritharan et al. 2007). Under subfreezing temperature both concrete and reinforcing steel experience an increase in strength without changing the deformability. This study also considered the change in reinforcing steel and concrete material properties under winter conditions which have been calculated using the following relationships.

![Figure 1. Details of the bridge studied.](image-url)
proposed by Montejo et al. (2008) and Browne and Bamforth (1981), respectively:

\[ f_c(T) = 1.1 f_c(20°C) \quad -25°C > T > -40°C \]  
\[ f_c'(T) = f_c'(20°C) - T_w/12 \quad 0°C > T > -40°C \]  

The material properties used in the FE model at summer and winter conditions are summarized in Table 1. The FPS is modeled using the zero-length Bearing 2 element available in SeismoStruct. The friction coefficient is calculated using the friction model proposed by Constantinou et al. (1999). The model requires the definition of friction coefficient at fast and slow sliding velocities which are used to calculate the velocity dependent friction coefficient using the following exponential relationship:

\[ \mu = \mu_{fast} - (\mu_{fast} - \mu_{slow})e^{-rv} \]  

where, \( \mu \) is the friction coefficient, \( \mu_{fast} \) and \( \mu_{slow} \) represent the friction coefficients at fast and slow sliding velocities, \( v \) is the reference sliding velocity = 150mm/s, and \( r \) is a rate parameter which is calculated using the following equation proposed by Dolce et al. (2005).

\[ r = \frac{1}{v_{ref}} \ln \left( \frac{\mu_{fast} - \mu_{slow}}{\mu_{fast} - \mu_{ref}} \right) \]  

Table 1. Material properties for Concrete and Steel

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Summer</th>
<th>Winter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>Compressive Strength (MPa)</td>
<td>35</td>
<td>38.3</td>
</tr>
<tr>
<td></td>
<td>Corresponding strain</td>
<td>0.0030</td>
<td>0.0033</td>
</tr>
<tr>
<td></td>
<td>Tensile strength (MPa)</td>
<td>3.33</td>
<td>3.33</td>
</tr>
<tr>
<td></td>
<td>Elastic modulus (GPa)</td>
<td>26.6</td>
<td>27.8</td>
</tr>
<tr>
<td></td>
<td>Elastic modulus (GPa)</td>
<td>194</td>
<td>194</td>
</tr>
<tr>
<td></td>
<td>Yield stress (MPa)</td>
<td>468</td>
<td>515</td>
</tr>
<tr>
<td>Steel</td>
<td>Ultimate stress (MPa)</td>
<td>692</td>
<td>692</td>
</tr>
<tr>
<td></td>
<td>Ultimate strain</td>
<td>0.14</td>
<td>0.14</td>
</tr>
<tr>
<td></td>
<td>Plateau strain</td>
<td>0.016</td>
<td>0.016</td>
</tr>
</tbody>
</table>

The properties of the FPS isolation bearings considered for summer and winter conditions are provided in Table 2. The FPS bearings were designed for a vertical load of 12500kN. The values listed in Table 2 are obtained from tests conducted on prototype FPS bearings under summer and winter temperatures. Due to confidentiality, more details about bearing test results are not provided in this paper.

Table 2. Properties of the FPS at summer and winter temperatures

<table>
<thead>
<tr>
<th>Input Parameter in SeismoStruct</th>
<th>Summer (+40°C)</th>
<th>Winter (−40°C)</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective stiffness, ( K_{eff} )</td>
<td>9.73</td>
<td>12.06</td>
<td>kN/mm</td>
</tr>
<tr>
<td>Friction coefficient at fast velocities</td>
<td>0.062</td>
<td>0.097</td>
<td>--</td>
</tr>
<tr>
<td>Friction coefficient at slow velocities</td>
<td>0.043</td>
<td>0.063</td>
<td>--</td>
</tr>
<tr>
<td>Curvature radii of friction pendulum</td>
<td>2235</td>
<td>2235</td>
<td>mm</td>
</tr>
</tbody>
</table>
The accuracy of SeismoStruct in the prediction of bridge component and system response under different loading conditions has been confirmed by different researchers. To validate the accuracy of the adopted FPS modeling technique, results from the experimental testing on FPS bearings are considered here. Constantinou et al. (2007) tested an FPS bearing under a constant vertical pressure of 30.8MPa while the bearing was subjected to lateral displacement cycles at a frequency of 0.6 Hz. The comparison of the experimental force-deformation relationship and the numerically obtained ones are shown in Figure 2. The normalized force-displacement response under first three loading cycles are illustrated here which shows that the simulated response matches very closely with the experimental response.

![Figure 2. Force-displacement comparison between experimental and simulated response of the FPS](image)

**SELECTION OF GROUND MOTIONS**

To assess the seismic response of the FPS isolated bridges under summer and winter conditions, 100 ground motion records that resemble the bridge site location are selected. These records are selected from the Pacific Earthquake Engineering Research (PEER) Center Ground Motion Database. Following the works by Naumoski et al. (1988), each record is chosen such that the $A/V$ ratio is between 1.70 and 2.63 that characterize the ground motions in Eastern Canada, i.e. Montreal. The characteristics of the selected records are listed in Table 3. The earthquake excitations are applied to the bridge in both the longitudinal and transverse directions. Real accelerograms are chosen that represented the actual seismic conditions of the location the bridge under study is situated. The records of the selected ground motions are presented in Table 3. The selected ground motions are spectrally matched to the target response spectra using SeismoMatch. Matching was done within the period range of interest, which was $0.15T_1–2T_1$ as suggested in CHBDC (CSA 2019). Figure 3 illustrates the acceleration response spectrum for the chosen ground motions.

**RESULTS AND DISCUSSION**

To evaluate the performance of the FPS isolation bearings under summer and winter temperatures, the isolated bridges are analyzed using the selected ground motions. Using nonlinear time history analysis, the performance of the two bridges is compared in terms of the base shear in piers, deck acceleration, maximum and residual displacement of the bearings and FPS energy dissipation capacity.
Table 3. Records of selected ground motions

<table>
<thead>
<tr>
<th>Record ID</th>
<th>Earthquake Name</th>
<th>Year</th>
<th>Station Name</th>
<th>Magnitude</th>
<th>PGA (g)</th>
<th>PGV (m/s)</th>
<th>A/V</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQ#1</td>
<td>Helena_ Montana</td>
<td>1935</td>
<td>Carroll College</td>
<td>6.0</td>
<td>0.146</td>
<td>0.072</td>
<td>2.03</td>
</tr>
<tr>
<td>EQ#2</td>
<td>San Francisco</td>
<td>1957</td>
<td>Golden Gate Park</td>
<td>5.3</td>
<td>0.105</td>
<td>0.046</td>
<td>2.28</td>
</tr>
<tr>
<td>EQ#3</td>
<td>Parkfield</td>
<td>1966</td>
<td>Cholame - Shandon Array #5</td>
<td>6.2</td>
<td>0.434</td>
<td>0.255</td>
<td>1.7</td>
</tr>
<tr>
<td>EQ#4</td>
<td>Lytle Creek</td>
<td>1970</td>
<td>Wrightwood - 6074 Park Dr</td>
<td>5.3</td>
<td>0.198</td>
<td>0.096</td>
<td>2.06</td>
</tr>
<tr>
<td>EQ#5</td>
<td>San Fernando</td>
<td>1971</td>
<td>Lake Hughes #4</td>
<td>6.6</td>
<td>0.146</td>
<td>0.085</td>
<td>1.72</td>
</tr>
<tr>
<td>EQ#6</td>
<td>San Fernando</td>
<td>1971</td>
<td>Pacoima Dam (upper left abut)</td>
<td>6.6</td>
<td>1.075</td>
<td>0.577</td>
<td>1.86</td>
</tr>
<tr>
<td>EQ#7</td>
<td>Oroville-01</td>
<td>1975</td>
<td>Oroville Seismograph Station</td>
<td>5.9</td>
<td>0.084</td>
<td>0.044</td>
<td>1.91</td>
</tr>
<tr>
<td>EQ#8</td>
<td>Nahanni_ Canada</td>
<td>1985</td>
<td>Site 1</td>
<td>6.8</td>
<td>1.101</td>
<td>0.462</td>
<td>2.38</td>
</tr>
<tr>
<td>EQ#9</td>
<td>Parkfield-02_ CA</td>
<td>2004</td>
<td>PARKFIELD - TEMBLOR</td>
<td>6.0</td>
<td>0.269</td>
<td>0.145</td>
<td>1.86</td>
</tr>
<tr>
<td>EQ#10</td>
<td>Montenegro_ Yugoslavia</td>
<td>1979</td>
<td>Ulcinj - Hotel Albatros</td>
<td>7.1</td>
<td>0.042</td>
<td>0.016</td>
<td>2.63</td>
</tr>
</tbody>
</table>

Figure 3. Scaled response spectra of the chosen ground motions

**Base shear in piers:** The base shear demand in bridge piers resulting from seismic excitations is reduced when isolation bearings are installed. This results from the decoupling of superstructure from the substructure that allows using smaller substructure section sizes. Figure 4(a) presents the variation in base shear of the shorter pier at summer and winter temperatures for the ground motion EQ #10. Here EQ #10 is selected for comparison since it has the largest A/V ratio as shown in Table 3. It is seen that the maximum base shear in the shorter pier is 817.4 kN in summer which is increased by 5% in winter. The reason for this increment can be due to the fact that the stiffness of the FPS, as well as the strength of both concrete and steel, increased at the low temperature.
Also, as evident from Figure 4(b), the base shear during winter temperatures are higher than that obtained during summer for all the ground motions studied except for EQ #5 and EQ #8.

Figure 4. (a) Base shear-time history response of the shorter pier under EQ #10, (b) Comparison of base shear between summer and winter conditions.

Deck acceleration: One of the primary objectives of using FPS is to reduce the deck acceleration, which is proportional to the earthquake force transmitted to the structure. Figure 5(a) shows that during summer the maximum deck acceleration is 0.16g which is increased up to 0.19g in winter condition for the EQ #10 ground motion.
From Figure 5(b) it is seen that for EQ #5, #8 and #9, the deck acceleration is higher during summer than in winter condition by an average of 15.6%. On the other hand, for the rest of the ground motions studied in this paper, the deck acceleration is increased during winter temperatures by an average of 11%. This highlights the fact that the effect of temperature must be included with the ground motion for designing the FPS isolation bearing for bridges.

**FPS isolator response:** Figure 6(a) shows the variation of the FPS isolator displacement in summer and winter temperatures for EQ#10. It is seen that the maximum isolator displacement is decreased from 131.56 mm to 112.84 mm from summer to winter conditions, respectively. To assess the post-earthquake functionality of the bridge system, the residual isolator displacement provides an important indication. Higher residual displacement implies lower restoring capacity of the isolator. The residual isolator displacement in winter condition is found to be 95% greater than that observed during summer. However, irrespective of the ground motion, the maximum isolator displacement is higher during summer than that obtained in winter as seen from Figure 6(b). This observation is analogous to the conclusion by Zayas and Low (1999) that, at low temperatures the friction coefficient of FPS increases which in turn decreases its displacement due to seismic excitations.

**Figure 6.** (a) Bearing displacement-time history response under EQ #10, (b) Comparison of maximum isolator displacement between summer and winter conditions.

**Figure 7.** Hysteretic response of the FPS obtained during summer and winter conditions (a) under EQ #2, (b) under EQ #7.

**Energy dissipation capacity:** The ability of the FPS isolator to dissipate the energy that is exerted by the earthquake is represented by its energy dissipation capacity. Similar to the other
properties, the energy dissipation capacity of an isolator is also affected by temperature changes as identified previously (Billah and Todorov 2019). Figure 7 shows the variation in hysteretic response of the FPS during summer and winter conditions subjected to ground motions EQ#2 and EQ#7 for the longer pier. The total energy dissipation is calculated from the area of the hysteresis curves obtained, therefore, the fatter these curves are, the higher energy dissipation capacity. From Figure 7 it is observed that, at low temperature irrespective of the ground motion, the hysteresis curves take smaller area than that at higher temperatures. This provides evidence that the energy dissipation capacity of the FPS is less at low temperatures. From the hysteresis loop, the total energy dissipation at summer condition is calculated to be 230.62 kN-m and 79.41 kN-m which is reduced by 5.27% and 16.83% during winter for EQ#2 and EQ#7, respectively.

<table>
<thead>
<tr>
<th>Table 4. Statistical summary of different response parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Response Parameter</td>
</tr>
<tr>
<td>Pier Base Shear (kN)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Deck Acceleration (g)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Bearing Displacement (mm)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Bearing Residual Drift (%)</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

Finally, Table 4 compared the various response parameters of the FPS isolated bridge under summer and winter conditions. Table 4 summarizes the mean and coefficient of variation (COV) of different response parameters obtained from the 10 ground motions which clearly indicate the effect of cold temperature on the FPS isolated bridge response. As observed for individual motions, the mean values of pier base shear and deck acceleration are higher under winter condition. Similarly, a lower maximum bearing displacement but higher residual drift (%) is observed for the bearings under winter condition.

CONCLUSION

This study evaluated the seismic response of the FPS isolated bridges by analyzing the base shear, deck acceleration, pier top displacement, energy dissipation capacity, maximum isolator displacement, and residual isolator displacement through numerical modeling at temperatures of +40°C and −40°C (summer and winter, respectively). Based on the findings, the following conclusions are drawn:

- The variation in mechanical properties reveals that the FPS bearing showed increased friction and stiffness at extremely low temperatures thereby imparting higher demand on the substructure.
- In general, the deck acceleration of the FPS isolated bridge is found to increase at low temperatures compared to high temperatures.
- The energy dissipation capacity of the FPS isolator is lower during winter compared to that observed during summer. This implies the inadequate performance of the FPS at lower temperatures.
- FPS bearing experienced a 95% higher residual deformation under cold temperature.
compared to summer condition. On the contrary, the maximum isolator displacement is higher during summer compared to that during winter.

Further study should be conducted considering the effect of frictional heating and strength degradation under cyclic loading. Future studies should also focus on the effect of cold temperature on seasonal soil freezing and should consider more ground motions with representing different seismic hazard levels.

ACKNOWLEDGMENT

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Nondestructive Evaluation of a New Concrete Bridge Deck Subject to Excessive Rainfall during Construction: Implications for Durability in a Cold Region

Enoch T. Boekweg¹; W. Spencer Guthrie, Ph.D.²; and Brian A. Mazzeo, Ph.D.³

¹Dept. of Electrical and Computer Engineering, Brigham Young Univ., Provo, UT, USA. E-mail: etboekweg@byu.edu
²Dept. of Civil and Environmental Engineering, Brigham Young Univ., Provo, UT, USA. E-mail: guthrie@byu.edu
³Dept. of Electrical and Computer Engineering, Brigham Young Univ., Provo, UT, USA. E-mail: bmazzeo@byu.edu

ABSTRACT

This study demonstrated the application of nondestructive evaluation techniques for quality assurance of a newly constructed bridge deck in northern Utah that was subjected to an unexpected rainstorm during concrete placement. Because excess water can lead to lower concrete durability, evaluating the ability of water and chloride ions to penetrate the concrete and quantifying the overall protection of the reinforcing steel were important objectives. Several deck properties were measured, including concrete cover depth, deck surface temperature, resistivity, vertical electrical impedance (VEI), and Schmidt rebound number. Statistical analyses performed on the collected data indicated that the section most affected by the rain exhibited a lower Schmidt rebound number but was not different from the other sections in terms of resistivity or VEI; therefore, the results of the testing suggest that the effect of the rain was limited to a shallow depth of concrete, which was corroborated by petrographic analysis performed on several cores removed from the bridge deck. The upper approximately 0.13 in. was then milled from the deck surface before a polyester polymer concrete overlay was applied to seal the deck.

INTRODUCTION

Nondestructive evaluation (NDE) has been extensively used to inform decisions about repair and rehabilitation of existing transportation infrastructure (Guthrie et al. 2019). NDE can also be a valuable resource to provide quality assurance before acceptance of new infrastructure. In this study, NDE was used to evaluate a newly constructed concrete bridge deck in northern Utah. The durability of the bridge deck was potentially affected by an unexpected rainstorm during concrete placement. Because excess water can lead to reduced strength and/or increased permeability of concrete (Kim et al. 2014, Popovics and Ujhelyi 2008), evaluation of the deck was important. Specifically, given that chloride-induced corrosion of the top mat of reinforcing steel is the leading cause of deck damage in northern Utah as a result of routine deicing salt applications during winter maintenance (Barton et al. 2019b), evaluating the ability of water and chloride ions to penetrate the concrete and quantifying the overall protection of the reinforcing steel were important objectives. The following sections provide background information, explain the procedures, present the results, and offer conclusions.

BACKGROUND

The scope of this work was necessarily limited to nondestructive testing to preserve the condition of the bridge deck. To evaluate the durability of the deck, several deck properties were measured, including concrete cover depth, deck surface temperature, resistivity, vertical electrical...
As described in the following sections, each method provided potentially useful information related to the objectives of the work.

**Cover Depth:** The cover depth is the distance between the nearest reinforcing steel, or rebar, and the surface of the concrete deck. Cover depth is typically estimated with an electromagnetic cover meter, which relies on the conductive properties of the steel reinforcement to produce an eddy current in response to a magnetic pulse (Fernandez 2005). Cover meters can accurately estimate the depth of the rebar, particularly when the diameter of the steel reinforcement is known. Cover thickness is an indication of the protection of the steel reinforcement against chloride ingress.

**Surface Temperature:** Based on inferred emissivity properties, the surface temperature of a concrete surface can be easily estimated using many readily available devices. A handheld infrared thermometer, as used in this study, is especially useful for obtaining spot readings at locations of interest. Because chloride ion diffusivity, and hence electrical resistivity, is affected by temperature, measuring deck surface temperature can be useful if large temperature differences occur during resistivity or VEI testing, in particular.

**Resistivity and Vertical Electrical Impedance:** Both resistivity and VEI are electrical measurements designed to quantitatively assess the resistance of concrete to chloride ion penetration, where higher resistivity or impedance typically indicates reduced diffusivity of chlorides through concrete (Guthrie et al. 2018, Bartholomew et al. 2012). Assessing the resistance to chloride ion penetration is critical in cold regions where chloride-based deicing salts are routinely applied to the deck surface during winter maintenance operations. As chloride ions diffuse through the concrete deck and accumulate in the vicinity of the rebar within the deck, steel corrosion, concrete cracking, and premature deck failure can occur (Barton et al. 2019b, Broomfield 2003).

Traditionally, depending on the probe spacing of the testing device, resistivity measurements are used to evaluate concrete to a depth of approximately 2 in. (McCarter et al. 2009), which is a typical concrete cover thickness (Hema et al. 2004). VEI measurements, however, evaluate the total protection offered to the steel reinforcement by the full depth of the concrete cover as well as any epoxy coatings that may be present on the rebar. In a VEI test, the electrical impedance between two electrodes is measured. One of the probes is much larger than the other probe and is called the large-area electrode (LAE) (Mazzeo and Guthrie 2019, Barton et al. 2019a). The LAE forms a low-resistance electrical connection to the rebar as required for the testing. Current measured through the smaller probe allows calculation of the electrical resistance between the concrete surface and the rebar.

**Schmidt Rebound Number:** A Schmidt rebound hammer can be used to estimate the compressive strength of concrete by measuring the rebound of a sprung mass after it strikes the surface of the concrete, where higher rebound numbers indicate harder, stronger concrete. When testing at a particular location is desired and surface grinding is not performed, repeated testing is warranted because crushing of the concrete matrix constituting the surface texture at that location absorbs energy that would otherwise contribute to the hammer rebound. Therefore, to ensure more representative data, only later rebound numbers are analyzed in this approach.

**PROCEDURES**

The motivation for the testing performed in this research was to compare specific sections of the bridge deck, at least one of which was placed during active rain, to assess potential differences in selected concrete properties among the sections. Depicted in Figure 1, the deck was constructed...
using epoxy-coated rebar, was approximately 120 ft long and 60 ft wide (between the parapets), and was divided into three sections that were labeled A, B, and C in order from south to north, as shown in Figure 2 (not to scale). The section boundaries, which are delineated by dashed lines in Figure 2, were defined by contractor personnel. To minimize any possible bias in the evaluation, the section(s) of the deck that was placed during active rain, as well as the amount of rainfall, was not disclosed to the researchers until after the testing and analyses were complete. Within each of the three sections, a pattern comprising 10 test locations was marked for evaluation, for a total of 30 test locations.

Figure 1. Concrete bridge deck.

The bridge deck was constructed in March 2020, and the testing was performed in May 2020, almost two months later. The results of the testing are specific to the deck conditions that were prevalent during the testing period. Specifically, at the time of the testing, the deck surface was in direct sunlight and appeared to be dry, the air temperature was generally between 85 and 90°F, and wind gusts of 10 to 20 mph were typical.

As previously indicated, several deck properties were measured, including concrete cover depth, deck surface temperature, resistivity, VEI, and Schmidt rebound number. All measurements were made starting at the south end of the bridge and moving to the north end, as numbered in Figure 2.

At each test location, the positions of the nearest longitudinal and transverse bars were determined using a cover meter and marked on the deck surface. The cover depths over those bars were then recorded. Next, the surface temperature of the concrete was measured using a spot radiometer, and then resistivity was measured using a four-prong resistivity device with a prong spacing of 2.0 in, which was consistently oriented in the transverse direction; the prongs were always placed 2 to 3 in. away from the marked rebar. After the resistivity measurements, VEI was measured at each point. The LAE of the VEI apparatus was placed in the northwest corner of the deck, as shown in Figure 3, where it remained for the duration of the testing. The VEI probe was connected to the LAE by a flexible wire and was moved from one test location to the next. In order
to establish a reliable electrical connection between the LAE and the deck surface, the concrete under the LAE was soaked with water and regularly re-soaked, and plastic sheeting was placed over the LAE to minimize water evaporation. In addition, the concrete at each test location was soaked with water prior to obtaining a measurement with the VEI probe. VEI measurements were recorded for at least one minute at each location. After VEI measurements were obtained and the deck surface at each test location appeared to be dry, the Schmidt hammer test was performed at each test location. The Schmidt hammer test was repeated four times at each test location, and the fourth test at each location was recorded.

![Figure 2. Test locations within sections of the bridge deck.](image)

After data collection, an analysis of variance (ANOVA) was performed to determine if statistically significant differences were present between the three sections. An ANOVA is a method of hypothesis testing that results in a calculated probability, or $p$-value, that is compared to a threshold value for determining whether a null hypothesis can be rejected or not. If the null hypothesis can be rejected, an alternative hypothesis is accepted. In this test, the null hypothesis was that all three sections were the same, while the alternative hypothesis was that at least one section was different from another section. When the $p$-value resulting from the ANOVA was less than or equal to 0.05, which was the threshold value specified for this analysis, the null hypothesis could be rejected, and the alternative accepted; in this case, Tukey’s mean separation procedure was subsequently employed to identify the specific sections that were different from each other.
RESULTS

The results of the testing are presented in Table 1, which shows cover depth, deck surface temperature, resistivity, VEI, and Schmidt rebound number for each of the 30 test locations, where cover depth was measured over both the transverse and longitudinal reinforcing steel. Table 1 provides the average and standard deviation for each measured property for each section, while Table 2 provides corresponding $p$-values and conclusions from the ANOVA testing.

The $p$-values for four properties were less than or equal to 0.05, indicating that at least one section was different from another section with respect to those properties. Cover depths over both the transverse and longitudinal reinforcing steel were determined to be significantly different between sections B and C. Deck surface temperatures were determined to be different between sections A and C and also between sections B and C. Schmidt rebound numbers were determined to be different between sections A and C and also between sections B and C. Section C had the highest average cover depth, the lowest deck surface temperature (it was tested last and had experienced measurable cooling from progressive changes in ambient conditions), and the lowest average Schmidt rebound number.

The $p$-values for the remaining two properties, resistivity and VEI, were both greater than 0.05, indicating that insufficient evidence existed to differentiate among the sections. Although the lowest average resistivity measurement occurred in section C, the comparatively small differences between section C and either section A or B are not statistically significant. The lowest average VEI value occurred in section A, but the comparatively small differences between section A and either section B or C are also not statistically significant. Although variations in deck temperature
and/or cover depth could potentially affect resistivity and VEI, explicitly accounting for
differences in these properties among the sections did not change the outcome of the analyses.

<table>
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<tr>
<th>Section</th>
<th>Location</th>
<th>Cover Depth (in.)</th>
<th>Deck Temp. (°F)</th>
<th>Resistivity (kΩ-cm)</th>
<th>VEI (Ω)</th>
<th>Rebound Number</th>
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DISCUSSION

After the testing and analyses were completed, contractor personnel explained that the concrete in section A was finished and covered prior to the onset of the rain, the concrete in section B was mostly placed but not yet covered at the time of the rain, and the concrete in section C was actively placed during the rain; 0.31 in. of rainfall was measured at the site during the bridge deck placement. Therefore, section C was the most likely to exhibit reduced concrete strength and/or increased permeability. Several points of explanation were subsequently developed for consideration.

If the concrete in section C was internally vibrated during construction, as would be expected, excess water in the concrete would have risen to the surface. While some of the excess water may have been incorporated into the surface of the concrete during finishing operations, it may not have affected the full cover depth given that the resistivity test, which measures to a depth of approximately 2 in., could not differentiate among the three deck sections.

### Table 2. Results of Statistical Analyses

<table>
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<tr>
<th>Measurement</th>
<th>P-Value</th>
<th>Conclusion</th>
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<td>Cover Depth (Trans.)</td>
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<td>B Differs from C</td>
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<td>Cover Depth (Long.)</td>
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<td>B Differs from C</td>
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<td>Deck Temp.</td>
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<td>A Differs from C</td>
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<td></td>
<td></td>
<td>B Differs from C</td>
</tr>
<tr>
<td>Resistivity</td>
<td>0.399</td>
<td>Sections Do Not Differ</td>
</tr>
<tr>
<td>VEI</td>
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<td>Sections Do Not Differ</td>
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<td>Rebound Number</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>B Differs from C</td>
</tr>
</tbody>
</table>

The VEI testing, which measures the impedance from the deck surface to the top mat of reinforcing steel, was likely governed by the epoxy coating on the rebar. Because the concrete and the epoxy coating are in series (in terms of an electrical circuit), the one with the most resistance governs the overall result. If the epoxy coating was substantially damaged in one or more locations, the concrete would have governed in those cases. The impedance testing also could not differentiate among the three deck sections, suggesting that the reinforcing steel had consistent protection from chloride ions across all three sections.

The lower Schmidt rebound numbers in section C indicate that the concrete in section C was weaker at the surface compared to the concrete in sections A and B. However, section C also had the highest cover depth, which ensures greater protection of the reinforcing steel compared to lower cover depth, all other factors equal. Therefore, although the degree to which the higher concrete cover depth may mitigate the effects of reduced concrete strength and/or increased permeability at the surface was not determined, the results of the testing suggest that the effect of the rain was limited to the surface of section C.

After reviewing the results of the nondestructive testing, contractor personnel arranged to investigate the depth of affected concrete through petrographic analysis of several cores removed from the bridge deck. The petrographer verified that the concrete in section A had not been affected by the rain and reported that the rain had affected only the upper 0.08 in. and 0.13 in. of the concrete in sections B and C, respectively. Contractor personnel then milled the deck surface to a depth of 0.13 in. to remove the affected concrete and applied a 0.90-in.-thick polyester polymer concrete overlay, which has been demonstrated in previous research to provide excellent bridge deck...
CONCLUSION

This study demonstrated the application of NDE techniques for quality assurance of a newly constructed bridge deck in northern Utah that was subjected to an unexpected rainstorm during concrete placement. To evaluate the durability of the deck, several deck properties were measured, including concrete cover depth, deck surface temperature, resistivity, VEI, and Schmidt rebound number. Statistical analyses performed on the collected data indicated that the section most affected by the rain exhibited a lower Schmidt rebound number but was not different from the other sections in terms of resistivity or VEI; therefore, the results of the testing suggest that the effect of the rain was limited to a shallow depth of concrete, which was corroborated by petrographic analysis performed on several cores removed from the bridge deck. The upper approximately 0.13 in. was then milled from the deck surface before a polyester polymer concrete overlay was applied to seal the deck. The techniques demonstrated in this study may be useful for assessment of other bridge decks for which evaluating the ability of water and chloride ions to penetrate the concrete and quantifying the overall protection of the reinforcing steel are important objectives.

ACKNOWLEDGEMENTS

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REFERENCES


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Quantification of Rut Detection and Height Mapping in Winter Terrains for Off-Road Mobility

Anthony J. Fuentes¹; Sergey N. Vecherin, Ph.D.²; Mark O. Bodie³; and Michael W. Parker⁴

¹U.S. Army Engineer Research and Development Center, Cold Regions Research and Engineering Laboratory, Hanover, NH. E-mail: Anthony.J.Fuentes@usace.army.mil
²U.S. Army Engineer Research and Development Center, Cold Regions Research and Engineering Laboratory, Hanover, NH. E-mail: Sergey.N.Vecherin@usace.army.mil
³U.S. Army Engineer Research and Development Center, Information Technology Laboratory, Lorton, VA. E-mail: Mark.O.Bodie@usace.army.mil
⁴U.S. Army Engineer Research and Development Center, Cold Regions Research and Engineering Laboratory, Hanover, NH. E-mail: Michael.W.Parker@usace.army.mil

ABSTRACT

Off-road autonomous vehicle navigation in winter environments requires reliable identification and quantification of potential obstacles, such as deep vehicle rutting or buried objects. The advent of consumer-grade light detection and ranging (LiDAR) sensors and unmanned aerial system (UAS) based photogrammetry present new avenues for the implementation of change detection algorithms for the purpose of obstacle identification. Few studies have provided a quantifiable statistical method for determining the input parameters of these change detection algorithms based upon user-defined confidence metrics. Previous detection methods also fail to derive the degree of assurance associated with the identification of a perceived obstacle. Here, we present an automated method for identification of snow-covered obstacles and vehicle ruts within LiDAR-derived digital elevation models based on false-alarm and detection probabilities. Detection maps and accurate height maps are generated for snow-covered objects by the algorithm to demonstrate the reliability of this method to assist with obstacle avoidance in snowy off-road conditions. The algorithm described here is a reliable and fast method for the identification and measurement of snow-covered obstacles. While this study is concerned with snow-covered terrain, the methods described here may be leveraged to monitor route deformation features as a result of vehicle traffic across a variety of terrain types.

INTRODUCTION

The deformation of northern soils and terrains due to heavy vehicle traffic is of great importance to both civilian and military cold region operations. Intensive rutting of frozen soils and snow-covered terrain by extensive vehicle traffic can significantly reduce mobility and necessitates continuous monitoring throughout a vehicle’s route in order to mitigate degradation of the terrain. Manual measurements of ruts are highly time consuming and requires a team of surveyors to properly characterize each set of ruts (Gezero and Antunes, 2019). A military convoy operating in winter conditions will require a more comprehensive estimation of the rut depths with a higher recurrence interval than the manual measurements, which can only characterize a single cross section of a rut at a time. Other studies have highlighted a similar need in northern forestry operations for fast and comprehensive rut depth measurement algorithms using remote sensing data from visual imagery and Light Detection and Ranging (LiDAR) sensors (Nevalainen et al., 2017; Marra et al., 2018; Salmivaara et al., 2018; Botha et al., 2018).

Commercially available red, green, blue (RGB) cameras have been used to quantify rut depths...
by several studies using UAS and terrestrial based photogrammetry (Nevalainen et al., 2017; Marra et al., 2018). While relatively inexpensive, the quality of visual camera imagery depends on the illumination of the environment, limiting their usefulness in low light or nighttime operations. LiDAR has the advantage over visual imagery based collections because it is not dependent on the quality of incident light, can operate under complete darkness, and can leverage multiple returns to better characterize the structure of an object. A variety of object recognition approaches have been developed for LiDAR data that leverage low-level attributes such elevation, point density and return intensity (Che et al., 2019). Many of these algorithms are scene specific (e.g. urban environments, forests) with underlying assumptions about the structure of the environment, such as consistent geometry in railway tracking or planarity in urban environments (Che et al., 2019). These assumptions result in effective object recognition for a specific environment but limit the generalizability of these approaches to a broader range of scenes. Few studies have presented a statistical assurance framework whereby the user is provided a probability that the identified features within a scene actually correspond to a true obstacle. Here, we present the initial application of a proprietary statistical object detection algorithm to identify and quantify vehicle ruts in snow from LiDAR-derived Digital Elevation Models (DEMs). This novel algorithm incorporates user defined risk acceptance and detection metrics to produce object detection and height maps that can be incorporated into route planning for manned and autonomous off-road vehicles.

METHODS

LiDAR Data Collections

Two winter testing collections were conducted on 12/18/2019 and 01/09/2020 at the Cold Regions Research and Engineering Laboratory (CRREL) in Hanover, NH. The test site is a flat 50 x 30 meter concrete surface with raised rubber speed bumps of differing dimensions that were placed approximately two-thirds of way through the traverse (Fig 1.). The dimensions (height x length x width; in cm) of the bumps from west to east respectively are 7.62 x 365.76 x 29.85, 5.08 x 365.75 x 29.85, and 5.08 x 355.6 x 89.54. The LiDAR sensor employed in this study is a Velodyne HDL-32E 3-Dimensional LiDAR that is commonly used in autonomous vehicle research. The HDL-32E is 32 channel, single and dual return (strongest return, last return), 3-D LiDAR sensor with a +10° (upward) and -30° (downward) field of view (FOV) and a range of 100 meters. The standard accuracy of returns for the sensor is ±2 cm. The HDL-32E is paired with a KAARTA Stencil 2 mobile mapping system that uses an integrated visual camera to determine the sensor pose through the identification of common landmarks within the scanned region using a proprietary Simultaneous Localization and Mapping (SLAM) algorithm. The advantage of the Stencil 2 system is that it is able operate in GPS-denied or GPS-limited regions such as under tree cover or inside of a tunnel.

The test vehicle is a High Mobility Multipurpose Wheel Vehicle (HMMWV) instrumented with an array of sensors for recording the response of the HMMWV as it travels over the snow-covered speed bumps at varying speeds. A 5th wheel was mounted to the right rear of the vehicle to measure the true speed of the vehicle and therefore only the left rut was measured manually for depth due to the potential distortions in the rut from the 5th wheel. The Velodyne HDL-32E was mounted on a tripod near the center of the HMMWV such that it is elevated above the cabin of the vehicle to minimize the presence of the vehicle within the scans (Fig. 1). Previous collections with the Stencil 2 demonstrated that the integrated visual camera should be oriented parallel with the
horizon in order to acquire enough fiduciary objects to properly register the point cloud to the local coordinate system of the sensor. Angling of the sensor resulted in too few tie points identified for the SLAM algorithm resulting in distortions within the point cloud so the sensor was kept in its vertical orientation.

Figure 1. Overview of LiDAR acquisition and test site configuration. A. HMMWV test vehicle with the tripod-mounted Velodyne HDL-32E LiDAR mounted on the rear, B. photo of vehicle ruts after final vehicle traverse, C. image of the snow-covered rubber speed bumps, and D. example of virgin snow depth measurement.

The tests consisted of six South to North, unidirectional vehicle traverses over the buried speed bumps. The traverses were spaced such that the test vehicle did not reenter the ruts of previous passes in order to preserve the original structure. The HMMWV traveled at 8 kph over the first four passes and 16 kph during the final two passes. Manual virgin snow depth measurements were collected concurrently with the vehicle testing. Rut depth measurements were collected at four locations in the direction of travel of each left rut to characterize the deformation around and directly over the buried speed bumps. Virgin snow depth measurements are defined as the distance from the underlying concrete surface (or solid ice layer) and compacted snow depth is measured from the concrete surface, or bump surface when on top of the bump, to the top of the compacted snow in the wheel track. The rut depth is the difference between the virgin snow depths directly adjacent to the ruts and the height of the compacted snow in the ruts (Table 1). Snow pits were dug on both testing days in order to characterize the snow temperature, grain size, and density of the snow pack to provide additional details about the strength of the snow.

**LiDAR Point Cloud Processing**

The resulting raw point cloud data was pre-processed and corrected using the open-source CloudCompare software. Differential Global Navigation Satellite System (DGNSS) ground control points collected within study area were used to georeferenced the point cloud prior to DEM creation. Due to the small overlap between the FOV of the sensor and the HMMWV, a blind radius of 2 meters was applied to the data in order to remove returns reflected by the test vehicle. In order to better resolve the ruts, it is necessary to apply a ground object detection filter to remove non-ground objects such as trees or buildings in order to produce a Digital Terrain Model (DTM) or
bare-earth DEM. We compared the ground point classification results of the Progressive Morphological Filter (PMF) of Zhang et al., 2003 and the Simple Morphological Filter of Pingel et al., 2013 to the Cloth Simulation Filter (CSF) method of Zhang et al., 2016.

Based upon its superior performance and simplicity the ground point classification algorithm used in this study is the CSF algorithm, which works by inverting the point cloud and applying a semi-rigid modeled ‘cloth’ to the inverted point cloud to classify non-ground objects. For brevity, a conceptual description of the algorithm is given here and a comprehensive description of the cloth fitting procedure can be found in Zhang et al., 2016. The simulated cloth is composed of interconnected points whose overlay behavior is governed by a mass-spring model based on Hooke’s law. The inverted LiDAR points and the cloth point are projected onto the same 2D horizontal plane and the simulated cloth is mapped to the LiDAR point cloud surface following the protocol described in Zhang et al., 2016. The nearest cloth point for each LiDAR point in the projected 2D plane is identified and the elevation difference between the two points is calculated. All LiDAR points with a height difference greater than the threshold value (default is 0.5 meters) are classified as non-ground objects.

Table 1. Manual rut depth measurements taken within the left rut (relative to the south-north direction of travel) at four locations along each traverse for the 12/18/2019 and 01/09/2020 testing days.

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<td>2m North of Bump</td>
<td>1/9/2020</td>
<td>10.5</td>
<td>11</td>
<td>10.5</td>
<td>9.5</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

The CSF algorithm exhibited better ground classification results than either of the two PMF methods and retained more ground points adjacent to the buildings within the scene than the PMF or SMRF algorithms. The morphological filtering methods of Zhang et al., 2003 and Pingel et al., 2013 require that a slope parameter be supplied in order to account for regions with variation in the relief of the terrain. Both of the morphological filtering methods require some a priori knowledge of the terrain within the region of interest, such as the amount of slope and the height and size of non-ground objects such as buildings or vegetation. The advantage of the CSF algorithm is that it does not require as many a priori assumptions about the terrain (e.g. slope, object size) making the algorithm more applicable to a wider variety of situations while retaining high classification accuracy. The pre-processed point clouds were subjected to non-ground point classification and filtering using the CSF function in the Point Data Abstraction Library (PDAL) in Python. The filtered point clouds were then converted into 5-cm spatial resolution DTM in order to preserve the full profile of the ruts.
Height Map Algorithm

A desirable object size must be taken into consideration when defining an obstacle within a given scene, as features below or above a certain threshold may or may not significantly impact mobility. This threshold is dependent upon the dimensions of the vehicle (e.g., wheel base, ground clearance) as very large (hills) or small features (snow dunes) may not present a challenge to mobility, while intermediately sized objects like ruts may limit trafficability. As with the filtering algorithms, some a priori knowledge of the potential obstacles in the region of interest as well as vehicle tolerances are required in order to adjust the resultant height maps to the appropriate scale to improve visibility of smaller anomalies. The algorithm presented here consists of three steps, the generation of a smoothed background surface, subtraction of background from the original DTM surface, and the scaling of the resulting height map to allow for increased visibility of the ruts.

A background surface that represents the undisturbed scene is generated by passing a moving window average across the DTM to produce a smoothed surface with the ruts removed. For this study a 12 x 12 pixel window was used, but the window size of the moving average can be adjusted based upon the size of the obstacles of the scene. The background surface is then subtracted from the original DTM generating a difference raster, effectively removing the slope from the scene and converting the rut pixels from mean sea level elevation to relative height in meters. The difference raster was then scaled to remove positive values (objects above mean surface) to improve the visibility of the ruts in the height maps. The height map algorithm was used to create scaled rut depth maps for both testing days.

Figure 2. Probability density functions (PDFs) generated using kernel density estimation (KDE) for the standardized X (blue, dashed line) and Y (brown, solid line) spatial derivative rasters (vertical and horizontal Sobel filters), X derivative background images (black, solid line) and Y derivative background images (grey, solid line). A. KDE results for the 12/18/2019 data and B. PDFs for the 01/09/2020 data.

Object Detection Algorithm

The goal of object detection can be framed as a binary classification problem where an obstacle
(e.g. vehicle ruts) can be either detected or not detected. This framing allows the problem to be evaluated using a Receiver Operating Characteristics (ROC) framework, where the probability of an obstacle detection (\(P_d\)) is dependent on the specified probability of false alarm (\(P_{fa}\)) (Fawcett, 2006; Powers, 2011).

Under the ROC model, \(P_d\) and \(P_{fa}\) both reach limits of 1 and 0 simultaneously, meaning that 100% detection is only possible if 100% of the objects identified are also false alarms and zero false alarms are only possible if there are zero detections. The corollary to this is that well-performing detection algorithms will have high \(P_d\) with low \(P_{fa}\). In this section, we outline our novel object detection algorithm based on the ROC framework applied to common edge detection and texture analysis methods.

Edge detection algorithms are among the most commonly used feature identification techniques used in image processing. Large changes in adjacent elevation values in a DTM can be used to define the edge of an object such as a fence or a rut. While there are several approaches for edge detection, we focus on the Sobel method for this test case. The Sobel method involves the computation of the X and Y spatial derivatives of the DTM through the use of 3 x 3 window in order to identify the horizontal (Y derivative) and vertical edges (X derivative). The final Sobel edge detection raster is generated by calculating the 2-D spatial gradient.

Texture analysis seeks to determine the relationship between a pixel and a neighborhood of surrounding pixels. Pixels at the boundary between objects or other terrain classes will show large variation that can be used to segment an image or other raster. For the texture analysis of the CRREL backpack data, we employed a 3 x 3 moving standard deviation filter that calculates the standard deviation for a neighborhood of pixels and assigns that value to the central pixel of the filtered raster.

In order to define a statistical detection framework it is necessary to determine which families of distributions can be used to describe a scene through evaluating the probability distribution function (PDF) of a dataset. A Kernel Density Estimator (KDE) is a non-parametric estimation method for determining the distribution of a set of random values free from any assumptions about the shape of the PDF (Wand and Jones, 1994).

A KDE was used to estimate the PDF of the X and Y derivatives of the original DTM (vertical and horizontal Sobel filtered rasters) with the Seaborn Python library. The appearance of the estimated PDF for a given raster is contingent upon the optimum bandwidth value selected for the KDE. There are several approaches to determining the bandwidth of the KDE, but K-fold cross-validation is the most robust method for the estimation of the optimum bandwidth that avoids overfitting the model. We employed 5-fold cross validation on the KDE model for the X and Y derivative of the original DTM to determine the bandwidth of the model. For comparison of the distributions, an analytic background normal distribution for each derivative was generated. The background distribution is representative of the scene without the obstacles and therefore, must be described using metrics that are insensitive to outliers in order to mask the influence of obstacles. Robust estimations of the mean and standard deviation of a given scene that are insensitive to outliers can be determined using the sample quantiles using Eqn. 1, where \(\mu_x\) is the mean and \(\sigma_x\) is the standard deviation for the normal background PDF, and \(q_{25}\), \(q_{50}\) and \(q_{75}\) are 0.25, 0.5, and 0.75 quantiles for DTM pixel values of the image (Johnson and Wichern, 2002).

\[
\mu_x = q_{50}, \quad \sigma_x = (q_{75} - q_{25})/1.349 \quad \text{Eqn. 1}
\]

Background analytical distributions for the vertical and horizontal Sobel filtered rasters were generated assuming a normal PDF with the mean and standard deviations calculated by Eqn. 1 (Fig. 2). Figure 2 shows that the data are well described by a normal distribution. Since the X and
Y spatial derivatives of the DTM follow a normal distribution and are independent of one another, the chi-squared distribution was used to describe the magnitude of the spatial gradient.

To do so, one must first standardize both rasters using Eqn. 2, where $\mu$ and $\sigma$ are derived from Eqn. 1. This results in data that follow a standard normal distribution and their sum of squares is by definition a chi-squared distribution with two degrees of freedom (Eqn. 3).

$$
\bar{g}_x = \frac{g_x - \mu_x}{\sigma_x}, \quad \bar{g}_y = \frac{g_y - \mu_y}{\sigma_y}
$$

Eqn. 2

$$
\bar{g}^2 = \bar{g}_x^2 + \bar{g}_y^2
$$

Eqn. 3

Given that $\bar{g}^2$ follows a chi-squared distribution, it is possible to define the object detection threshold for a specific probability of false alarm. The object detection threshold value for the 2D spatial gradient is determined from Eqn. 4 where the threshold value is derived from the inverse chi-squared cumulative distribution function with two degrees of freedom at a given $P_{fa}$ (Eqn. 4). This threshold can also be defined for other filtering methods such as the standard deviation filter or a moving average using an inverse chi-squared distribution with 1 degree of freedom. The threshold value produced from Eqn. 4 can be used to segment the Sobel or standard deviation filtered DTM to output a Boolean raster in which pixels are defined as obstacles if they exceed the threshold at a supplied $P_{fa}$.

$$
T = \text{invCDF}_{\chi^2_2} \left( 1 - P_{fa} \right)
$$

Eqn. 4

RESULTS

HMMWV Rut Depths

Scaled rut depth maps for both testing days output by the height map algorithm are presented in Figs. 3A and 3B. The left wheel rut depths for each test date are presented in Tables 1 and 2 for each of the four measurement locations relative to the snow covered speed bumps. The average virgin snow depth for the 12/18/19 and 01/09/20 tests were 8.5 and 11.5 cm respectively and the average rut depths are 6.46 and 9.73 cm. Right wheel rut depths are reported in Table 3.

Rut depths from the height maps were extracted at approximately the same locations as the manual measurements using ArcGIS (Fig 4A). These values were imported into Python to develop the cross-sections presented in Fig. 4A. The subtraction of the original DTM from the smoothed background DTM successfully removed the minor slope within the scene as illustrated by the cross-sections in Fig 4B and 4C. It is apparent in Figs. 3A and B that the ruts directly behind the vehicle (leftmost ruts in both figures) appear shallower than those of the previous runs as shown by the lighter blue color within these ruts. Additionally, the rut depths calculated for the ruts adjacent to a given pass (e.g. the second to last set during the last run) are deeper (approximately 5 cm) than the ruts further away from the current pass (Figs. 4C and 4B).

Object Detection

For obstacle detection, the Sobel edge detection and the STD filters were used with detection thresholds derived from Eqn. 4 at 0.1 and 0.3 $P_{fa}$ in order to test the sensitivity of each filtering method to changes in the detection threshold (Fig. 5). The standardized Sobel edge detection raster is less sensitive to changes in $P_{fa}$ but classifies more of the background scene pixels as obstacles than the standard deviation filter which showed greater variation in the amount of obstacles detected as the threshold value was altered (Fig.5). The deeper ruts within the scene are resolved
better at lower probability of detection values in the standard deviation detection map than the shallower ruts due to the larger gradient between the bottom of the rut and the surface. The Sobel filter resolved the shallower ruts more clearly along the entire length of the ruts at both 0.1 and 0.3 $P_{fa}$.

Table 2. Height Map derived HMMWV maximum rut depths for the left wheel (traverse is South to North) measured at the same intervals as the manual measurements. Ruts numbered from west to east.

<table>
<thead>
<tr>
<th>Measurement Location</th>
<th>Test Date</th>
<th>Rut 1 Height (cm)</th>
<th>Rut 2 Height (cm)</th>
<th>Rut 3 Height (cm)</th>
<th>Rut 4 Height (cm)</th>
<th>Rut 5 Height (cm)</th>
<th>Rut 6 Height (cm)</th>
<th>Rut 7 Height (cm)</th>
<th>Rut 8 Height (cm)</th>
<th>Rut 9 Height (cm)</th>
<th>Rut 10 Height (cm)</th>
<th>Rut 11 Height (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Meters South of Bump</td>
<td>12/18/2019</td>
<td>-3.52</td>
<td>-3.52</td>
<td>-4.16</td>
<td>-3.81</td>
<td>-4.81</td>
<td>-4.19</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5 Meter South of Bump</td>
<td>12/18/2019</td>
<td>-5.01</td>
<td>-3.81</td>
<td>-3.54</td>
<td>-4.45</td>
<td>-5.41</td>
<td>-4.73</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Of Bump</td>
<td>12/18/2019</td>
<td>-0.05</td>
<td>0</td>
<td>-1.31</td>
<td>-2.78</td>
<td>-4.6</td>
<td>-2.57</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Meters North of Bump</td>
<td>12/18/2019</td>
<td>-3.53</td>
<td>-2.87</td>
<td>-3.21</td>
<td>-5.05</td>
<td>-4.3</td>
<td>-4.41</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Meters South of Bump</td>
<td>1/9/2020</td>
<td>-4.67</td>
<td>-5.84</td>
<td>-6.18</td>
<td>-7.3</td>
<td>-7.77</td>
<td>-7.6</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5 Meter South of Bump</td>
<td>1/9/2020</td>
<td>-3.41</td>
<td>-4.98</td>
<td>-6.54</td>
<td>-8.78</td>
<td>-8.31</td>
<td>-7.84</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Of Bump</td>
<td>1/9/2020</td>
<td>-3.71</td>
<td>-2.45</td>
<td>-4.41</td>
<td>-6.85</td>
<td>-4.6</td>
<td>-5.38</td>
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<tr>
<td>2 Meters North of Bump</td>
<td>1/9/2020</td>
<td>-4.49</td>
<td>-4.09</td>
<td>-5.08</td>
<td>-7.56</td>
<td>-7.68</td>
<td>-7.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Height Map derived HMMWV maximum rut depths for the right wheel (traverse is South to North) measured at the same intervals as the manual measurements. Ruts numbered from west to east.

<table>
<thead>
<tr>
<th>Measurement Location</th>
<th>Test Date</th>
<th>Rut 2 Height (cm)</th>
<th>Rut 3 Height (cm)</th>
<th>Rut 4 Height (cm)</th>
<th>Rut 5 Height (cm)</th>
<th>Rut 6 Height (cm)</th>
<th>Rut 7 Height (cm)</th>
<th>Rut 8 Height (cm)</th>
<th>Rut 9 Height (cm)</th>
<th>Rut 10 Height (cm)</th>
<th>Rut 12 Height (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Meters South of Bump</td>
<td>12/18/2019</td>
<td>-4.03</td>
<td>-3.79</td>
<td>-4.86</td>
<td>-5.13</td>
<td>-2.72</td>
<td>-3.4</td>
<td></td>
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</tr>
<tr>
<td>0.5 Meter South of Bump</td>
<td>12/18/2019</td>
<td>-4.26</td>
<td>-3.49</td>
<td>-4.04</td>
<td>-4.69</td>
<td>-2.9</td>
<td>-2.35</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Of Bump</td>
<td>12/18/2019</td>
<td>0</td>
<td>-0.37</td>
<td>-2.55</td>
<td>-4.38</td>
<td>-3.98</td>
<td>-1.17</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Meters South of Bump</td>
<td>1/9/2020</td>
<td>-5.78</td>
<td>-7.26</td>
<td>-7.15</td>
<td>-7.04</td>
<td>-4.05</td>
<td>-6.79</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5 Meter South of Bump</td>
<td>1/9/2020</td>
<td>-3.86</td>
<td>-5.96</td>
<td>-7.95</td>
<td>-7.94</td>
<td>-4.11</td>
<td>-5.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Of Bump</td>
<td>1/9/2020</td>
<td>-3.97</td>
<td>-3.50</td>
<td>-5.11</td>
<td>-7.42</td>
<td>-3.70</td>
<td>-3.66</td>
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<td></td>
<td></td>
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<tr>
<td>2 Meters North of Bump</td>
<td>1/9/2020</td>
<td>-3.89</td>
<td>-3.79</td>
<td>-6.73</td>
<td>-7.57</td>
<td>-4.40</td>
<td>-3.05</td>
<td></td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

DISCUSSION

The advantage of the $P_{fa}$-based algorithm described here is it allows the user to define the level of risk they are willing to accept that a potential obstacle is not identified. In the case of buried ordinance for example, one would be willing to increase the sensitivity of the algorithm at the cost of more false alarms due to the penalty of missing a potential explosive. A homogenous environment, such as a scene with undisturbed snow, will exhibit very few identified obstacles even at very high false alarm probabilities.
The results shown here illustrate the capability of the algorithm to distinguish vehicle ruts quickly and without the need for computationally expensive pre-training. A remaining challenge is to provide an estimate to the user of the likelihood that a detected object is actually an obstacle. The binary detection algorithm will be integrated with a probability of detection algorithm that is currently in development within the CRREL Cold Mobility Team. This functionality will output a
probability map where each identified pixel from the detection step is assigned a probability that it is a true obstacle.

The height map algorithm presented here is a solution for deriving rut depths in the absence of a virgin snow (rut-free) DEM and can be applied to other northern terrain environments. While the algorithm functions as intended, the accuracy of the results were limited by the mounting and viewing geometry of the LiDAR sensor. While a single rotating 3D, multibeam, LiDAR sensor such as the HDL-32E can provide a robust dataset of the environment around a vehicle, a single sensor may still experience occlusion of potential obstacles due to a single perspective (Meadow et al. 2019).

An issue with the mounting of the HDL-32E centrally on the vehicle is that the oblique viewing angles obfuscate the ruts directly to the rear of the vehicle (Fig. 3). This is exacerbated by the necessary blind-angle correction used to remove the returns reflected from the HMMWV itself. The relatively shallow ruts evaluated during the 12/18/19 and 01/09/2020 tests allowed for the adjacent ruts along a given traverse to be captured with better accuracy under the current experimental design. This is made apparent in Fig. 3 as the first set of ruts are shallower than in subsequent passes, where these first set of ruts are more accurately represented in the later passes. While these results appear promising for shallow snow depths, deeper snowpacks will likely yield less accurate results due to the occlusion of the bottom of the ruts by the taller sidewalls of the ruts. Even with the relatively shallow snow pack, the LiDAR measured ruts are still shallower than the true depths due to obstruction of the bottoms of the ruts from the sidewall.

Due to the aforementioned problems related to viewing angle, future data collections will be conducted with Unmanned Aerial Systems (UAS) based LiDAR collections over the test site after the vehicle traverses in order to obtain more optimum viewing angles. Collections with a UAS will likely result in more accurate measurements of the ruts since the overhead view angle will be less susceptible to issues associated with oblique returns and allow for longer collection times resulting in a denser point cloud from which to derive height values. Despite the issues related to the mounting of the LiDAR, the height maps generated for these collections have demonstrated that the approach presented here can generate accurate height maps for vehicle ruts in a snow-covered environment.
CONCLUSION

The initial object detection and quantification framework presented here has demonstrated the ability to identify vehicle ruts and provide an estimate of depth across the length of the ruts. While the algorithm was evaluated on LiDAR-derived DEMs, the algorithm itself is applicable to DEMs derived from photogrammetry provided the raster dataset has sufficient spatial resolution to represent potential obstacles. Future data collection operations will seek to optimize LiDAR sensor mounting, likely producing improved results. The object detection algorithm will also be expanded in upcoming analysis to provide probability of detection to better quantify the certainty of detection.

ACKNOWLEDGEMENTS

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REFERENCES

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Early Warning Frost Detection System

Rob Clark, P.E., M.ASCE\(^1\); and Doug Burghart\(^2\)

\(^1\)Geotechnical Services Engineer, Geokon LLC, Lebanon, NH, U.S. E-mail: rclark@geokon.com
\(^2\)Interim Engineering Manager and Project Engineer, Bridges and Hydraulic Section, Whatcom County Public Works, Bellingham, WA, U.S. E-mail: dburghar@co.whatcom.wa.us

ABSTRACT

Occurrence of freeze and thaw conditions in rural roadway subgrades throughout the years resulted in significant labor efforts from Whatcom County Public Works engineers to monitor and evaluate conditions for implementing road restrictions. Extended periods of frost conditions followed by warmer temperatures can result in extensive damage to the road system if restrictions are not applied. Previously, subsurface temperatures were measured manually at various locations throughout the county to assist with evaluation of roadway conditions. Recently, a network of automated measurement and remote communication systems were designed and implemented to facilitate improved monitoring and response for the County engineers. Sixteen remote monitoring locations were selected throughout the county and at each site a 1-meter long tube with 18 thermistors at 50 mm spacing was installed into the roadway subgrade. Additional instrumentation at each site included ambient air temperature sensors and moisture sensors for the datalogger enclosures. Data is collected and transmitted to a web-based data management system for County personnel to access, and to provide alarm notifications to County personnel with indications as to when temperature thresholds are exceeded. Having the automated system allows the County to monitor the thaw process more accurately and be more confident with respect to when road restrictions are applied and their duration. The current monitoring system has increased the effectiveness and efficiency of the County’s rural roadway management process during freeze/thaw cycles, resulting in significant savings in operating and maintenance costs.

INTRODUCTION

In the upper northwest corner of the continental United States, Washington state’s Whatcom County covers an area of approximately 6,475 square kilometers. The county is bordered by our Canadian friends to the north, as shown in Figure 1, neighboring with Metro Vancouver Regional District and Fraser Valley Regional District in Canada.

The County maintains over 1,560 kilometers of roadway infrastructure by preventing, reducing, or restoring the deterioration of the road system. The goal of the public works department is to perform these services in an efficient, safe, and cost-effective manner (Whatcom County Public Works, 2020).

Weather in this portion of Washington State is heavily influenced by winds from the Canadian Frasier valley to north and the onshore flows from the Pacific Ocean to the west. Winter temperatures will often stay well below freezing for days, even weeks, and then occasionally warm up resulting in periods of thawing. It is not uncommon for numerous freeze and thaw cycles to occur throughout the winter.

Prior to the 2019-2020 winter season, the County used manual monitoring methods to evaluate road conditions, and which provided them with information that was used to restrict traffic or close certain roadway sections. This was particularly important during freeze and thaw periods, where the roads can be more susceptible to damage. Most of the rural roads are constructed on granular
sub-base material that is intended not only to support the roadway but provide a drainage path for any water that may get into the roadway structure. When water gets trapped in this subbase and then freezes, ice or frost will develop, expanding within the subbase material. From the force of the expansion, material can heave upwards, deforming the pavement of the roadway in the process. As temperatures rise, the ice within the formation will thaw, weakening the subbase and leaving gaps or voids where the ice had formed. Heavy loads across the roadway surfaces can then damage the pavement, often causing potholes to develop.

Fig. 1 – Whatcom County, Washington State, United States

In addition to visual observations, the County also used thermistors that had been installed in the mid-1980s by Whatcom County Maintenance and Operations personnel. These thermistors were single point instruments, with up to three thermistors at various depths within the roadway subgrade at each monitoring location. Measurements of the thermistors required accessing monuments on the roadways and connecting to each thermistor to take a reading. Many of the thermistors had stopped working throughout the years and, as such, the monitoring coverage had diminished over time.

To minimize expenditures but maintain a means of monitoring the rural roadways around the county, the Public Works department investigated an alternative to their current manual operation by considering automated subsurface temperature monitoring systems. It was understood that initial capital costs would be more than annual expenses associated with this monitoring program, but that the labor associated with the current manual monitoring program would eclipse the cost of this automated monitoring system after just a couple of years. Perhaps even more important, Whatcom County weighed the potential maintenance and repair costs if its roadways sustained damage from freeze / thaw cycles, in addition to the economic impacts of road restrictions on businesses using Whatcom County roadways.

DESIGN AND PROCUREMENT

After the County performed research on various types of subsurface temperature measurement and remote data monitoring systems, they determined the need for pursuing this type of system. The desire was to have a means of measuring temperature at 50 mm spacing to a depth of about 1 meter below the surface of the roadway. This would allow Public Works engineers to monitor the
depths at which freezing ground layers were occurring in addition to how the temperatures varied with depth. Sixteen sites around the county, shown on Figure 2, were identified as the first locations that would be incorporated into this automated monitoring program. The sites were selected based on prior thermistor locations and to geographically represent the bulk of the roadway system to obtain a broad picture of roadbed temperature trends during freeze/thaw cycles.

![Figure 2 - Sixteen monitoring locations within Whatcom County](image)

With a preliminary design in hand, the County issued a request for qualifications for the design and procurement of an automated roadway subsurface temperature monitoring system. After review of the submitted proposals, the County selected Geokon, of Lebanon, New Hampshire, to provide the required monitoring system.

With the knowledge of the requirements put forth by the County, Geokon worked on modifying some of their existing thermistor designs (Geokon Model 3810A) to come up with a probe which consisted of 18 closely spaced addressable digital thermistors mounted within a 25 mm diameter PVC pipe. The 18 thermistor nodes were fully encapsulated in epoxy at 50 mm spacing within a 1-meter long length of Schedule 80 PVC. The ends of the pipe were capped and sealed, with a digital RS-485 signal cable exiting the top of the probe assembly (as shown in Figure 3).
Due to the distance and topography between each of these locations, an individual data acquisition system allowing for remote data collection was designed for each location. The data acquisition system consisted of a Campbell Scientific CR300 with a built-in cellular modem (CR300-CELL210) and an RS-485 Multidrop Interface (CSI MD485). The system was powered by a 100 Ah 12 VDC deep cycle marine battery charged by a 20 Watt regulated solar panel.

Fig. 3 – Geokon Model 3810A Modified Addressable Thermistor String

Fig. 4 – Drilling hole for thermistor string
INSTALLATION

In the summer of 2019, the County awarded a contract to Razz Construction, Inc. of Bellingham, WA for the installation of the sixteen automated monitoring systems. Razz was responsible for traffic control, utility clearances, and the efforts associated with the installation of each monitoring system. The County provided construction oversight and Geokon assisted with the first installations.

Each thermistor string was installed into a 100-mm diameter auger hole drilled just outside the fog line on the shoulder of the road at each of the sixteen locations. The holes were drilled, as shown in Figure 4, to a depth of a little over 1 meter. A 6-m-long x 450-mm-deep trench was then excavated from the hole, perpendicular and off to the side of the roadway to where the data acquisition system was installed. The data acquisition system enclosures were attached to a 50-mm diameter galvanized steel pole that was concreted into the ground.

The thermistor string was placed within each drilled hole and backfilled with well graded sand. The top of the instrument was set approximately 50 mm below ground surface and the signal cable from the instrument was then routed through liquid tight flexible metal conduit along the bottom of the trench into the data acquisition enclosure. The trench was backfilled and compacted with excavated materials. Finally, a 100-mm diameter roadway monument was installed around the top of the thermistor string and cemented in place.

Two lockable enclosures were mounted on the 50-mm diameter galvanized steel pole – one housed the datalogger and associated accessories, while the other housed the 12 VDC battery. A solar panel was mounted near the top of the pole which was then installed near the County’s Right of Way, away from the ditch-line and snowplow areas. Images of two of the datalogger installations are shown in Figure 5.

Fig. 5 – Remote Data Acquisition Installations
After the Contractor completed the installation of all sixteen sites, Geokon and County representatives visited each site to confirm instrument connections, test measurements, and to initiate remote communication through the cellular modems. Each site was programmed to take measurements at 30-minute intervals.

Fig. 6 – VDV data windows. Top showing output for all sites at 50 mm below ground surface, and bottom showing dashboard view with Site 11 real-time data displayed.
SYSTEM INTEGRATION

Near real-time monitoring of the thermistor measurements was provided to the County using a web-based data management system from Vista Data Vision (VDV). Geokon worked with the County to optimize the configuration of the VDV system that would be most beneficial for them. This included determining threshold limits for alarm notifications, data plot layouts, web-site dashboard layout, and automated report content.

The effectiveness of these systems required near real-time notifications be sent to select County personnel indicating changes in temperatures at the sites which would be conducive to development of frost heave and thaw conditions. These notifications were triggered when temperature thresholds were reached and would be sent to the designated recipients via email in each instance.

Plots of the data included time versus temperature for each of the thermistor depths at each of the monitoring sites, temperature profile plots for each site, and system status plots (datalogger battery, enclosure humidity levels, and datalogger temperatures). VDV provided near real-time status of each of the sites on the dashboard of the visualization platform, letting users get a quick look at conditions and help evaluate roadbed temperatures across the county more effectively. Examples of the plotted data and dashboard image are shown in Figure 6.

Users of VDV can view, modify, or develop plots of the measurements acquired from each of the monitoring sites, depending on the level of access privileges available to them (Vista Data Vision, 2020). Data is automatically updated at 30-minute intervals and, if a site does not provide updated data over two scheduled monitoring periods, then a notification is sent to the administrator of the VDV site.

MONITORING

The intent of this system is to monitor the frost and thaw cycles of the roadway subgrade at the sixteen designated locations throughout the county. As this is only a concern for a few months out of the year, the system is operational from December through March, after which it is placed in standby mode until the next season. During the non-monitoring periods, the batteries are removed and placed in storage on trickle charge. The services for VDV and the cellular modems are put on hold during the non-monitoring periods and are reinstated where they left off at the next winter monitoring interval.

Monitoring of the sixteen sites this past year began in mid-December 2019 through the end of March 2020. During this time, the system provided extensive and timely information to the County engineers of the subsurface temperatures at each site. The engineers were able to determine depths to which freezing conditions were being observed and the duration over which the soils were subjected to freezing conditions. This, along with weather forecasts, provided the information necessary for determining if roadway travel restrictions were necessary.

As with any remote automated system, there are often problems that arise that require additional support. Fortunately, each of the systems worked very effectively throughout the first monitoring season, and all thermistors provided consistent and reliable data. The only issues were periodic lapses in communication from a couple of the more remote locations within the county, where cellular service was spotty, resulting in a lag of data on those occasions. For the current monitoring season (2020 – 2021 winter), stronger cellular antennas have been installed at the problematic locations. Two months into the second monitoring season have shown all thermistors and monitoring systems at each of the sixteen locations to be working as intended.
Maintenance of the systems will include periodic replacement of desiccant packets within the enclosures, and the replacement of batteries every 3 or 4 years, depending on how they are stored in the off-season and how well they maintain a charge. The only other maintenance that may be required, would be the result of curious or mischievous animals (or humans!). Considering the remote installation locations, the datalogger components have thus far been fortunate to avoid any vandalism or thefts (of solar panels or enclosures).

CONCLUSION

With the automated monitoring systems and near-real time availability of temperature data from the multiple sites around the county, engineers and field support personnel at Whatcom County have significantly reduced the labor hours that had previously been committed to evaluating the roadway conditions. The costs associated with the installation of this monitoring program will be recouped in a couple of years when compared with the effort previously put forth in getting a portion of the information now available at their fingertips. The system will provide more accurate and real-time data, which will help prevent unnecessary road restrictions in the future. Having better indicators for when road restrictions are necessary will be beneficial for not only the County (with roadway wear and tear) but will likely provide positive economic impacts to businesses that are affected by road restrictions.

In addition to the cost savings that will be realized, the consistency and reliability of the data provided, and the accessibility of the data, there is also the benefit of a significant improvement in safety. With the monitoring system in place, there is no longer a need for county personnel to be out monitoring remote sections of roads at 4 or 5 in the morning, taking manual readings of thermistors installed within the roadbed (as was the case previously). Any service or maintenance on these systems will typically be limited to data acquisition components, located well away from the edge of the roadway.

The County is now considering incorporating weather monitoring systems into the dataloggers at each of the existing sites. This would include rain measurements, humidity, and possibly wind measurements, to compliment the ambient air and roadway subgrade temperature measurements. With these additional measurements, the monitoring system can be left in operation throughout the year to provide not only the Public Works data for the roadway, but meteorological information to other departments in various sections of the county.

REFERENCES


Deformation Caused by Frost Heave on a Rock Slope of Mudstone

Otgonjargal Dagvadorj1; Dai Nakamura, Ph.D.2; Takayuki Kawaguchi, Ph.D.3; and Shunzo Kawajiri, Ph.D.4

1Doctor Course of Cold Regions Engineering, Environmental and Energy Engineering, Kitami Institute of Technology, Hokkaido, Japan. E-mail: d1971408013@std.kitami-it.ac.jp
2Division of Civil and Environmental Engineering, Kitami Institute of Technology, Hokkaido, Japan. E-mail: dnaka@mail.kitami-it.ac.jp
3Division of Civil and Environmental Engineering, Kitami Institute of Technology, Hokkaido, Japan. E-mail: kawa@mail.kitami-it.ac.jp
4Division of Civil and Environmental Engineering, Kitami Institute of Technology, Hokkaido, Japan. E-mail: skawajiri@mail.kitami-it.ac.jp

ABSTRACT

In this study, we measured freezing depth, amount of frost heave, and weathering depth of a mudstone slope where damage had occurred. In addition, rocks collected from the survey field were subjected to a frost heave test and a slaking test. The results of the laboratory tests show that the rocks collected at the survey field were easy to slake and had high frost susceptibility. The field survey showed that the rock slope frost heaves significantly in winter and that the surface layer of the slope thaws in spring and becomes extremely weak. Furthermore, the weathering depth of the rock slope was found to be in good agreement with the freezing depth. On the other hand, the weathering depth of the rock slope did not change during the summer, and no slaking was observed due to repeated wetting and drying.

INTRODUCTION

Hokkaido, the northernmost island in Japan, suffers from frequent frost damage due to its cold climate (e.g. Koyama and Sasaki 1967). As has also been reported, civil engineering structures such as tunnels, roads, and slope protection works can be deformed by the frost heave of rock. Therefore, an important task is to understand the frost susceptibility of bedrock in Hokkaido, Japan. Especially, the occurrence of freezing phenomenon on a rock slope may lead to a major disaster such as a bedrock collapse, and therefore, it has been attracting a lot of attention (Japanese Geotechnical Society, Hokkaido Branch 2016).

With this background, we have been investigating the deformation of rock slopes. In this study, we measured the freezing depth, the amount of frost heave, and the weathering depth of a mudstone slope where damage had occurred. In addition, rocks collected from the field were subjected to a frost heave test and a slaking test.

OVERVIEW OF THE SURVEYED AREA AND DAMAGE SITUATION

The survey field is a road slope in eastern Hokkaido, Japan. The gradient of the road slope was 45 degrees at the time of construction.

Figure 1 shows a view of the survey field. Around the survey field, surface collapses were common. Also, slipped vegetation and sediment by weathering of the slope were observed. In addition, the gabions used for slope protection were found to be deformed in several places.

Figure 1 shows the deformation of a gabion. In Hokkaido, gabion work is frequently used as a countermeasure against frost heave. A gabion is a cage, a box filled with rocks, for use in civil engineering structures.
engineering. A gabion constructed in Hokkaido is lifted up by frost heave in winter. The ground melts and settles in the spring, but the rocks in the cage move downward. As this movement is repeated, the gabion is deformed as shown in the figure. From the figure, it can be seen that the top of the cage is greatly deformed, as shown by the white dotted line. In recent years, this kind of deformation of gabion has been observed in some cases where a long period of time has passed since their construction.

Figure 1. View of the survey field.

BASIC PHYSICAL PROPERTIES OF ROCKS COLLECTED FROM THE SURVEY FIELD

Figure 2 shows the rocks collected from the survey area. The type of rock is dark gray mudstone; Table 1 summarizes the basic physical properties. In this survey, rock samples were collected by block sampling and basic physical properties were measured. The strength tests were carried out in accordance with the American Society for Testing and Materials Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens (ASTM/D2938-95) and Method for Splitting Tensile Strength of Intact Rock Core Specimens (ASTM/D3967-16). Table 1 clearly indicates that mudstone is a soft rock with high water absorption, high porosity, and low strength. In addition, the strength of the mudstone in the wet condition is extremely low compared to its strength in the dry condition.

SLAKING TEST

The slaking test was carried out in accordance with the American Society for Testing and Materials Standard Test Method for Slake Durability of Shales and Other Similar Weak Rocks (ASTM/ D4644-16)

Specimens: The specimens were sampled and trimmed into a column with dimensions of 55 mm in diameter and 20 mm in length, and dried in a drying furnace for 24 hours.

Test Procedure: The dried specimens were placed in a container filled with distilled water. Periodically, the specimens were photographed. Simultaneously, X-ray CT scans were performed to observe the interiors of the specimens. The X-ray CT scanner used in this study was model inspeXio SMX-225CT manufactured by Shimadzu Corporation (Kyoto, Japan). The container was fixed on the CT stage in the X-ray CT scanner. X-rays were generated at 150 kV and 50 μA, and the imaging configuration yielded a voxel size of 98 μm. The details of the X-ray CT scans performed in this study are described in Song et al (2017).

Test Results: Figure 3 shows the changes of rocks over time in a slaking test. Cracking occurred in the specimen immediately after the start of the test, and after 2 hours the specimen was
divided into several rock fragments. These test results suggest that the mudstone is a rock that is susceptible to slaking due to repeated wet and dry effects.

![Figure 2. Rock collected from the survey area.](image)

<table>
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<td>Wet state</td>
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![Figure 3. Changes of rocks over time in a slaking test.](image)
FROST HEAVE TEST

Specimens: The specimens were sampled and trimmed into a column with dimensions of 55 mm in diameter and 70 mm in length. The specimens were soaked in distilled water for 1 weeks.

Test Procedure: Initially, the inside temperature of the specimen was adjusted to +5°C by setting the temperature of the top and bottom plates at +5°C. Afterwards, the temperature of the bottom plate was stabilized at +5°C, and the temperature of the top plate was stabilized at −15°C. Therefore, freezing progressed from the upper surface of the specimen. The details of the frost heave test performed in this study are described in Nakamura et al (2009).
Test Results: Figure 5 shows the appearance of the specimen after the frost heave test. A thick ice lens could be observed on the lower part of the specimen. Figure 6 shows the changes in the temperatures of the top and bottom plates of the test equipment and the frost heave amount over time. The maximum frost heave rate of the mudstone was very high (1.31 mm/h), partially because of its low tensile strength (0.04 MPa) under wet conditions. These test results are considered to be reasonable with respect to the actual disaster situation.
OVERVIEW OF FIELD SURVEY

Figure 7 shows the cutting of a rock slope using a backhoe. In this study, in advance of field measurements we removed the slipped vegetation and the sediment-covered bedrock to a depth of 0.5 m, with a slope length of 6.3 m and a width of 14.4 m. This was done to understand the frost heave behavior of the fresh (un-weathered) bedrock immediately after slope cutting. We also aimed to determine the depth of weathering of rock slopes by undergoing a season of freeze-thaw history. The angle of the slope after cutting was 40 degrees.

Figure 8 shows a front view of the survey field and its photograph. In this study, two cases were constructed in order to clarify the effect of freeze-thaw history on deformation of the area. One case (Case A) is the original cut slope and the other case (Case B) is a cut slope covered with an insulation material of 100 mm thickness. In the following, case A is described as no-insulation and case B as insulation. Figure 8 also shows the layout of various measuring instruments. The insulation was covered with non-woven fabric to prevent degradation by UV light.

Figure 9 show schematic diagrams of the various measuring instruments installed at the survey field. The freezing depth, frost heave amount, and soil moisture were measured automatically once per hour during the survey period. Furthermore, a fixed-point camera was used to photograph the
cut slope.

**Figure 9. Schematic diagram of the various measuring instruments.**

**Freezing Depth:** The freezing depth on the slope was measured using a temperature measurement rod. The temperature measurement rod was inserted into a borehole formed by drilling a slope vertically. The temperature measurement rod was made by fixing temperature sensors to the wood. The temperatures sensors were placed on the ground surface and at depths of 20 cm and 60 cm from the ground surface. In both cases, the temperature measurement rods were placed at two locations, on the upper and lower slopes.

**Frost Heave Amount:** The frost heave amount on the slope was measured with a displacement meter. An iron rod (16 mm in diameter) was inserted into a borehole formed by drilling a vertical slope. The bottom end of the iron bar was fixed with mortar to make it a fixed point. A displacement meter was attached to this bar to measure the vertical movement of the rock surface.

**Soil Moisture:** Soil moisture was measured using an FDR-type sensor. The sensors were inserted perpendicularly into the slope after digging into the slope to a depth of 10 cm. In Case A, the soil moisture sensors were installed at two locations. However, data could not be collected due to the failure of one sensor.

**Weathering Depth:** The weathering depth of the rock slope was measured by conducting a cone penetration test in the survey field. The cone is penetrated vertically into the slope.

**MEASUREMENT RESULTS AND DISCUSSION**

Figure 10 shows the changes of snow depth, daily rainfall, temperature, volumetric water contents, and frost heave amount of rock slopes.

**Figure 10. Changes of snow depth, daily rainfall, temperature, volumetric water contents, and frost heave amount of rock slopes.**
contents, and frost heave amount of rock slopes. In this figure, we used data on snow depth and daily rainfall measured and published by the JMA (Japan Meteorological Agency). Snowfall was very low in 2018, when the survey began, and snow depth was less than 30 cm throughout the season. Therefore, the rock slope was not at all affected by the insulation effect of the snow. The rock slope was directly affected by the freeze. The temperature was below 0°C from the end of November, and the frost heave amount was measured at the same time.

In this study, a displacement meter capable of measuring up to 50 mm was installed. But the target rock mass had very high frost susceptibility, to the extent that the frost heave amount reached the measurement limit on December 18 and was impossible to measure thereafter. The frost heave amount on December 18 was 42.4 mm, and the frost heave velocity was 3.47 mm/day.

Figure 11 shows the appearances of frost heave on the rock slope taken at survey field on March 6, 2019. Figure 11(a) is an ice lens observed in the surface layer of Case A (no-insulation), and an ice lens up to 7 cm in thickness can be seen at a depth of 3 cm from the rock slope. Figure 11(b) shows ice lenses observed on a slope adjacent to the target rock slope. We excavated the upper part of the slope to a depth of 30 cm, and observed thickly developed ice lenses and cracks.

Figure 11. Appearances of frost heave on the rock slope taken at survey field on March 6.

Figure 12 shows the changes of the freezing depth in Case A (no-insulation) and Case B (insulation). The freezing depth is the position of 0°C as calculated by proportionally distributing the measurement results obtained by multiple temperature sensors. The maximum freezing depth was 58 cm from the ground surface in the upper part of Case A (no-insulation) and 23 cm from the ground surface in the lower part of Case B (insulation). The maximum freezing depth of Case B is much smaller than that of Case A, because of the installation of insulation material.
Figure 12. Changes of the freezing depth in Case A (no-insulation) and Case B (insulation).

Figure 13. Appearances of the rock slope taken by a fixed-point camera.

Figure 13 shows the appearances of the rock slope taken by a fixed-point camera. On November 28, 2018, approximately one month after the cuttings, the surface of rock slope was still in a good condition. In the severe winter season on February 3, 2019, the rock slope was lifted up by the frost heave. On March 26, during the melting period, the observations can confirm that the surface layer of the rock slope had melted, turned into sediment, and collapsed. On May 6, in the spring, the surface water generated by rainfall was found to have caused gully erosion on the rock slopes.

Figure 14 shows examples of a cone penetration test results conducted to determine the weathering depth of a rock slope. In this study, 4 to 6 penetration tests were carried out in each survey, and the maximum penetration depth was recorded in each test. The freezing depths at the lower of Case A (no-insulation) and Case B (insulation) are also shown in the figure. Mudstone is a soft rock with low strength, but penetration resistance was high in the unfrozen state, and the
cones penetrated only 4 cm at most. However, in both Case A (no-insulation) and Case B (insulation), the rock clearly becomes sediment due to freeze-thaw history, and the maximum penetration depth became obviously deep. Focusing on the April 17, 2019 results, the maximum penetration depth is almost coincident to the maximum freeze depth in each case, which indicates that the freeze-thaw history has a very significant effect on the weathering of the rock slopes. In addition, after May 18, the penetration depth does not become deeper, and it seems that the effect of slaking that may occur in the summer is small.

The volumetric water contents of the rock slope have remained high except in winter (see Fig. 10), which suggests that the slope was not affected by slaking.

**CONCLUSION**

In this study, we reported on a case of frost heave deformation of a rock slope composed of mudstone.

Various laboratory test results show that the mudstone collected from the rock slope was a soft rock with high water absorption and porosity and low strength. The mudstone was also found to be a rock that was susceptible to slaking due to repeated wet and dry effects. In addition, the rock was found to have extremely high frost susceptibility.

The field survey results confirmed that the frost heave phenomenon occurred in the rock slope. We were able to observe thickly developed ice lenses. In addition, cone penetration tests were conducted before and after freezing and thawing, and the maximum penetration depth was found to almost coincide to the maximum freeze depth. These results indicate that the freeze-thaw history has a very significant influence on the weathering of rock slopes.

We will continue to conduct more surveys and accumulate information on the damage caused by the frost heave of rock slopes.

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REFERENCES

Use of a Portable Friction Tester on Snow and Ice Pavement

Amelia Menke\textsuperscript{1}; Sally Shoop, Ph.D., P.E., M.ASCE\textsuperscript{2}; and Bruce Elder\textsuperscript{3}

\textsuperscript{1}U.S. Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Hanover, NH. E-mail: menke.amelia@gmail.com
\textsuperscript{2}U.S. Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Hanover, NH. E-mail: sally.a.shoop@erdc.dren.mil
\textsuperscript{3}U.S. Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Hanover, NH. E-mail: Bruce.C.Elder@usace.army.mil

ABSTRACT

The objective of this project was to determine if portable friction testers could be used for friction measurements on compacted snow and ice surfaces. First, the effect of cold temperatures on the operation, consistency, and accuracy of commercially available portable pavement friction measuring tools was evaluated. Tests entailed a series of experiments in a controlled cold room environment. Two portable fixed slip continuous measurement devices and one deceleration spot measurement device were evaluated. The controlled temperature testing determined how ambient temperature and duration of exposure can affect results, but that with care, the devices could be operated in conditions as cold as -25°C. This was followed by using one of the devices on outdoor testing on snow, ice, and asphalt surfaces and compared the portable tester to the well-known SAAB vehicle runway friction tester. Results showed good agreement between the portable tester and the SAAB Friction tester, providing validation for the operational use of a portable tester on frozen surfaces.

INTRODUCTION

A variety of commercially available devices exist to characterize the friction, or skid resistance, of pavement surfaces. Friction testing is often performed in conjunction with construction activities as a means of quality control and assurance in meeting design specifications. In temperate regions, these tests usually occur during the summer construction season. Pavement friction testing is also performed throughout the year, most notably to monitor the condition of runways. Testing during the winter season can present unique challenges. A previous study of friction measuring equipment under winter conditions found that “correlations of ground vehicles on wet pavements do not apply to vehicles on surfaces covered by ice and snow” and recommended further investigation into the influence of temperature (ASTM 2009).

This study was conducted as part of a larger program assessing test equipment for characterizing the mechanical properties of snow and ice for vehicle mobility. The friction testing equipment was evaluated with an eye to potential use in characterizing pavements constructed solely of snow or ice. The three devices evaluated in this study were selected for their portability, in recognition of the need for easily-deployed equipment at remote field locations where such pavements are more likely to exist. The two Continuous Friction Measuring Equipment (CFME) systems evaluated in this study are the Findlay Irvine Micro Griptester (mGT) and the T2Go from SARSYS-ASFT. Continuous friction measuring equipment is often vehicle or trailer mounted, but these portable systems can be hand carried and operated. The deceleration spot measurement device, the Dynamic Friction Tester (DFT) from Nippo Sangyo, is relatively compact and can be powered by a vehicle battery.
Test values of pavement friction are known to vary with the type of testing device used, as well as with the speed at which the testing is performed. For example, the Federal Aviation Administration (FAA) requires a minimum reading of 0.43 when using the Findlay Irvine Griptester, and 0.50 when using the Airport Surface Friction Tester at 40 mph; and the minimum readings required for safe airfield operation using these devices are 0.24 and 0.34, respectively, when testing at 60 mph (FAA 1997). The Griptester and Airport Surface Friction Tester are each larger versions of the two CFME systems evaluated in this study - the Findlay Irvine Micro Griptester (mGT) and the T2Go from SARSYS-ASFT.

Though correlations may exist, previous studies have shown that friction numbers are notably “instrument specific” (Norheim et al. 2001). Thus, we made no attempt to compare test results (friction numbers) between devices until field validation. Friction is traditionally defined as the ratio of normal load to horizontal force developed at the interface between the moving and the static materials. Therefore, friction numbers are essentially dimensionless.

FRICITION TESTING EQUIPMENT

Dynamic Friction Tester (DFT): The DFT (Figure 1) is a portable instrument used to measure the frictional characteristics of paved surfaces. The system includes several components: the apparatus that engages with the pavement, a controller unit, a water supply tank, and a laptop computer. The test apparatus is approximately 0.4 m by 0.5 m with a spinning disk in the base. When the disk reaches the desired rotational speed, which can be programmed from 20 to 100 km/h, a weight is released onto the disk. Three rubber sliders 6 mm by 16 mm by 20 mm on the base of the disk come into contact with the test surface, and the torque generated by the sliding forces measured during the spin down is then used to calculate the friction as a function of speed (Nippo 2005).

![Figure 1. The DFT test apparatus (right) with controller unit (left) in the cold room.](image-url)

The normal test procedure requires application of water to the surface being tested. Since the test objective temperatures are all below freezing, we conducted most of the tests dry. The wet method was used only during baseline (warm) testing at 25 °C. For the remaining tests, the water reservoir provided with the system was simply not used, which did not impede the system from operating normally. To operate the DFT with liquid water in an extremely cold environment presents several challenges. Most obviously, water can freeze in the hoses and valves potentially
damaging the device. Also, the addition of warm water to the frozen pavement surface may alter the surface properties in a manner not representative of a moving vehicle. This is particularly true for a pavement surface of snow or ice. The effect of the water on the pavement surface, and thus the friction measured by the device, may also be sensitive to the temperature of the water being used, which would create more variables between tests.

At each temperature and exposure period, we recorded five repetitions of the DFT test. All tests used an initial rotational speed of 40 km/h. Higher speeds were not attempted to minimize the possibility of damaging the equipment under the stress of extreme cold. New rubber sliders were installed prior to testing at each temperature point in order to prevent changes in performance due to wear.

**Micro Grip Tester (mGT):** The mGT (Figure 2) is a continuous friction measuring device designed for use on a dry or wet surface. It is manually pushed with a three wheel design: two drive wheels in the rear and a smooth-tread measuring wheel mounted on an instrumented axle in front. The mGT is a battery powered and self-contained device which measures friction by the braked wheel, fixed slip principle (Findlay 2018). Tests can be completed by one trained operator.

![The Micro Grip Tester (mGT).](image)

During testing, speed is controlled by the operator, who simply pushes the device forward at a steady pace as they walk behind. The mGT includes a function allowing the operator to monitor their speed in real time on the device’s handle-mounted control screen. The target test speed can be selected, and the device will beep to alert the operator when they are out of range. Our testing was performed with a target speed of 0.9 m/s.

Knowing that tire pressure changes with temperature, we verified that all tires were inflated to the manufacturer’s recommended pressure prior to the start of testing at each temperature point. The tires were also cleaned to remove any grit or debris prior to testing. Once introduced to the cold room environment, tire pressure was not adjusted. The potential change in tire pressure is one aspect of the effect of cold on equipment accuracy.
At least four runs were conducted at each time and temperature interval, with the average taken from results of the most consistent three runs. The accuracy of ancillary features such as the onboard temperature sensor and GPS were not evaluated for this study.

**T2Go Portable Continuous Friction Measuring Equipment:** The T2Go, like the mGT, is a portable continuous friction measuring device. It features two in-line wheels, the reference wheel behind with the slip wheel in front. It is designed to operate in wet or dry conditions, and an optional water system is available. Most aspects of the test are controlled through a separate computer tablet loaded with the appropriate software. The T2Go provides several additional features such as temperature and humidity sensors and GPS tracking which were not evaluated in this study. The system can be used by one operator, but two are preferable due to the difficulty of using the tablet and test device simultaneously. Speed is controlled by the operator, and may be monitored in real time on the tablet.

As with the mGT, each tire was cleaned and checked to verify inflation to the manufacturer’s recommended pressure prior to the start of testing at each temperature point. The target speed used during testing was the same as for the mGT: 0.9 m/s. Four test runs were conducted at each time and temperature interval, with the average taken from results of the most consistent three runs.

![Figure 3. The T2Go and UHMW polyethylene test surface in the cold room.](image)

**LABORATORY TESTING TO GUIDE COLD TEMPERATURE OPERATIONS**

The laboratory cold room testing was designed to evaluate the variability in friction test results and equipment performance due to duration of cold exposure. Each friction tester was subjected to temperatures of 0 °C to -25 °C for periods of up to 24 hours prior to being engaged in a standardized friction test. All tests were performed in a cold room on a sheet of ultra-high molecular weight (UHMW) polyethylene 9.525 mm thick and 0.76 m wide by 1.22 m long (Figure 3). This surface material was seated on a rubber mat approximately 0.9 m by 2.4 m. The role of the test material was not to replicate a specific pavement surface, but to provide a robust and inert surface that would offer consistent and repeatable surface properties throughout the range of testing temperatures.

Testing at ambient temperature, about 25 °C, was performed first to establish a baseline “warm” reading for comparison. During cold testing, each device was brought into the cold room...
environment from warm storage and a series of tests was conducted immediately; the test procedure was performed again after 30 minutes, 1 hour, 90 minutes and 2 hours of exposure to the cold. The friction-measuring devices were then left in the cold room to fully acclimate and a final test was performed after 24 hours of exposure to the test temperature. All detachable electronic components such as laptops and tablets were removed from the cold room between the short-term testing and the 24-hour cold soak tests.

The full series of exposure testing was performed at each of four temperature points (not all of the devices were capable of operating at some of the test temperatures). The cold room environment was adjusted to nominal temperatures of 0 °C, -10 °C, -20 °C, and -25 °C and allowed to stabilize at least 12 hours before testing. Actual air temperatures varied by ±1 °C during the course of testing.

COLD ROOM TEST RESULTS

**Dynamic Friction Tester (DFT):** The DFT records friction data at every 0.1 km/h interval from the initial rotational speed selected until it slows to a stop. The friction numbers recorded at 20 km/h, 40 km/h, 60 km/h, and 80 km/h are typically reported, when available (Nippo 2005). For our analysis, the readings taken at 20 km/h are used. The average coefficient of friction measured at 20 km/h in warm conditions was 1.016 using the dry method and 0.007 using the wet method. This difference between the wet and dry method results is extreme, and serves to illustrate the impact of test method and surface water on measuring and interpreting pavement friction.

Table 1 shows the average of five runs performed at each temperature point and time interval. These results reveal a consistent trend towards lower friction readings after longer periods of exposure. There is a significant drop in the friction numbers between 0 °C and -10 °C. However, at colder temperatures the results rose slightly from -10 °C levels with the exception of the 24 hour exposure period.

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<td>0.747</td>
<td>0.633</td>
<td>0.572</td>
<td>0.557</td>
<td>0.544</td>
<td>0.259</td>
</tr>
</tbody>
</table>

The effect of temperature on DFT results is significant. Tests performed within 30 minutes of exposure to 0 °C were very similar to baseline results, as shown in Figure 4, while all other tests showed significantly lower friction values. At temperatures below 0 °C, the drop in friction values was more dramatic.

A potentially significant influence on friction measurement with the DFT device is the change in stiffness of the rubber sliders on the base of the DFT. Being the points of contact with the target surface, changes in the properties of the sliders with temperature are likely to impact the measurement results.
Figure 4. DFT test results after exposure to 0 °C compared to 25 °C baseline.

Other components of the DFT appeared to operate as expected with limited cold exposure. However, the controller screen became unreadable after 24 hours exposure to temperatures at or below -20 °C. Nonetheless, a meaningful test could still be performed using the laptop. Additionally, we noted that the DFT’s cables are more difficult to connect and disconnect at cold temperatures, but this is very common in all manner of equipment.

Although the components of the DFT functioned adequately after exposure to a cold environment, its usefulness may be limited by the prescribed use of water in the standard test procedure. Significant differences were apparent in measured friction between wet and dry testing. While the system functioned well without use of the water reservoir, it is unclear if results from the dry test method have practical use. However, the values obtained during dry-method testing may be useful in assessing pavements, in spite of their failure to correlate to wet-method results. The ASTM Standard E1911 states that “the values measured in accordance with this method do not necessarily agree or directly correlate with those obtained utilizing other methods of determining friction properties or skid resistance” (ASTM 2009). The dry test method could be useful if a standard for interpretation of the results were established.

<table>
<thead>
<tr>
<th>Temperature</th>
<th>0 min</th>
<th>30 min</th>
<th>1 h</th>
<th>90 min</th>
<th>2 h</th>
<th>24 h</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 °C</td>
<td>0.28</td>
<td>0.29</td>
<td>0.32</td>
<td>0.29</td>
<td>0.26</td>
<td>0.28</td>
</tr>
<tr>
<td>-10 °C</td>
<td>0.30</td>
<td>0.27</td>
<td>0.27</td>
<td>0.28</td>
<td>0.28</td>
<td>0.27</td>
</tr>
<tr>
<td>-20 °C</td>
<td>0.27</td>
<td>0.24</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.24</td>
</tr>
<tr>
<td>-25 °C</td>
<td>-</td>
<td>0.25</td>
<td>0.23</td>
<td>0.22</td>
<td>0.23</td>
<td>0.22</td>
</tr>
</tbody>
</table>

Micro GripTester (mGT): Baseline testing of the mGT produced an average friction coefficient, referred to by the manufacturer as “GripNumber” or GN, of 0.29. As shown in Table 2, somewhat lower friction values were recorded at colder temperatures and after prolonged exposure. Data at 0 minutes exposure to -25 °C was not available solely due to a file transfer error. The most significant changes, greater than 20%, were seen after 1 hour or more exposure to -25
°C. At 0 °C and -10 °C, recorded friction numbers deviated from the baseline value of 0.29 by no more than 10%.

Throughout the course of testing the mGT provided excellent operability at each temperature and period of cold exposure. Only minor issues were encountered: a slight delay in screen function was noted after 24 hours of exposure to -25 °C. We noted that the yellow outer weather covering or shell is easily broken, especially if the device is lifted by the shell. This vulnerability, however, was also evident at warm temperatures.

<table>
<thead>
<tr>
<th>Temperature</th>
<th>0 min</th>
<th>30 min</th>
<th>1 h</th>
<th>90 min</th>
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<th>24 h</th>
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</thead>
<tbody>
<tr>
<td>0 °C</td>
<td>0.27</td>
<td>0.24</td>
<td>0.25</td>
<td>0.24</td>
<td>-</td>
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<tr>
<td>-10 °C</td>
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</tr>
<tr>
<td>-20 °C</td>
<td>0.29</td>
<td>0.23</td>
<td>0.22</td>
<td>0.22</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>-25 °C</td>
<td>0.26</td>
<td>0.22</td>
<td>0.22</td>
<td>0.22</td>
<td>0.23</td>
<td>0.23</td>
</tr>
</tbody>
</table>

**T2Go Portable Continuous Friction Measuring Equipment:** The results of warm baseline testing with the T2Go produced an average friction coefficient of 0.30. At colder temperatures, shown in Table 3, consistently lower values were recorded. Exposure to 0 °C, for 30 minutes or more resulted in 17% to 20% lower friction numbers. At -20 °C and -25 °C, recorded friction decreased by 23% to 27% after 30 minutes or more of cold exposure compared to the warm baseline reading. Prolonged exposure to the cold environment also resulted in consistently lower friction numbers compared to the “0 minute” test performed on initial entry into the cold room.

![Irregular characters on the T2Go control screen.](image)

The full test plan could not be completed with the T2Go. During testing, the device lost power or failed to start after storage in the cold room. Power loss occurred after as little as 90 minutes of cold exposure, despite extended battery charging periods. Due to the limited length of time the
T2Go was available for testing, the equipment was on loan from another agency, it could not be determined whether the battery failures were systemic or due to a problem with the battery pack.

During testing at -20 °C and -25 °C, the control screen mounted on the handle of the T2Go became unreadable, displaying irregular characters as shown in Figure 5. The screen typically appeared normal at start-up and malfunctioned after one to three test runs were completed. The screen could be restored to normal function temporarily by restarting the system. While inconvenient, the screen malfunction did not prevent further testing as the system can be operated by pressing a single button on the handle mounted unit to start and stop test runs, while all other functions can be completed through the associated tablet.

Figure 6. Image of KRC winter test course in Hancock, MI.

VALIDATION IN FIELD ENVIRONMENT

After the encouraging cold room use of the mGT, the opportunity to test this unit on outdoor winter surfaces came as part of a mobility testing program at the Keewenaw Research Center (KRC) during Feb 2020. KRC is located in Hancock, MI and operates a variety of winter surfaces specifically for vehicle testing and evaluation under winter conditions (Figure 6). KRC uses a SAAB friction tester to characterize the friction of their winter surfaces and to quantify improvements and treatments to those surfaces. Additional details regarding the SAAB friction tester and its comparison to other winter traction testing devices in Shoop et al. (1994). Photos of the SAAB and the mGT winter surface testing are shown in Figure 7.
Figure 7. Winter testing with the mGT (left) and SAAB (right) on a packed snow.

Figure 8. Friction measurements on various cold surfaces with the mGT.

The comparison tests were performed on a variety of packed snow and ice surfaces as well as split or alternating asphalt and ice surfaces used to tune vehicle stability control systems. Data from the mGT operated across an area that includes several types of winter surfaces is given Figure 8, showing a wide range of friction values on packed and groomed snow, ice, and asphalt.

A comparison of the SAAB and the mGT friction coefficient measurements on several of the
winter test course surfaces is given in Figure 9, showing excellent agreement between the SAAB and mGT on the winter surfaces with exact agreement of friction coefficients to the nearest 0.01 for the well-groomed ice surfaces, and to within 0.03 for the snow surfaces; and even the largest variation, on the asphalt surface, showing a difference of only 0.05.

![Figure 9. Comparison between the mGT and the SAAB Friction Tester on various surfaces.](image)

**CONCLUSIONS**

Our results document the effect of cold exposure on the accuracy of pavement testing equipment. Tests performed immediately after bringing the devices into the cold room from warm storage were comparable to the baseline tests performed in warm conditions. Performance degraded with colder temperatures as well as longer periods of cold exposure. All three devices evaluated in this study exhibited the similar trends, to different degrees. Pronounced changes in the T2Go test results occurred after 30 minutes of exposure to -20 °C and below. The mGT was most robust to operating in cold temperatures, with significant decreases in measured friction occurring only after one hour of exposure to -25 °C. The DFT was most sensitive to temperature change with significant changes in friction readings after 90 minutes of exposure to 0 °C or any exposure to temperatures below 0 °C.

Not surprisingly, the display screens of all of the devices encountered problems during cold room testing. The mGT experienced only very minor delays in screen function, while the T2Go had persistent screen problems during testing at -20 °C and below. Like the mGT, the DFT controller screen only presented issues after prolonged exposure to -25 °C.

Based on the laboratory cold room study, comparing data from friction testing performed at different cold temperatures is problematic, as the variation between tests may arise from the effect of temperature on the equipment rather than changes in the pavement surface. Based on our experience, we advise that the test equipment should not be stored in freezing temperatures prior to testing, and that temperature and time of exposure should be recorded for each test.

To evaluate the portable friction testers in the field and compare results with a friction tester...
commonly used in cold environments, we evaluated the Micro GripTester (mGT) during field tests on packed snow and ice surfaces at the Keweenaw Research Center in Feb 2020. Several of these tests were run side by side with the SAAB Friction tester, which has been used on cold runway surfaces for many years. The testing showed excellent agreement between the SAAB and the mGT and therefore validating the use of the mGT portable friction tester for snow and ice surface friction measurements.

ACKNOWLEDGEMENTS

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REFERENCES

Developing Pavement Performance Prediction Models Using Rutting Criteria for a Cold Region Environment

Holly P. Trisch¹ and Osama A. Abaza, Ph.D., CEng, M.ASCE²

¹Graduate Student, Univ. of Alaska Anchorage, Anchorage, AK. E-mail: htrisch@alaska.edu
²Professor, College of Engineering, Univ. of Alaska Anchorage, Anchorage, AK. E-mail: oabaza@alaska.edu

ABSTRACT

Maintenance and rehabilitation of roadway pavements in Alaska’s south central and south coast regions are triggered by rutting based on the FHWA threshold of 0.5 in. (12.7 mm) used by Alaska DOT&PF. This leads to short pavement life due to a relatively high rut rate caused by studded tire use during wintertime. Research into the overall rut in these regions shows insignificant rutting from truck traffic. Average pavement resurfacing life in these regions, when considering the effects of rutting, is seven to nine years for freeways and higher for other road classes. This represents about half of the expected design life for principle arterials. Establishing a prediction model that considers the overall rutting will allow agencies to incorporate these models in their pavement management system (PMS). Using rutting and age as the dependent and independent variables, respectively, two models were developed, one for interstate pavements and another for other principal arterial pavements.

INTRODUCTION

The primary objective of this study is to develop the necessary criteria required to produce pavement performance prediction models for the Alaska Department of Transportation and public facilities (AKDOT&P based on the overall rutting, predominantly from the impact of studded tires. Using engineering judgement and data collected by the DOT&PF, criteria for developing these models will be established. Impact of the studded tires and wheel load on rut development are addressed in a previous effort by AKDOT&PF (Abaza, 2019). Furthermore, countermeasures utilized to reduce the impact of studded tires, being the prime cause of rut, are addressed in other research projects (Abaza (2021), Abaza and Dahms (2021), others).

The highway network in the United States represents a multi-billion-dollar investment essential for the movement of goods and people and represents one important factor contributing to the overall growth and economy of the country. One challenge faced by many state highway agencies (SHAs) is how to properly maintain the current quality of pavements in a timely manner and within budgetary constrictions. The Federal Highway Administration (FHWA) defines a pavement management system as a set of tools or methods that (can) assist decision makers in finding cost-effective strategies for providing, evaluating, and maintaining pavements in a serviceable condition. A pavement management system (PMS) can help aid in the decision-making process of the maintenance, rehabilitation or reconstruction of pavement networks within a set amount of time and an allowable budget (Abu-Ennab, 2015). A PMS provides objective information and other helpful data which helps aid pavement engineers make consistent and cost-effective decisions. A PMS itself does not make conclusive decisions, instead, it provides information that provides a basis for understanding the alternatives and consequences of pavement maintenance and rehabilitation decisions (Vitillo, n.d.).

One major component of a PMS is the ability to project the pavement condition of a pavement
over a certain amount of time based on its current pavement condition and other relevant factors that contribute to its condition. The factors that can affect pavement conditions include: traffic loading, environmental factors, drainage conditions, pavement type, subgrade conditions, construction methods, and maintenance activities. One method of predicting future pavement conditions is using pavement performance prediction models. “A pavement performance prediction model is a mathematical equation that could represent condition as it relates to pavement age and/or use (Abu-Ennab, 2015).”

Studded tires contribute to rutting, defined as the abrasive action of the metal studs on pavement surface, in addition to other traffic safety issues in various cold regions. In Alaska, and other cold regions, pavement deterioration increases pavement resurfacing costs. In addition to studded tires, heavy wheel loads of trucks also can cause damage to pavements as well. Research on ruts in central and south coast regions showed insignificant contribution to the overall rut from heavy axle loads (Abaza, 2019). The average life of a pavement in Alaska that is exposed to studded tires is shorter than that of a pavement in the contiguous United States. Recent construction projects in Anchorage, Alaska show that the average pavement resurfacing life, when considering the effects of rutting, is seven to nine years for freeways and higher for other road classes, including arterials and collectors. Studded tires cause damage to hot-mix asphalt (HMA) and rigid concrete pavements by forming ruts on the pavement surface. Wear from studded tires is considered a major pavement distress on high volume roads in Alaska central region. Studies also have shown that there is a strong correlation in pavement wear caused by studded tires and traffic volumes. It was determined that the dynamic abrasion force from studded tires increases as traffic speed increases. The wear caused by studded tires increases when traffic speeds are increased from 50 mph to 75 mph (Abaza, 2019).

AKDOT&PF conducted several research projects in effort to reduce the impact of studded tires on roadway pavement by the use of hard aggregates and polymer modified asphalt in an attempt to reduce studded tire wear (Abaza and Dahms, 2021). More efforts went into developing new materials like the Steel Fiber-Reinforced Rubberized Concrete (SFRRC) as an alternatives in road pavement material to address the long-standing issues of short pavement life due to a relatively high rut rate caused by studded tire use during wintertime (Abaza, 2021).

The AKDOT&PF has a Pavement Management and Preservation Office within its Statewide Design & Engineering Services group. Alaska’s pavement management system involves the collection of current pavement conditions, such as roughness and rutting and cracking, on 4500 centerline miles per year (Alaska, 2019). Various means and methods are used to collect and store this data.

**LITERATURE REVIEW**

Studded tire wear is one of the most important contributing factors that govern pavement life on high traffic volume roads in Southcentral Alaska. Often roads exceed allowable rut depth thresholds of 0.5 in. (12.7 mm) before other pavement distresses have reached high severity. This calls for pavement maintenance or a construction project to address as the main criterion in the structural and material mix pavement design. It is estimated that the annual cost to repair damage caused by studded tires in Alaska is approximately $13.7 million. This is 42 times the amount that the State of Alaska collects in studded tire fees (Abaza, 2019). High rut depth also reduces ride quality and contributes to roadway crashes.

Pavement performance prediction models are a primary element of a successful PMS. These models estimate future pavement conditions and identify the timing for maintenance, rehabilitation,
or reconstruction. It is crucial that these models be reliable and accurate in depicting the actual deterioration trends seen in a systems pavement section. The more accurate a model is in predicting future deterioration patterns means the more beneficial a PMS can be to an agency (APTech, 2010).

In addition to predicting future pavement conditions and identifying the timing for maintenance, rehabilitation, or reconstruction, pavement performance models may also (Abu-Ennab, 2015):

- Predict remaining service life
- Improve project coordination, treatment and timeline goals of an agency
- Evaluate long-term effects of multiple programs
- Provide feedback on the pavement design process
- Determine pavement life-cycle costs and budgets

The four basic criteria that should be used in the development of pavement performance prediction models are:

1. Having an adequate functional form of the model
2. Including all variables affecting pavement performance
3. Having an adequate database
4. Ensuring the model is statistically precise and accurate

Ensuring that these four criteria are met in a pavement performance prediction model is crucial in the accuracy and reliability of the model (APTech, 2010).

Grouping pavements into families is a common approach to modeling pavement performance and has successfully been used by various agencies. It is assumed that the pavement sections within a family all share a similar deterioration pattern. Thus, the pavement performance model developed for that family will showcase the average deterioration for all the pavement sections used in the family. Using the family approach allows the regression analysis to only need pavement condition and age as its two main variables, thus reducing the number of variables required in the regression equation (Rajagopal, 2006).

An important aspect in the development of pavement performance models is selecting the most appropriate significant variables. At a minimum, inventory and monitoring information are two significant variables that should be used to develop these models. Inventory data included any data that does not change with time or traffic, such as geographic location and pavement section length. Monitoring data, such as pavement condition, crack quantity, pavement roughness, and pavement rutting, are all pavement condition data that varies with time or traffic. At its most basic form, a pavement performance model can be developed with only pavement age, surface type, and pavement condition data. The inclusion of additional significant variables, however, increases the reliability of these models. Such additional variables include traffic loads, environmental factors, and pavement structure (APTech, 2010).

Pavement performance prediction models can be linear or nonlinear. “The best equations to use to predict a value of Y based on some value of X is one that minimizes the discrepancies between the regression line and the actual data (Rajagopal, 2006).” In this study, regression analysis was used on data from an approved database and several regression functions were considered, such as linear, polynomial, logistic, exponential, and power function.

The Washington State Department of Transportation (WSDOT) has been using pavement performance models in the Washington State Pavement Management System (WSPMS) since the 1980s. They use a semi-automated pavement condition to determine the pavement condition of the roadways. From this condition data, a pavement structural condition (PSC) index is determined,
and ranges from 100 (good condition) to zero (poor condition). To determine this index, surface distresses related to alligator cracking for flexible pavements, and surface distresses to cracking for rigid pavement are used to determine deduct values. Rutting and ride information is also collected and used in the states pavement management system. Specifically, it is used to calculate and report the pavement rutting condition (PRC) and pavement profile condition (PPC) (APTech, 2010).

Many other states developed prediction models with variables that suit the local conditions. The influential variables that suit that locality were selected to build the models. In Alaska, more specifically, the Alaska Southcentral region had a unique pavement deterioration scheme that require models representing the actual behavior of pavement structures in the area.

**METHODOLOGY**

The four basic criteria that should be used in the development of pavement performance prediction models are: 1) An adequate database, 2) Variables which affect pavement performance, 3) An adequate functional form of the model, and 4) Model accuracy and precision. Adequate database, independent and dependent variables, as well as determining the appropriate functional form are considered. Identifying these criteria will assist the agency in developing models, which are created specifically for Alaska and its unique pavement maintenance and rehabilitation challenges.

It is recommended that models be created for specific locations, functional classification, and pavement type. The Central Region of Alaska, specifically the city of Anchorage will be the primary location of the study. The functional classification recommended to be chosen for modeling are interstate and other principal arterial roadways. These functional classifications were chosen based on the request of the DOT&PF, who desired to attain more information on these types of roadways. It should be noted that these types of roadways typically see high amounts of pavement damage, particularly due to rutting. Only asphalt concrete (AC) pavement will be included for modeling.

Family models, in lieu of individual section models, are being recommended so pavement sections with similar characteristics can be grouped together into various models. As previously mentioned, interstate and other principal arterial AC pavements in Anchorage will be analyzed. According to the FHWA, interstate roads can be defined as “the highest classification of Arterials and were designed and constructed with mobility and long-distance travel in mind (Federal, 2017). In addition, the FHWA defines other principal arterials as “roadways [which] serve major centers of metropolitan areas, provide a high degree of mobility and can also provide mobility through rural areas (Federal, 2017). Since all pavement sections within these two functional classifications have the same geographic location and pavement structure, they can be grouped together into a family model. Since two separate functional classifications are being analyzed, two separate family models can be created. One family model will represent interstate AC pavements in Anchorage, while the other model will represent other principal arterial AC pavements in Anchorage.

By combining pavement sections that share common deterioration characteristics, we can limit the number of variables required in the regression equations. This reiterates the requirement of creating simplistic pavement performance prediction models.
DATA COLLECTION, ANALYSIS AND RESULTS

Database

The current pavement management system database for the AKDOT&PF includes various pavement condition data, updated traffic data and construction information. The Pavement Management System department provided sample data from this database including Average Annual Daily Traffic (AADT) data from 2017, construction history of pavements ranging from 2000 to 2018, and pavement data from 2000 to 2018. In detail, the various data includes: pavement geographic location, functional classification, international roughness index (IRI), rutting index, maintenance, rehabilitation and reconstruction information (including type and date of when maintenance and rehabilitation occurred), and AADT.

The database of the AKDOT&PF had over 70,000 individual rows of pavement section data. The data required various stages of filtering and many rows of extraneous data had to be removed in order to generate information that provided acceptable results. In reality, this database provided a foundation to the development of the pavement performance prediction models. While it did contain a sufficient volume of data, the data was not initially considered complete, and functional. The database did, however, become complete, reliable and functional after careful analysis and consideration was given to remove all extraneous data.

It should also be noted that the means and methods used to collect the data found in the database significantly changed over the 18-year period. Changes in technology, data collection contractor, employment, and other various factors affected the way the data was collected, analyzed, and stored. This was also discussed with who understood the challenges involved in using this database. Two major variances seen in the data would be the type of construction applied to the pavement section and the overall rutting/IRI information. Opposing opinions on what constitutes maintenance from rehabilitation lead to some data entries being incorrectly labeled. In addition, the techniques and technology used to attain the rutting and IRI had significantly changed over the 18-year period. This change in collecting data meant that some information was more realistic and accurate than others. Through careful analysis, however, the appropriate data was removed to ensure that the data being used in the creation of the models was complete, reliable, and functional. It is worth missioning the data screening, retrieval and processing was coordinated with the Alaska DOT&PF to ensure the process follows the understanding of the responsible parties of the limitation of the dataset.

Variables

Since it is being recommended to combine pavement sections that share common deterioration characteristics, the number of variables required in the regression equations are limited. The model criteria being proposed includes the creation of family models that combine geographic location, pavement type, and functional classification of selected pavement sections in Anchorage. Thus, the recommended variables needed for regression equations and modeling are rutting, IRI, pavement age since the last maintenance and/or rehabilitation, and AADT. Further explanation of why these variables were chosen over others will be examined.

The primary pavement engineering/management problem in urban areas of Alaska is premature failure due to rutting or wearing of the asphalt concrete surface courses (urban rut modeling). Use of studded tire wear in the winter and plastic flow in the summer has a tremendous effect on Alaskan urban roadways. Studded tires are used by 36% of the traveling public (Abaza, 2019) and are legally allowed for use on roadways in Alaska for 6.5 months out of the year. The
freeze-thaw mechanism leaves many roads in Anchorage Alaska wet and/or bare during the wintertime, making them susceptible to aggregate picking and further damaged by use of studded tires. On the other hand, the long summer days cause the asphalt in pavement to soften, causing plastic deformation throughout the pavement section (Gartin et al, 2005).

It was determined that AC pavement surfaces found on high traffic roads in the cities of Anchorage, Juneau, and Ketchikan develop rutting patterns within five years of use. This is attributed to the use of studded tires in the winter and plastic deformation in the summer. Rutting failures of Alaskan roadways are typically found on those with traffic levels of at least 4,000 AADT (Gartin et al, 2005).

The pavement condition indicators recommended for use in the development of these models are rutting and IRI. As previously mentioned, the primary cause for deterioration of pavements in Anchorage Alaska is due to rutting and the wearing of the surface course of AC pavements. Therefore, using these variables are crucial in properly analyzing and predicting future deterioration patterns of given pavement sections. These variables will represent the dependent variables in the modeling and regression equations. Hence, these variables are dependent on the independent variables, and may or may not fluctuate under certain circumstances.

The independent variable recommended for use should accurately describe the performance of the pavement sections being analyzed. The age of the pavement is most commonly used in pavement performance prediction models and is an extremely important variable to be considered. Therefore, the age of the pavement should be used as an independent variable in modeling and the regression equations. Equations 1 and 2 are used to find actual pavement life cycle (age).

\[
\text{Age} = [(\text{IRI measurement Year}) - (\text{Year of last maintenance/rehabilitation})] \quad (19)
\]
\[
\text{Age} = [(\text{RUT measurement Year}) - (\text{Year of last maintenance/rehabilitation})] \quad (20)
\]

In addition to age, the AADT may also be considered as an independent variable, specifically when paired with a rutting dependent variable using the AADT as an independent variable could be cumbersome in comparison to using age as the independent variable, but nonetheless, could provide a different outcome that may be helpful in modeling deterioration patterns of selected pavement sections. It is important to note that since typically pavement failures due to rutting specifically are correlated with AADTs of 4,000 and over, that only pavements meeting this requirement should be used.

**Functional Form**

It is recommended that both independent and dependent variables be used in the development of these models. Hence, it is evident that these models will be classified as deterministic models, which predict a dependent value (pavement condition) based on multiple independent variables. Regression analysis is then conducted to predict deterioration patterns, based on these selected variables. The functional form recommended for these models is linear regression, which is a simplistic method of linear regression. The logarithmic (power) equation was also considered for use in these models. Least linear squares regression will then be used to determine a best fit line from the data, minimizing the smallest squared differences between the observed and predicted data (APTech, 2010).

**Model Development**

Once potential independent and dependent variables were established for use in the pavement performance prediction models, modeling and analysis could be conducted. The results of these models will ultimately determine how the final database will be filtered and which pavement
variables will be used. Initially, family curves were used, combining the following pavement data characteristics: 1) Located in Central Alaska, more specifically Anchorage; 2) Constructed of AC pavement; 3) A functional classification of principal arterial and interstate roads; and 4) Treatment type of Preventative maintenance, Rehabilitation, and Reconstruction. The first two variables to be modeled were age (independent variable) and IRI (dependent variable). A linear functional relationship between Age and IRI were established. However, this relation has a very low regression fitting indicating a weak relationship. Furthermore, the changes in the IRI over the life of the pavement are limited or non-existent.

A typical pavement life associated with other principal arterial and interstate roads in Alaska ranges from seven to thirteen years. Thus, it would have been expected, over this amount of time to see a significantly higher IRI, indicating increasing deterioration. Based on this initial model, it was initially determined that IRI would be excluded from future models, due to its inability to show proper deterioration rates over the lifespan of the pavement structure in this region. However, it was ultimately decided that an AADT vs. IRI model should be created to see if this model would show more realistic and expected trend. Once again, a functional relationship between AADT and IRI is evident. However, weak strength of association is seen. In addition, this model shows a significant number of outliers. An increase in AADT should show a more pronounced deterioration trend when associated with IRI. While both IRI and Rut are significant pavement condition variables that show levels of deterioration in the selected pavements, it is evident that IRI may not be the best variable to use in future models. As previously noted, IRI is a longitudinal measurement of depth. The majority of the pavements in Alaska, however, deteriorate due to rutting conditions, which is a transverse depth measurement. In addition, due to short pavement life in this region due to rutting, irregularities in the pavement profile (IRI) do not develop. Understanding the current deterioration trends in pavements in Alaska and the results of the initial models led to the decision to exclude IRI from the final models. Therefore, IRI will be eliminated from the primary database and will not be used as a dependent variable in the AKDOT&PF’s pavement performance prediction models.

Using the same family curves, models were produced of the next two variables. These variables included an independent variable of Age and a dependent variable of Rut. There is a weak association by the variation in data from the trendline. A more defined trendline is evident using regression fitting ($R^2$), however, which shows a more realistic deterioration pattern that one would expect to see from these variables. Most of the deterioration associated with the pavements in Anchorage is due to rutting – thus this is a variable that could produce deterioration trends that could help the AKDOT&PF better manage the current pavement management system. Therefore, it was decided to keep rut as a main dependent variable pending further improvement. Next, the variables AADT vs. Rut were modeled to ensure that the same functional form and realistic trendline was evident. This model shows the same features as in the previous model, including a linear trendline, which positively increases as AADT increases. It is anticipated that an increase in AADT would generate higher rut depths, and this model depicts these expectations. It is evident that AADT, along with Age are independent variables that will model a deterioration trends pending further refinement.

To further enhance the models several trials and functions were used to improve the regression fitting. Furthermore, data were separated into two main functional classes, Other Principal Arterials, indicating classes other than interstate roads. Amongst other types of regression equations are the logarithmic regression function was used. An example of this model is shown in Figure 1. The range of regression fitting based on these measures reached about fifteen percent.
Linear regression showed about the same outcome. Based on the trails of the functions used, linear regression was chosen to reflect the relationship for the comparable results of regression fitting and simplicity of the model. It should be noted, however, that in Alaska (versus the rest of the United States), studded tires and heavy traffic loads contribute to the overall rutting in the first few years, after which only studded tires cause major ruts. This is depicted in the logarithmic regression in Figure 1 which reflects weak correlation. In Alaska, technically speaking, the threshold for new maintenance is warranted when ruts reach 0.5 in. (12.7 mm). Data showed much higher levels of rutting were reached before new maintenance was provided.

The low regression fitting achieved after the new measures may be attributed to the inconsistencies in the database. The AADT listed in the database is a combined AADT, which averages the AADT over one pavement section. While this may have initially been done for ease in filtering data, it depicts unrealistic values. It is realistic to say that the AADT value of a one-mile road is consistent, whereas in reality, this value may fluctuate throughout relative to pavement condition. Thus, it was determined that AADT would not be the best independent variable of choice for creating pavement performance prediction models for the DOT&PF due to the lack of segmental AADT database.

After eliminating AADT as an independent variable, the resulting variables considered for further modeling were Age (independent variable) and Rutting (dependent variable). At this point, linear regression equations have higher regression fitting depicting trends among the variables. To further examine the data, the individual rutting data points were analyzed to ensure that the data provided was realistic to the conditions in the field. After careful consideration, it was noted that there were various outliers in the data, which was negatively affecting the regression fitting. More specifically, many of the data points shown were unrealistic to the actual rutting that would occur over a certain time period. This was verified with the AKDOT&PF. Engineering judgement and
coordination with AKDOT&PF testify that any amount of rutting over 0.40 inches in the first year of a pavement’s life is unrealistic, thus this can be considered extraneous data. With that said, engineering judgement could also attest to the fact that in year 10, any rutting below 0.40 inches would also be unrealistic. This type of extraneous data was seen in both models of Other Principal Arterials and Interstates roads. Having this type of data in the models will negatively affect the regression fitting and is unrealistic to the true nature of rutting in pavements in the area. This was affirmed by the AKDOT&PF as the minimal measure that can be taken for the purpose of this study. Thus, further filtering of the data was completed to remove this extraneous data. It was confirmed by the AKDOT&PF the outliers are mostly discrepancies in the field measurements and/or documentation.

![Figure 2. Age vs. Rutting Depth, based on filtering PMS Data, Other Principal Arterial](image)

The typical pavement life of Other Principal Arterials is eight to thirteen years. Therefore, for the models using this functional classification, age was filtered to only show pavement age up to an age of thirteen. In addition, for Interstates, the typical pavement life is seven to nine years (Abaza, 2019). These models' ages were filtered to only show pavement age up to an age of nine years. Careful consideration and engineering judgment was used from this point forward to remove extraneous rutting data from both models. For other principal arterials certain limits were placed each year of the pavements age to determine whether certain rutting data was acceptable. An example of this can be seen in the model, results shown in Figure 2. For year one, it was determined that only rutting values of 0.00 inches to 0.20 inches were acceptable for that age. Thus, all data not within this set limit, was removed. Furthermore, in year 10, an acceptable limit of rutting values between 0.35 inches and 0.55 inches was determined. All data not within these limits were removed.

Removing the extraneous data would allow the models to show more realistic trends eventually, provided a more fitted model. The data shown for Age versus Rut for Other Principal Arterials is realistic to the rutting that would occur each year of a pavement’s life in the area. These values hold more truth to what is typically seen on this type of road in Anchorage. It should also be noted that a linear relationship was used in lieu of logarithmic. Removing the extraneous data showed that a linear regression equation would be more appropriate, realistic and would produce a higher
level of coefficient of determination. Using the same approach with Interstate roads, Figure 3 shows the outcome of the filtering process.

After careful analysis, the appropriate variables, functional form, and statistical regression methods were determined for use in a pavement performance prediction model for Anchorage Alaska. It was determined that rutting and age are the best variables to show proper deterioration of pavements within Anchorage in the Alaska Southcentral region. In addition, the focus of these models will be on AC for Other Principal Arterials and Interstates in the Alaska Southcentral region. Using the proper pavement age life based on functional classification, the data reflected in these models are a depiction of the unique pavement conditions found in Alaska.

![Figure 3. Age vs. Rutting Depth, based on filtering PMS Data, Interstate](image)

**Summary Findings**

Anchorage Alaska, similar to other cold region environments, experiences unique pavement deterioration patterns when compared to other states. Extreme weather conditions, especially those experienced in the winter, is the driving variable in the pavement damage experienced throughout Alaska’s pavements. More specifically, studded tires, along with continuous heavy traffic loads are the two contributing factors found to cause the most damage to pavements in the city of Anchorage Alaska Southcentral region. These variables lead to rutting, which as previously discussed, is the common denominator found among pavement damage. This study aimed at determining which variables are the most prominent/influential in helping to form pavement performance prediction models for Anchorage. Based on the results of this study, conclusions can be drawn and discussed, and recommendations can be presented on how to move forward with these proposed models.

Initially, the database provided by the DOT&PF appeared to be complete, reliable, and logical. Further analysis revealed, however, that the database contained extraneous information that would need to be filtered to provide data that was reliable and logical. Pavement age was filtered to show only pavements whose IRI and Rut data was taken after pavement age was reset due to maintenance, reconstruction, or rehabilitation. Further modeling then revealed discrepancies in the IRI data. The literature review, in addition to careful analysis, confirmed that rutting should be the primary dependent variable used in these models. The IRI data was then removed from the database as a result of the quality of data provided and the relatively short pavement life in this
region which might not show significant pavement roughness (IRI) during this short period.

Modeling continued, until the next challenge arose, which was found in the AADT data. The AADT data had previously been combined and averaged for one-mile sections of pavements. While this may have initially been done for ease of managing data, it did not present realistic data and did not depict the true traffic experience on pavement sections in Anchorage. Therefore, AADT was removed from the database. At this point, it was decided that the independent variable recommended would be pavement age and the dependent variable would be rutting. To further refine the models, it was determined that two separate models should be considered, each separated by different functional classifications. One model would represent pavement deterioration for other principal arterials and the remaining model would represent deterioration for interstates.

To ensure statistical accuracy, a functional form was tested for each model to determine which was best at representing the actual deterioration patterns. The R2 value, or coefficient of determination, was given for each model. This variable is a common statistical variable which is a measurement of statistical accuracy between actual and fitted data. Several functional forms were tested on these models including linear regression, second-order polynomial regression, and logarithmic regression. It was during this analysis that a major flaw in the DOT&PF’s database was noticed. It was determined that extraneous rutting data was skewing the regression equations and was producing unrealistic data. An example of this discrepancy would be to have abnormally high rutting depths in the first year of a pavement age, while then having abnormally low rutting depths during the more mature ages of a pavement section. This required filtering and analysis.

TABLE 1. Summary of Model Regression and Determination Coefficients

<table>
<thead>
<tr>
<th>Functional Form</th>
<th>Figure</th>
<th>X</th>
<th>y</th>
<th>b0</th>
<th>b1</th>
<th>b2</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>2</td>
<td>Age (Year)</td>
<td>Rutting Depth (in.)</td>
<td>0.097</td>
<td>0.03</td>
<td>3</td>
<td>0.7779</td>
</tr>
<tr>
<td>Linear</td>
<td>3</td>
<td>Age (Year)</td>
<td>Rutting Depth (in.)</td>
<td>0.104</td>
<td>0.03</td>
<td>7</td>
<td>0.6004</td>
</tr>
</tbody>
</table>

It was decided that the best approach was to limit the pavement age for each functional classification. Based on a literature analysis, it was determined that the average pavement life for other principal arterials was eight to thirteen years and for interstates, the average pavement life is seven to nine years. Thus, all extraneous data, outside of these limits, for each functional classification was removed. Doing so allowed each model to be properly fitted and produced a more realistic coefficient of determination. While initially it was thought that the best functional form to represent the data was logarithmic, filtering the data showed different results and proved that linear regression equations would be the most appropriate for the models. The resulting proposed models are shown in Figures 2 and 3 for Other Principle Arterials and Interstate, respectively. Table 1 summarizes the model regression coefficients and coefficient of determination.

CONCLUSIONS AND RECOMMENDATIONS

In developing prediction models for Anchorage, Alaska Southcentral region, age was determined to be the most appropriate independent variable, while rutting depth was determined to be the most appropriate dependent variable. These are the two main variables proposed for the
creation of pavement performance prediction models per the available dataset. Linear regression equations were chosen to be the most appropriate functional form used to fit the model. The accuracy of the model was confirmed with a high coefficient of determination which indicates a strong correlation between the actual and fitted values. These criteria are being recommended to the AKDOT&PF for the creation of a pavement performance prediction model for Anchorage. It is recommended that two models be considered, one for each functional classification, including both other principal arterials and interstates.

It is recommended that these models and their associated criteria, be used as a basis in further developing models. Additional research should be done to improve the variables being used within the models and the overall statistical accuracy of the model. More specifically, the AADT data could be further refined as to ensuring that it is properly depicting the actual traffic volume seen on these pavement sections. Once sufficient AADT data is available, it is recommended to create models based on sections of roadways with similar volumes of AADT. Developing models based on maintenance type (rehabilitation, reconstruction, and preventative maintenance) is also being recommended for consideration. Creating models that cover other various regions in Alaska should also be considered as environmental and traffic conditions differ. In addition, to ensure that these models are meeting the listed criteria, the pavement database also needs to be carefully monitored and updated with the latest and most accurate data. Consistency is key to a successful database. Ensuring that the database reflects the most current pavement conditions will allow the models to be fitted properly and thus produce more realistic results. Thus, continuing to research alternative variables and improve the statistical accuracy of the models, along with proper database management are the primary recommendations offered in this paper.

ACKNOWLEDGMENTS

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REFERENCES


Estimating Sub-Surface Snow Density Using GPR and the Surface Reflection Method

Adrian B. McCallum, Ph.D.  

1School of Science, Technology and Engineering, Univ. of the Sunshine Coast, Sippy Downs, Australia. ORCID: https://orcid.org/0000-0002-3718-614X. E-mail: amccallu@usc.edu.au

ABSTRACT

The surface reflection method is a popular method of determining layer dielectrics in road pavements. Previous research has also applied this technique to snow to estimate surface snow density. Here, this method is extended to estimate sub-surface snow density and layer thicknesses. An air-coupled 800 MHz ground penetrating radar (GPR) antenna was used to image alpine snow, and comparison was made with an ideal reflector, a metal plate. Quantitative analysis of the GPR trace and application of the surface reflection technique allowed sub-surface snow density and surface layer thickness to be resolved. Although discrimination of second layer thickness was poor, this technique introduces a simple method by which surface and sub-surface snow layer density and thickness could be rapidly estimated over large spatial areas.

INTRODUCTION

GPR is routinely used to image snow subsurface stratigraphy and amplitude comparison with an ideal reflector has been used to determine the radar wave velocity in firn (Jezek, 1983). However, the surface reflection method described by Maser (1991) had not previously been used to estimate the surface dielectric of snow using an air coupled impulse GPR antenna.

Although the surface reflection method (Maser, 1991) is typically used for road pavement analysis, McCallum (2014) provided preliminary analysis showing that snow surface density can also be estimated using this method. Maser and Scullion’s equation is shown (Equation 1) along with a graphical representation in Figure 1.

\[
\varepsilon_a = \left[ \frac{(1+\frac{A_1}{A_2})^2}{(1-\frac{A_1}{A_2})} \right]
\]

Figure 1. Typically applied to road pavements, the surface reflection method compares the amplitude of reflection from the medium surface (A1) with that from an assumed ideal reflector (metal plate) (A2).

By comparing the relative amplitude of the electromagnetic wave reflected off the pavement surface (A1) with that reflected off an assumed perfect reflector (steel plate, A2) the relative dielectric of the surface (\(\varepsilon_a\)) can be estimated. A simple equation to derive density of an air-ice...
mixture from the dielectric constant, such as that proposed by Kovacs (1993) for dry firn, can then be applied:

$$\varepsilon'_r = (1 + 0.845\rho)^2$$

where $\varepsilon'_r$ is the dielectric constant and $\rho$ is snow density ($\text{kg m}^{-3}$). McCallum (2014) outlined the application of the surface reflection technique to alpine snow and using Ulaby's simple equation relating dielectric constant and snow density (Equation 3 (Ulaby, 1986)), he obtained an estimate for surface snow density within 2% of the gravimetrically determined surface snow density:

$$\varepsilon_r = (1 + 1.9\rho)$$

where $\varepsilon_r$ is the dielectric constant and $\rho$ is bulk snow density ($\text{kg m}^{-3}$). However, in addition to estimating surface layer dielectric/density, the surface reflection method can be extended to estimate surface layer thickness and sub-surface layer dielectric properties (Maser, 1991).

![Figure 2. Time and amplitude information from subsequent layer interfaces can be used to estimate layer properties (from Scullion 2001).](image)

A typical air coupled GPR return waveform for a road pavement is shown, where peaks $A_1$, $A_2$ and $A_3$ are reflections from the surface, top of the granular base and subgrade respectively (Figure 2). From this information, surface layer thickness can be estimated (Equation 4):

$$h_1 = \frac{c\Delta t_1}{2\sqrt{\varepsilon_a}}$$

where $h_1$ is the thickness of the surface layer, $c$ is the speed of an electromagnetic wave in a vacuum, $\Delta t_1$ is the time delay between peaks $A_1$ and $A_2$ from Figure 2 and $\varepsilon_a$ is the dielectric of the surface layer. The dielectric of the base layer ($\varepsilon_b$) can also be calculated (Equation 5):

$$\sqrt{\varepsilon_b} = \sqrt{\varepsilon_a} \left[\frac{1 - [\frac{A_1}{A_m}]^2 + [\frac{A_2}{A_m}]^2}{1 - [\frac{A_1}{A_m}]^2 - [\frac{A_2}{A_m}]^2}\right]$$

where $\varepsilon_a$ is the dielectric of the surface layer, $A_1$ is the amplitude of the surface echo, $A_2$ is the amplitude of reflection from the top of the base layer and $A_m$ is the amplitude of reflection from a large metal plate, placed on the snow surface. Once the dielectric of the base layer has been
established, layer density can then be estimated using Equation 3 or similar then base layer thickness can also be estimated using Equation 6:

\[ h_{\text{base}} = \frac{c\Delta t}{2\sqrt{\varepsilon_b}} \]

where \( h_{\text{base}} \) is the thickness of the base layer, \( c \) is the speed of an electromagnetic wave in a vacuum, \( \Delta t \) is the time delay between peaks \( A_2 \) and \( A_3 \) from Figure 2 and \( \varepsilon_b \) is the dielectric of the base layer. This paper builds on McCallum’s work (McCallum, 2014) to examine whether this further adaption of the surface reflection technique can be applied to snow.

Figure 3. ~1 m high snowbank with 800 MHz GPR antenna shown in air-coupled configuration, sitting on an upturned plastic box some 270 mm above the 2 mm thick, 0.6 m x 0.6 m metal plate, resting upon the snow surface. Internal interface \( A_2 \) is marked.

### METHOD

The method used in this study to acquire GPR and snow density data was outlined comprehensively in McCallum (2014). Briefly, a snowbank approximately 1 m high was identified beside an asphalt carpark at Perisher Valley ski resort in the Australian Alps. After ‘squaring’ of the bank using a shovel and snow saw, snow layer density was assessed gravimetrically and then return amplitudes from an 800 MHz air coupled GPR were compared with returns from an ideal reflector (a flat steel plate) to estimate snow surface dielectric. Layer density and thickness are presented shortly (Table 1); layer origins are unknown, except that they probably represent two distinct formation events, are separated by a thin layer of ice, and are metamorphosed but unworked. Figure 4 shows a photograph of the snowbank with the 800 MHz antenna mounted in air-coupled mode upon an upturned plastic box, which is sitting upon the 2 mm thick metal plate.

From this, surface snow density was estimated using Equations 2 and 3. Additional radar parameters as identified in Figure 2 were then extracted from the radargram to input into Equations 4, 5 and 6.
METHOD

Figure 4 shows a representative gain-modified ‘wiggle’ trace obtained from the air-coupled 800 MHz antenna suspended over the snow surface.

Figure 4. Gain-modified ‘wiggle’ trace obtained from the air-coupled 800 MHz antenna suspended 270 mm above the snow surface. The first three -‘ve/+‘ve cycles evident within this trace represent the direct radar wave (A₀; see Figure 2) and are not reflections. Additional returns from an internal interface (A₂) and the base of the snow (A₃) can be seen. The surface reflection (A₁) occurs at a similar time to the direct wave and is not evident.

The return at ~ 5 ns is interpreted as the direct wave or end reflection (A₀; see Figure 2), the return at ~ 12 ns is interpreted as an internal interface (A₂) and the return at ~ 18 ns is interpreted as the snow/ground boundary (A₃). Because the antenna was suspended only 0.27 m above the snow surface, the surface reflection (A₁) occurs at a similar time to the direct wave and is not evident. X-axis (‘wiggle trace’) is pulse amplitude (volts) and y-axis is return travel time (nanoseconds).

DISCUSSION

Surface layer thickness (h₁): The estimated thickness of the surface layer is considered first. The interpreted two-way travel time between the surface reflection and the interpreted internal boundary is ~ 6 ns. If this value is input into Equation 4 along with the calculated average dielectric value for the snow (εᵢ) of 2.1 (McCallum, 2014) then a surface layer thickness equal to ~ 0.62 m is obtained. This agrees very well with the measured first-layer snow thickness of ~ 0.63 m (Table 1).

2nd layer density: The first interface return (A₂) will vary in both phase and amplitude depending on the dielectric variation between the surface and base layer (Figure 2): if the dielectric difference is small, the amplitude of the return will be small; if the dielectric increases, A₂ will be
If the dielectric decreases, A₂ will be -ve. In this situation, density and thus dielectric is less in the lower layer, therefore A₂ assumes a negative value.

### Table 1. Measured and estimated snow layer thickness and density.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Measured Thickness (m)</th>
<th>Estimated Thickness (m)</th>
<th>Measured Density (kg m⁻³)</th>
<th>Estimated Density (kg m⁻³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.63</td>
<td>0.62</td>
<td>587</td>
<td>578¹</td>
</tr>
<tr>
<td>2</td>
<td>0.17</td>
<td>0.63</td>
<td>555</td>
<td>515¹ 558²</td>
</tr>
<tr>
<td>3</td>
<td>0.1</td>
<td>Not estimated.</td>
<td>Not measured.</td>
<td>Not estimated.</td>
</tr>
</tbody>
</table>


Data from Table 1, calculated surface dielectric value (ε_a = 2.1) and A₂ (Figure 2) can now be input into Equation 5. An estimated A₂ value of -100 Volts generates a second layer dielectric (ε_b) equal to 2.06. This is equivalent to a density of 515 kg m⁻³ (Equation 2) or 558 kg m⁻³ (Equation 3). These values agree very well with the gravimetrically measured snow density of ~555 kg m⁻³ (McCallum, 2014). However, it should be noted that selection of the interface return in situations of low dielectric contrast such as in this case, is quite subjective.

2nd layer thickness. The second layer dielectric value can now be input into Equation 6 generating an estimated base layer thickness of 0.63 m. This is approximately three times the measured thickness between the internal snow layer interface and the snow/ground interface. In this situation, this method has not enabled accurate assessment of second layer thickness.

This may be because of the insensitivity of Equation 5 to variations in echo amplitudes identified herein; layer dielectrics remain almost the same; also, unreasonable dielectric values are required in Equation 6, to generate viable estimates for layer thickness. Further, snow moisture may have impacted radar echo amplitudes, and thus snow dielectric, density, and thickness estimates.

### CONCLUSION

In snowpacks exhibiting distinct stratification, quantitative assessment of GPR data using the surface reflection technique may provide insight into surface and subsurface layer composition. In this proof-of-concept examination, surface layer thickness and second layer density were successfully resolved, from quantitative analysis of the GPR trace. However, assessment of second layer thickness using this technique was poor.

Application of this road pavement assessment technique to snow appears theoretically sound. However, further controlled experimentation is required to better assess the applicability of this technique to snow. A simple method by which surface and sub-surface snow density and layer thickness may be remotely estimated using commercial GPR equipment has been presented; this may enable large-scale geospatial snow stratigraphy mapping, potentially via UAV.

### ACKNOWLEDGEMENTS

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Synopsis: Permafrost Engineering in a Warming Climate—Current State and Future Strategy

Kevin Bjella, P.E., M.ASCE; Heather Brooks, Ph.D., M.ASCE; Zhaohui Yang, Ph.D., M.ASCE; and Edward Yarmak, P.E., M.ASCE

1Cold Regions Research and Engineering Laboratory, Fort Wainwright, AK. E-mail: kevin.bjella@usace.army.mil
2BGC Engineering Inc., Edmonton, AB, Canada. E-mail: HBrooks@bgcengineering.ca
3Dept. of Civil Engineering, Univ. of Alaska Anchorage, Anchorage, AK. E-mail: zyang2@alaska.edu
4Arctic Foundations Inc., Anchorage, AK. E-mail: eyarmak@arcticfoundations.com

ABSTRACT

The permafrost engineering profession is challenged with the next stage of evolution, as the last few decades have seen a steady rise in permafrost temperature. The changing environment is generating uncertainty regarding safe engineering, and longevity, and the result is often overly conservative designs that greatly increase development costs. The ASCE Cold Regions Engineering Division (CRED) committees agreed a review of the current state of the profession and an outline of a future strategy were needed. The International Permafrost Association (IPA) sanctioned an action group to perform a workshop and look at the permafrost engineering discipline, determine knowledge gaps, and suggest a path forward. This paper presents a synopsis of the more important findings such as: the need for programmatic work in developing ‘living’ permafrost temperature forecast tools, technical advances in methods to design for increasing thaw sensitivity, designing and planning with threats from altered hydrology (thermo-erosion) and slope instabilities, and techniques to mitigate thaw-affected vertical and horizontal infrastructure, to name a few. Most importantly, the engineering profession, with the help of universities, must promote the backfill of the retiring frozen ground engineering workforce.

INTRODUCTION

Formalized North American permafrost engineering has its genesis with the construction of the Alaska Highway in WWII and Cold War activities spurring basic and applied studies on freezing soils and permafrost properties, all with an aim for better understanding of the design and operation of all forms of infrastructure, vertical and horizontal. The 1970’s was a pinnacle period where the knowledge of permafrost properties and new technology for maintaining ground in the frozen state, culminated in the successful design of a warm oil pipeline constructed to traverse the thaw sensitive permafrost terrains of Alaska. Many of the design parameters were predicated on a continued cold climate.

Since that time, advances have been made, such as further innovations for maintaining ground in the frozen state, and improved methods for characterizing permafrost terrains. Now, permafrost engineers are confronted with the next stage of the profession evolution. Natural, and anthropogenic impacts, mostly attributable to a warming climate, are progressively instigating thaw and creating unique challenges for the design of stable infrastructure. Some issues requiring attention include designing for increasing thaw sensitivity, planning and designing for threats from altered hydrology (thermo-erosion) and slope instabilities, and mitigating effects of permafrost thaw on vertical and horizontal infrastructure, just to name a few. In general, the changing...
environment has created unknowns, which in turn are generating uncertainty on the level of progressive planning, and safe engineering. This often results in conservative designs, which are robust for life safety and longevity, but may be over-engineered and needlessly increase development costs. This affects government and private sector projects alike.

Warming permafrost has been recognized for at least the last two decades as an emerging problem for design and development. Hinkel (2003) and Tucker (2004) defined the needs and gaps to address the issue, and a more recent, higher-level look was conducted by Andreassen et al. (2011). In this context, we believe the next generation of parameters for design, construction, and maintaining infrastructure in a warming permafrost environment is a high priority need. Recently three committees of the Cold Regions Engineering Division (CRED) of the American Society of Civil Engineers (ASCE); Frozen Ground, Structures and Foundations, and the Transportation and Infrastructure, all agreed that a review of the current state and an outline of a future strategy are required to facilitate advances within the frozen ground engineering profession, for the common good of everyone in cold regions.

To assist in this effort, the International Permafrost Association (IPA) sanctioned an Action Group to hold a workshop and discuss the current state of the permafrost engineering discipline, determine knowledge gaps, and suggest pathways forward to address deficiencies and issues. The meeting was held on the campus of the University of Alaska – Fairbanks on November 13, 2018, with participation of over 30 individuals including representatives from academia, government engineers/scientists, private industry. The group discussed emerging topics and specific issues that are of most importance to the permafrost engineering community. This paper presents a synopsis of the more important findings and provides a baseline for further discussion. The primary objectives consisted of developing a comprehensive look at the issues most plaguing the profession with three ultimate goals:

- Identify knowledge and technology gaps;
- Identify any procedural or methodological shortcomings; and
- Identify pathways to address the issues.

DISCUSSION

Participants were asked to contribute ideas via questionnaire (Table 1) prior to the actual workshop, to allow for a more focused discussion during the roundtable.

**Table 1. Pre-workshop Questionnaire**

<table>
<thead>
<tr>
<th>General Engineering</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. What guidelines are you using if at all in your design work?</td>
</tr>
<tr>
<td>a. Do you utilize UFC 3-130</td>
</tr>
<tr>
<td>2. Refined modeling capabilities for hydro conditions – surface and subsurface</td>
</tr>
<tr>
<td>3. Role of hydro-geology – surface and subsurface</td>
</tr>
<tr>
<td>4. Geophysics</td>
</tr>
<tr>
<td>a. Do you conduct this work in your investigations?</td>
</tr>
<tr>
<td>b. Do you contract this work out?</td>
</tr>
<tr>
<td>c. Do you not utilize?</td>
</tr>
<tr>
<td>5. Seismic considerations</td>
</tr>
<tr>
<td>a. Testing – shear wave velocity</td>
</tr>
<tr>
<td>i. Frozen condition</td>
</tr>
</tbody>
</table>
ii. Loose thawed condition
   b. Hazard classification
6. Thaw consolidation: what is the consensus of how to compute, what is the realism of the results?
7. Other parameters that need further testing
8. Bi-modal foundations: good, bad, indifferent
9. Mechanical properties, strength (creep and bearing capacity) Thermal properties

Climate Warming
1. How are you projecting climate warming into your designs?
   a. Does your method change depending on the type of project?
   b. Does your method change depending on the client?
2. Which SNAP scenario is appropriate for a project?
3. How is the unknown balanced with over-engineering and high costs to the client?
4. Suggest research project - Current engineering guidelines are utilized for a large-scale PF project, perhaps deep piles, and the cost impact is determined based on the chosen SNAP scenario.

Education
1. With respect to permafrost engineering, are there notable deficiencies with the general undergraduate population that need to be addressed?
2. Do undergraduates have a strong enough background in permafrost morphology to adequately interpret scant borehole data, or make inferences from resistivity or GPR data?
3. What changes are recommended that could reasonably be attained and the school remain accredited?
4. Are undergraduates adequately trained in thermal modeling techniques?
5. What do we need to train the next generation of permafrost engineers for this changing climate?

The questionnaire responses were assembled with the comments and ideas recorded in the workshop minutes, resulting in eight over-arching themes which generally capture all the issues discussed. The following sections summarize those themes and provide an outline of the issues. The themes are listed in subjective order that reflects the level of importance perceived during the discussion. While a few of the themes can be considered as technical challenges, for example remote detection of in situ conditions related to engineering properties, a few of the themes are deterministic challenges, such as the forecasting of permafrost temperatures or the need for predictive capabilities to identify locations sensitive to degradation related geo-instabilities.

The themes identified:
1) Climate Warming
2) Site Characterization
3) Standards and Procedures
4) Linear Infrastructure
5) Vertical Infrastructure
6) Seismic Considerations
7) Thermal Modeling
8) Education
Climate Warming: Noting that permafrost (ice) temperature is so vitally important to so many aspects of long-term foundation design, particularly with adfreeze piles, the participants were most concerned that the profession does not yet have a codified method to allow for long-term forecasting of permafrost temperatures. It was clearly identified that many methods are being utilized across the profession, from simple linear regression of long-term air temperature to utilization of global climate models (GCM), sometimes downscaled to the local region, such as those from the Scenarios Network for Alaska and Arctic Planning (SNAP). There was some consensus: whatever method be derived, it should allow for incorporation of the latest output of GCM’s, a ‘living’ process where ideally the user is not required to input the GCMs model outputs.

Individual roundtable comments:
- “My current work has been using a fragility assessment model for climate warming. Specifically, how do things change within the design as the mean annual air temperature warms? At what temperature is there a tipping point requiring action to continue the serviceability of the infrastructure.”
- “I would also like to see engineers looking at, referencing and, maybe, using the confidence language used by the International Panel on Climate Change (IPCC) to guide clients in understanding the possible consequences to infrastructure from climate change.”
- “In the past, using the warmest extremes when designing permafrost foundations has been a method of designing for a warming climate, without the end-product becoming uneconomic. Now, many of those extremes have become the norm. Because climate is fluid, our design values must be able to change with the times, still reflecting history but making some logical prediction of the future. I offered one method at the 2017 ASCE Conference in Duluth. It’s in the proceedings” Yarmak and Zottola 2017
- “I, and many in our office, rely on SNAP (https://snap.uaf.edu/tools/community-charts) data and estimated forecasts for developing climate freeze/ thaw indices and air temperature forecasts. These values are then used for models to predict permafrost performance and/or seasonal freeze/thaw depths over the design life of a project. However, within our consultation office, this reliance is met with resistance by some. I feel the SNAP (Scenarios Network for Alaska + Arctic Planning) data is defensible, as it is publicly available to all, and can be peer reviewed. I trust the SNAP team is better equipped than the typical consulting engineer to forecast climate data. Perhaps improving the industry’s confidence on the SNAP forecasts could be beneficial.”
- “Alaska Department of Transportation, Northern Regions Material Section (AkDoT-NRMS) has developed an internal practice, but it is somewhat arbitrary and not well defined. As an engineer, if someone at a higher pay grade made the current SNAP 8.5 RCP (representative concentration pathways) model projections for 50-year design life (or whatever) a policy, at least we would all be shooting at the same target. Right now, we may all be using different modelling inputs and assumptions, but we don’t even have a consistent set of goals. Unofficially AkDoT primarily uses (RCP) 6.0”
- Many practitioners stated they were using SNAP data, with a specific comment: “…with SNAP not too much difference using RCP 4.0 or 6.0, or 8.5 until 2040, however after that a big change in results is noticed”
- “Climate models are very good between each other; they were run for 2006 to 2012 and the correlation looks good. Deadhorse MAGT (mean annual ground temperature) at top of permafrost is now -3.5°C, by 2050 it will be 0°C. MAGT is following the MAAT (mean annual air temperature) pretty closely.”
• “Not too many clients looking only until 2040, need much longer projections.”
• “n-values are not valid any longer.”
• “Some folks are simply finding ATDD (annual thawing degree days) from the internet, need to know extreme values as well for worst case scenario.”
• “Maybe taking 3 warmest summers for 30 years might be good enough.”
• “Reality vs budget is +13 (°F) degree change realistic. Should there be a more probabilistic approach to engineering design?”
• “Need to use more of reliability approach to engineering, or fragility assessment. How does risk change with the climate? Use ASCE guidelines for climate adaptation. Hard to justify the higher cost of over-engineering over 50 years.”
• “We use a weighted approach; we’ll give the client a 50% reduction in over-engineering at possibly a 20% reduction in the overall cost.”

**Site Characterization:** Noting ice content is so vital to determine the ultimate design method, and can vary by an order of magnitude within meters laterally and vertically, there was discussion of the need for better methods of site characterization.

**Individual roundtable comments:**
- “The two major technology gaps I see during my research. Effective detection of ice-rich permafrost or ice wedges. The development of infrastructure such as roads, runways, pipelines, or structures and climate change will inevitably alter the original ground conditions and cause degradation in underlying permafrost. Effectively and accurately detecting the existence of ice-rich permafrost or ice-wedges is essential for engineering activities in permafrost regions. Drilling and sampling can be very reliable but is very expensive. Innovative and cost-effective technologies are needed for site exploration.”
- “Good characterization upfront, need to understand ice content. Also need to provide a good idea of water movement.”
- “FHWA understands that geotechnical investigations are not up-to-date.”
- “What are the geophysical techniques or other emergent technologies?”
- “Micro-gravity is used to find large masses of buried ice.”
- “Need more innovation to data gathered from SPT (standard penetration test).”
- “Geophysical methods (ground penetrating radar and electrical resistivity) are good for ice-rich permafrost, not good for soil type. Need to better understand how the CCR can be used to guide the drilling. Often it is over-sold and it underperforms.”
- “Can get a lot of this information from good terrain mapping.”
- “Getting good results from terrain mapping, from a good geologist.”
- “Geophysics needs to be more of tool, used by educated people.”
- “Need better guidance on geophysics applications to frozen ground engineering.”
- “More work needs to be done to correlate geophysics with soil type, salinity, ice content, thawed vs frozen.”
- “There needs to be more work to understand the best methods for characterization”
- “This includes drilling type and methods. Correlating DPT in cold permafrost is not very useful. Notice big correlation with vegetation and drainage.”
- “First rule is avoidance (ground ice) to eliminate future problems.”
- “Lidar or drone-based elevation mapping.”
- “Combining terrain mapping with geophysics and geotechnical for community planning.”
- “Is near-surface seismic a possible useful method?”
• “Micro geo-phone, for seismic refraction.”

Considerable work has been conducted in this topic area of surface-based geophysics, including the detection of ground ice content, and the direct and indirect correlations to the frozen terrain (Bjella et al 2015), (Arcone and Bjella 2016), (Bjella 2014), (Holloway and Lewkowicz 2019), (Calmels et al. 2018). In particular, an exhaustive analysis was conducted at 1.0m borehole spacing, collecting moisture content and grain size analysis every 60cm with comparison to electrical resistivity methods (Bjella et al, 2020).

**Standards and Procedures:** Various issues were raised regarding miscellaneous standards and procedures. In particular, the questionnaire inquired on the practitioner’s confidence in current thaw consolidation (thaw strain) analysis techniques.

**Individual roundtable comments:**
- “There should be design standards in the US for building on permafrost. I understand that some standards exist in other places. They should be adopted circum-Arctic.”
- “During pile design, really only need to know permafrost temperature, soil type does not really matter.”
- “Need better test data for strength, thaw consolidation, Yuri Shur and Misha Kanevskiy have thaw consolidation curves.”
- “North Slope clients do not like to pay for geotech.”
- “Consolidation and thaw settlement is done in the lab, Crory using e values.”
- “Need work on thaw consolidation and also liquefaction potential and lateral stability of permafrost (bearing capacity and creep) of warm permafrost soils.”
- “Some natural materials can simulate insulation properties, such as peat, something with great thermal offset.”
- “Need mechanical properties of warm permafrost, all types, and saline.”
- “Need new technologies to cool the ground”
- “How do you design for longer term freezing needs? Like Kotzebue hospital thermo piles are already set up with an internal hybrid heat exchanger for future needs.”
- “What technology exists to predict these slope failures?”

**Linear Structures:** The public perception is that linear structures on permafrost can be economically constructed with minimal maintenance. However, maintenance costs of existing permafrost roadway infrastructure can be high in comparison to its non-permafrost counterparts. For example, Permafrost sections of the Alaska Highway in Yukon Territory Canada have maintenance costs of $20,000 CAD higher per kilometer than that of non-permafrost sections. While engineers most likely can design for minimal maintenance, the public may not be able to afford the capital cost of such infrastructure. There are no panaceas or silver bullets that engineers can use for designing linear structures over permafrost due to the natural inhomogeneity of the material. The concerns related to changing permafrost conditions with regards to linear infrastructure are real and currently being felt. These infrastructures have a high exposure to damage from a changing climate given their continuous interaction with the ground surface. Although not rigorously discussed during the workshop, the authors suggest much greater attention should be focused on identifying subsurface water flow and the methods for mitigating and prevention (de Grandpre’, 2012)

**Questionnaire response:**
- Air Convection Embankment (ACE) Shoulders have the potential to preserve and possible recreate permafrost under road embankments. Additional research is required to determine efficient design parameters:
What is the minimum thickness of ACE shoulders?
What is the minimum height of ACE shoulders?
Is insulation board needed within the road embankment?
What is the optimal placement depth from the road surface for the insulation board?
How is the effectiveness of ACE shoulders affected by ACE fill material characteristics?
  ▪ Rounded (alluvial gravel) vs. angular ACE fill?
  ▪ Gradation of ACE Fill – how does this affect intrinsic permeability and therefore the effectiveness of the ACE shoulder?
  ▪ What is the minimum size range of material that will function as ACE?
• How does the air temperature regime at the site affect all of the above?

Questionnaire response:
• The following is written with a focus on roads. We know where a road overlies ice-rich ground; we shake our fingers at the road, and say, “You simply cannot design a traditional embankment over ice-rich permafrost!” But inexpensive, out-of-the-box design techniques to build the road correctly to preserve the permafrost do not exist or are not in the hands of the ADOT&PF designers. The focus on better designs comes and goes, and right now there is a loss of institutional knowledge on even identifying the problem (folks who knew retired).

Individual roundtable comments:
• “Discontinuous zones it is very hard to preserve PF with roadways.”
• “It comes down to economics.”
• “DoT already is letting maintenance take over.”
• “Installation of insulation in roadways in dis-continuous zone is not useful, does not allow for cold to enter the embankment.”
• “Data is still coming from Thompson drive, and need to reanalyze Thompson Drive and take a second look and need the engineering community to publish the data.”
• “Insulation will slow down the thawing if done correctly.”
• “9-Mile Hill (Dalton Highway) is having big problems with ‘Hell Holes’ and shoulders sloughing.”
• “We design for 15% failure of pavement, not known to most people. So this is built in somewhat.”
• “Transport Canada is funding for mitigation techniques. Heat drains and ACE.”
• “DoT is looking at performance criteria. Do you include PF and life cycle costs for roadways over permafrost?”

Vertical Structures: Founding vertical structures on thaw stable material is the best option to maintain stability. Most often, this is impossible or extremely expensive in remote locations. Vertical structures founded at-grade with thermosyphons, post and pad, ad freeze piles, Triodetic Foundations, etc. all rely on the preservation of permafrost. With changes in the permafrost thermal regime from climate change, settlement of the foundations is likely for existing structures and the predicting settlement is uncertain for planned structures. The settlement may exceed the design limits before the design life of the structure has been met. Advances in permafrost characterization are providing high resolution and continuous mapping of the subsurface. These techniques can be used to great efficacy to preferentially locate structures on lower ice content areas, or when conditions are technically feasible, can provide the information required to cost analyze for the complete removal of the high ice content soils/rock, preparing sites as ice-free. It is noted research...
needs to be conducted on effective methods for rehabilitation of thaw-affected structures.

**Individual roundtable comments:**
- “The Army Corps of Engineers are the only buildings that spec zero settlement, most other construction is designed as flexible.”
- “Same for the slope and oil companies, flexible bridges and camps and so forth.”
- “What other foundation styles could be utilized for a village home, not expensive pile foundations?”
- “If you build a home that is rigid and the foundation adjustable, when it needs re-leveling, the locals do not have the ability to do so.”

**Seismic Evaluation:** There were only sporadic field studies of S-wave distribution in permafrost (e.g., Cox et al., 2012). The numerical simulation results from Yang et al. (2011) demonstrate that the presence of permafrost and the thawed, unconsolidated surface layer can significantly alter local soil amplification due to impedance contrast between them. To date, no recorded ground motions data have been collected on permafrost-dominated soil profile, nor an assessment of site-specific seismic hazard has been conducted for such soil conditions. Overall the authors feel much research work is needed in this area to insure life-safety is accounted for and needless over-engineering is not occurring.

**Questionnaire response:**
- Seismic hazard identification and mitigation in permafrost in light of changing conditions. With increasing development in permafrost regions with substantial seismicity, it is essential to consider the potential seismic hazard and use engineering measures to mitigate the seismic hazard. How to effectively identify the potential seismic hazard in a permafrost site? And how to mitigate the potential hazards? For example, it is often an option to pre-thaw a site with warm permafrost for infrastructure development. What is the potential for settlement due to thaw and consolidation? How to classify such site for seismic site response analyses? What is the potential of liquefaction during seismic event? How to design ground improvement measures for such a site? Literature in this area is rare, if any.

**Questionnaire response:**
- Vibratory loads on permafrost foundations is one area that could use advancement. Typically, it seems, we apply a large factor of safety due to the limited understanding of full-scale foundation applications. Advancement in soil-pile interaction under long-term vibratory loads may add confidence in the designs, as well as reduce overbuilding. This would be a good topic to get Sivan Parameswaran’s input (who is active with CRED). It’s been several years since I did a project with vibratory loads, however, at that time, we relied heavily on Sivan’s modeled pile work. Or perhaps industry guidance on the best methods available today could be developed?

**Individual roundtable comments:**
- “Literature suggests fundamental properties, p-wave, elasticity is understated in the literature for geophysics, need work in this area.”
- “Seismic issues are important from summer to winter, as the hinge point changes with season.”
- “Warm permafrost can liquefy due to unfrozen water content -2°C, coarse-grained.”
- “Yang worked on the seismic effects to structures, tall structures are less affected than short structures. In the frozen condition, bridges are well documented, however not for other structures?”

**Thermal Modeling:** Thermal modelling is wildly used as a very important tool to predict
Future thermal conditions from climate projections. However, the methodologies used vary significantly by application and no or few standards have been developed to-date to provide consistency and guidance. In addition, each software has its limitations and must be considered in understand the results.

**Questionnaire response:**
- When I have done thermal modeling, I have relied on others who know more about how to estimate warming (such as SNAP). As an example, when doing modeling at AKDoT for a rural airport, I used one of the SNAP scenarios to modify the air temperature function to look 50 years into the design life of the embankment. I also used the current ground temperatures for the area in question, and let the model predict how the thermal regime would change.

**Questionnaire response:**
- Margaret’s (Darrow) modelling paper could be used as a standard by which we define contracted services. At least then, we could rest assured there is some consistency off the bat. I suggest some type of deliverable where input parameters and assumptions are clearly defined (maybe a table format), with a justification of any controversial or highly sensitive assumptions. Also, a sensitivity analysis, at least at a basic level. Kevin—your observations were right on about that—arguing over hundredths is probably meaningless for most input. Sensitivity to heterogeneity of the PF, or of solar orientation, hydraulic heat transfer could be orders of magnitude more significant.

**Individual roundtable comments:**
- “Need links between the thermal and mechanical behaviors of permafrost.”
- “Need spatial variability data in individual parameters (i.e. moisture content, permafrost temperature, albedo, etc.) as opposed to aggregate parameters (i.e. active layer depth)
- “Guidance on the use of and variability within climate projections.”
- “Need data for stochastic analyses of permafrost (random field finite element analyses in thermal modelling).”
- “More research is needed in this area. I am still trying to figure out which model or models are more trustworthy.”
- Vladimir Romanovsky presents data on the North Slope where Deadhorse and West Dock will warm to 0°C from now at -4.0° or -3.5°C.
- Vladimir presents (permamap.gi.alaska.edu), using borehole data from 33 USGS and Permafrost Lab boreholes, then modeled with RCP 4.5 or 8.5, summer n=1.5 gravel, nothing for winter. Soil moisture is constant, Styrofoam (Vlad can’t remember thickness, maybe 4 inches). Uses SNAP 700m grid cell, downscaled to four ecotypes per kilometer. Each cell is calibrated to the boreholes, then upscaled to 700m (~1Km) which is much less computation time. Can change ecotype to ‘infrastructure’ and then have inferences for the built environment. Currently ecotype does not have moisture included, but it could. Currently this is a USGS project. There is ecotype map for Dalton and have completed that project, not released yet.”
- “Need ecotype map for the rest of Alaska, not just Torre (Jorgenson) map of the N. Slope, and also need shallow borehole data to fill south of the Brooks Range.”
- “This would be good for Elliot, Steese, Richardson, Tok Cutoff.”

**Education:** As the cadre of engineers who designed the existing infrastructure built on permafrost retires, replacements are needed to maintain, renew, replace, and extend that infrastructure for the public to use in the future. As communities grow, the needs of the public will
also grow, and the reliability of infrastructure need to increase. Because of warming due to climate change, permafrost may not be naturally sustainable. There will be challenges for the engineers of tomorrow that we today cannot fathom.

**Individual roundtable comments:**

- “Lack of interest in Arctic engineering coming from new students.”
- “The institutional knowledge is held by the materials section. Design only lays out the road, if the budget is busted, design goes back to Materials for another solution.”
- “Need to get undergraduates involved in permafrost.”
- “You can provide guidance but developing a code on geotech sampling is probably not in the works.”
- “Conoco had very ambitious project to get cores down to 600 feet, cores donated to UAA and kept at +15F.”
- “Industry is pushing for entry level as a graduate, this is where you get the geo interest.”
- “Knowing the subsurface is not an undergraduate level topic.”
- “In general, there is less interest in Arctic Engineering. John (Zarling?) helped develop Arctic Engineering class.”
- “PF and Arctic engineering should be discussed at the ASCE level to understand continued education and bring new engineers on board.”
- “PF engineering is not interesting any longer.”
- “Perhaps recommending PYRN (Permafrost Young Researchers Network) get more involved with permafrost engineering.”
- “Who will fund the teaching of the permafrost engineers.”
- “When does the discussion of geology come into the discussion, to better inform the probabilistic methods? Need more geology integrated into the curriculum.”
- “Students don’t even know what geotech is, many engineers do not care about how it was formed.”
- “Need to know more about geomorphology of permafrost and geotech variability.”
- “Permafrost engineering should stay within civil, probably. We need support from above to have an Arctic Engineering program.”
- “Lots of interest in geophysics science and permafrost, lots of interest in biology and carbon. Hard core permafrost scientist is becoming a minority.”
- “Not many jobs require a knowledge of permafrost engineering.”
- “It is not a requirement, and most education comes from OTJ training.”
- “How do we train for vocational skills in the villages.”
- “Utilize the 13 campuses to teach the local teachers on vocations, and they teach the village students.”
- [www.uaf.edu/permafrost](http://www.uaf.edu/permafrost)
- “UAF lost M.S. in Arctic Engineering, maybe could be revived with new chancellor, but there is no interest.”
- “Cannot grow Arctic Engineering knowledge, you need to develop it over time.”
- “Develop advance placement Engineering courses.”
- “Project Lead the Way.”
CONCLUSION

While it was hoped this workshop would provide consensus on identification of the issues, and solid direction to begin solving those issues, in the end this effort raised a lot of questions and very few answers. The authors endeavor to hold a follow-on panel discussion at this conference (RCOP/ICCRE2021) with the goal of identifying the consensus key issues. The group should seek follow-on funding from the National Science Foundation, Navigating the New Arctic program to suggest direction in research and education. Additionally, public outreach and awareness of the potential problems of permafrost warming should be conducted by the group.

Participants (*Chair)
Matt Billings – AkDoT
Matt Bray – UAF
Heather Brooks – BGC Engineering
Billy Conner – Alaska University Transportation Center (AUTC)
Pepe Croft – Shannon and Wilson
Jeff Curry – AkDoT
Margaret Darrow – UAF
Doug Goering – UAF
Phil Hoffman – Alyeska Pipeline (Also, I believe that Phil has passed away.)
Torsten Mayberger – PND
Steve McGroarty – AkDoT
John Rajek – U.S. Army Corps of Engineers – Alaska District (CEPOA)
Vladimir Romanovsky - UAF
Chuck Shultz – Alyeska
Yuri Shur – UAF
Robin Garber Slaught – Cold Climate Housing Research Center (CCHRC)
Frank Wuttig – Alyeska
Joey Yang – University of Alaska Anchorage (UAA)
Ed Yarmak – Arctic Foundations
John Zarling – University of Alaska Fairbanks (UAF)
*Kevin Bjella – Cold Regions Research and Engineering Laboratory (CRREL)

REFERENCES


Arctic Expeditionary Infrastructure Research

Kevin Bjella, P.E., M.ASCE1; Rosa T. Affleck, Ph.D.2; Lynette Barna3; Justine Yu4; Daniel Vandervort5; and Andrew Margules6

1U.S. Army Engineer Research and Development Center, Cold Regions Research and Engineering Laboratory, Alaska Research Office, Fairbanks, AK (corresponding author). E-mail: kevin.bjella@usace.army.mil
2U.S. Army Engineer Research and Development Center, Cold Regions Research and Engineering Laboratory, Hanover, NH. E-mail: rosa.t.affleck@usace.army.mil
3U.S. Army Engineer Research and Development Center, Cold Regions Research and Engineering Laboratory, Hanover, NH. E-mail: lynette.a.barna@usace.army.mil
4U.S. Army Engineer Research and Development Center, Construction Engineering Research Laboratory, Champaign, IL. E-mail: justine.a.yu@usace.army.mil
5U.S. Army Engineer Research and Development Center, Cold Regions Research and Engineering Laboratory, Alaska Research Office, Fairbanks, AK. E-mail: daniel.t.vandervort@usace.army.mil
6CRREL Liaison to U.S. Northern Command, U.S. Army Engineer Research and Development Center, Cold Regions Research and Engineering Laboratory, Hanover, NH. E-mail: andrew.m.margules.civ@mail.mil

ABSTRACT

The warming of high latitude regions is causing geopolitical concerns and spurring increased human presence across the Arctic. Potentially these situations require only a short-term occupation facilitating the need for tested and developed expedient infrastructure. Operating requirements for high latitude conditions are vastly divergent from temperate locations and parameters have been established for habitable conditions to -60°F, withstand 100 mph wind speed, and support 25 lb/ft² snow load. Although great advances have been made in providing efficient and comfortable Arctic infrastructure since the onset of the Cold War, significant work remains to further increase efficiencies, and adapt to changing climate parameters. To address infrastructure technology gaps, the U.S. Army Corps of Engineers-Engineering Research and Development Center (USACE-ERDC) has established an Arctic Infrastructure Research Group (AIRG). Current members comprise U.S. Department of Defense (DoD) and federal agency researchers, program managers, and shelter end users. The purpose of the AIRG is to provide a forum to synchronize research activities and pursue needs, ideas, and technical projects. Current ERDC efforts include the development of an external insulation wrap for rigid wall shelters, mobile insulation system for energy reduction (MISER), to include ground cover for permafrost protection. Additionally, a parallel effort is initiated to fully characterize the effects of the extreme environments on several expeditionary structures at the Cold Regions Research and Engineering Laboratory (ERDC-CRREL) Farmers Loop Permafrost Experiment Station (FLPES), in Fairbanks, Alaska. This initiative has created a research centric facility to test new technologies and innovative materials while validating and verifying the requirements of Arctic hardened infrastructure, both vertical and horizontal, particularly for the DoD.

INTRODUCTION

The Arctic geopolitical condition is rapidly evolving and is noted as a strategic area for U.S.
national interests (Conway 2017; Waller 2011) and of international concern (Boulègue 2019; Depledge et al 2019; Heininen et al 2020). The 2019 Department of Defense (DoD) Arctic Strategy states “DoD’s end state for the Arctic is a secure and stable region where U.S. national interests are safeguarded, and nations work cooperatively to address shared challenges”. Additionally, continued climate warming is causing detrimental and costly problems, forcing the rethinking of long-standing infrastructure design practices. Large DoD facilities exist in Alaska (sub-Arctic latitudes) and in Greenland (high Arctic latitude), with mid-sized supporting installations located in the high latitudes of Alaska and Canada. To accommodate the evolving Arctic DoD requirements, established installations are repurposing existing facilities and also constructing new infrastructure. Of paramount importance, is the concurrent and ongoing requirement for survivable expeditionary infrastructure to accommodate rapid force projection to any Arctic location (Conway 2017; Waller 2011) as temporary housing or encampments while permanent facilities may be constructed. Although DoD has made great advances in providing efficient and comfortable Arctic infrastructure since the onset of the Cold War, significant work remains to further increase construction and operational/energy efficiency and adapt to changing climate parameters.

Various commercial manufacturers of expeditionary shelters exist for accommodating operational encampment requirements. However, these encampments had been assessed primarily in temperate climates and thus requirements for current cold regions capability has been lagging. Given the regional military shift, preliminary evaluations of selected commercial shelters have been fielded in a few Arctic military exercises. However thorough comparative assessments geared toward application of consistent methodologies and parameters are needed to ascertain operational suitability.

DoD stakeholders have commissioned experts in cold regions building technology be involved for holistic evaluation of current capabilities. USACE-ERDC supports the Arctic initiative by addressing new technology for permanent construction, as well as developing and testing infrastructure for expedient Arctic missions. The purpose of this paper is to describe ERDC’s current efforts to address the near-term extreme cold climate requirements for expeditionary infrastructure technology.

CONDITIONS FOR CONSIDERATION AND REQUIREMENTS

Operational planning must consider the unique conditions in the high latitude. These include the challenges of a very cold winter season with durations up to 8 to 9 months of the year. Timing and logistics are key with the short construction and shipping season, and utilities must remain functional and efficient for sustaining life safety. The current DoD requirements are for scalable expeditionary structures to provide rapid construction and deployment with minimal logistical footprint. Ideally this allows for camp establishment in extreme cold temperature with minimal manpower, both deployment and redeployment. High latitude Arctic parameters requires sustaining human habitation at -60°F (-80°F survivable), withstand wind speeds of greater than 100 mph and support 25 lb/ft² snow load. Other important provisions include permafrost considerations to limit thermal degradation and loss of foundation stability, expedient foundations with minimal earthwork, and ease of leveling, while building materials must be strong and lightweight, fire resistant and resist impacts. The general criteria are summarized in Table 1.

Various military transportable encampments have been designed and fielded in the desert and temperate environments for decades (Figure 1). These shelters must be retrofitted or modified to produce a hardened structure for Arctic requirements, where adaptation encompasses unique
challenges for addressing extreme cold, high wind loads, permafrost soils, snow and ice loading, snow drifting, and interior moisture control. Additional requirements include extended periods of darkness and often continuous diurnal cold temperatures during the core winter months. Advances are needed in light weight, high strength, low thermal conductivity structural components to meet harsh temperature extremes while maintaining structural integrity after many freeze/thaw cycles. While energy efficiency is likewise of great importance, retrofit capability is also required to repurpose and reconfigure the expeditionary infrastructure. The need for tested and developed Arctic infrastructure, particularly the more expedient type, is required.

Table 1. Summary of criteria.

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<tr>
<th>Criteria</th>
<th>Description</th>
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<tr>
<td>Affordability</td>
<td>Low cost for operation/maintenance/logistics</td>
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<tr>
<td>Survivability</td>
<td>-60°F; snow loads: minimum value of 25 lb/ft²; and wind speed of greater than 100 mph</td>
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<tr>
<td>Sustainability</td>
<td>permafrost stability, lightweight and high strength materials, low thermal conductivity, and resistance to thermal expansion and contraction</td>
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Figure 1. Configurations for expeditionary structures. a) Modula-S, b) MINATORS, c) Army Force Provider System, and d) Air Force BEAR Base System.

Not only must vertical infrastructure sustain these rigorous requirements, similarly horizontal infrastructure must also be hardened to limit movement due to thermal contraction and expansion, limit the safety and operation issues due to freezing precipitation events, and also provide stable foundation conditions for long term viability.
To expand planning opportunities and build an Arctic operations-focused forum to increase collaboration and information sharing, USACE-ERDC recently launched an Arctic Infrastructure Research Group (AIRG). Current members include ERDC researchers, DoD Branch Services members, Department of Homeland Security personnel, the Corps of Engineers and Districts, and other Federal agency partners, as well as Defence Research and Development Canada. Current stakeholders and collaborators include the North American Aerospace Defense Command (NORAD) and U.S. Northern Command (USNORTHCOM), U.S. Air Force Civil Engineering Center (AFCEC), Naval Facilities Engineering Command (NAVFAC), U.S. Coast Guard Research and Development Center (USCG-RDC), and many others. Each stakeholder has distinct encampment needs unique for their operational missions. The group discussions are centered on DoD-based infrastructure requirements or needs, gaps, and methods to pursue technical advancement across the range of fixed, temporary, or expeditionary infrastructure. The forum enables support by leveraging ERDC research capabilities to develop innovative ideas and formulate solutions for addressing a complex problem for robust infrastructure in a changing Arctic environment. Aside from advancing infrastructure requirements, the AIRG facilitates complementary discussions with science and engineering for information relevant to the Arctic. Lesson learned from expeditionary structures used by private industries in the Arctic, and for civil applications in the Antarctic environment are also pertinent for understanding the operational and logistical rigor needed for military operations.

Testing and Evaluation

Constructing a transportable base camp system in Arctic climates has considerable design aspects to consider. Flat roofs may work well in temperate climates; however, designs are typically advisable where a proper insulated-ventilated sloping roof can help control or prevent the formation of icing or ice dams. Likewise, the founding soils/rock are typically sensitive to heat transfer due to high ground ice contents, therefore thermal de-coupling is often required to limit or prevent permafrost thaw instability. Conventional methods to maintain the permafrost in the frozen condition are very costly in time and manpower, therefore methods are needed to expediently allow for deployment of structures to prevent permafrost degradation, yet reduce the typical amount of earthwork/structure work associated with frozen ground foundations (Bjella, 2020). Flexible insulation systems suitable for protecting the permafrost, yet providing high floor loading are an integral part of the transportable base camp.

Field Demonstration: Various Small Business Innovation Research (SBIR) projects have taken place for demonstrating capabilities in the Arctic expeditionary shelter needs. ERDC can build upon advances made in cold regions science and engineering, in the new materials and methods utilized for these structures. Centered around a joint research facility located in the sub-Arctic (Fairbanks, Alaska), the location of the ERDC-CRREL Alaska Research Office (CRREL-AK), has created an engineering/academic center to foster novel thinking and innovation to solve the evolving Arctic infrastructure and installations needs. CRREL has suitable space to create field research sites and the capability to test for Arctic requirements. CRREL-AK owns 12 acres of land at the Permafrost Tunnel Research Facility and an additional 134 acres at Farmers Loop in Fairbanks, Alaska. In addition, the CRREL Hanover, New Hampshire site has cold rooms to study specific components of the system. Models, systems large and small, components and materials can be tested and evaluated in extreme conditions in controlled, field or in situ conditions.
**Expeditionary Shelters**: Preparing for testing in Fairbanks, CRREL-AK has received a variety of transportable shelters in Fall 2020 for field testing. These include:

1. **Billeting, administration, laundry, latrine and shower modules of the new Arctic variant; the well-established Force Provider system, similar to what is shown in Figure 1. The general configurations of the multi-function Force Provider is made of a rigid wall shelter with dimension of 20 ft long by 20 ft wide and 8 ft high, utilizing folding walls, floors and roofs to triple the shipping volume**

2. **MINATORS** is a structural insulated hard-walled panel shelter system with dimension of 14 ft 9in wide by 14 ft long. This system is developed by Compotech. Inc. (compotechinc.com). They are also multi-functional, re-deployable in small pack volume with no tools or heavy equipment required and with leveling capability.

3. **MODULA S**, funded by a US Air Force AFWERX SBIR, with support from N&NC. The MODULA S, FLEX FX is a shipping container core with flexible casing or outer shell for small building with multi-function purpose (https://www.modula-s.com/).

4. **USAF Expedient Small Asset Protection (ESAP) systems and other expeditionary structures through the Air Force Civil Engineering Center.**

Most of these expeditionary shelters are initially tested at the Permafrost Tunnel location (Figure 2). All these expeditionary shelters have been instrumented for testing under extreme cold weather conditions for at least one winter season and evaluated based on testing criteria. All four systems are being evaluated for energy efficiency, robustness through the winter season with repeated use and foot traffic, air leakage, water leakage, humidity control, and snow loading.

![Figure 2. Sites to accommodate for field testing at Permafrost Tunnel Research Facility.](image)

**Insulation and Ground Cover**: ERDC pursued novel solutions for improving the energy efficiency of Rigid Wall Shelters (RWS) such as the B-Hut which are commonly used in remote locations as sleeping quarters, command and communications offices, sensor and controls buildings, weather stations, and weapon system support equipment storage. The newest of these technologies consists of an insulation wrap system on the exterior of RWS which reduces energy consumption and increases mission performance. This Mobile Insulation System for Energy Reduction (MISER) has shown promise when tested in the very cold climates, including sub-Arctic at the Permafrost Tunnel during winter 2019-2020 and 2020-2019, with a 20 ft. converted ISO structure (Figure 3).

The fundamental concept of the MISER wrap is an easily deployable insulation system...
comprised of modular components that can be erected in the field to create a continuous envelope around the exterior of the shelter, with properties that are appropriate for controlling heat transfer and air infiltration between the shelter and the external environment. This is accomplished via a combination of high-performance thermal insulation, and air-tight joints and interfaces, such that conductive and convective heat transfer mechanisms can be minimized.

A key benefit of an externally applied system is that none of the internal space within the shelter is required for its use, while maintaining appropriate shipping dimensions as applied by the International Organization for Standardization (ISO). Additionally, attachment and interface to the exterior of the shelter is accomplished in such a manner as to avoid any interference with existing external functionality of the shelter.

![Preliminary test on MISER on rigid shelter in Fairbanks, AK.](image)

Table 2. Instrumentation and data collection summary.

<table>
<thead>
<tr>
<th>Sensors</th>
<th>Range</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature</td>
<td>-50° to +70°C</td>
<td></td>
</tr>
<tr>
<td>Temperature/relative humidity</td>
<td>-40° to +60°C; 0 to 100%</td>
<td>±0.6°C; ±5 to 7%</td>
</tr>
<tr>
<td>Moisture</td>
<td>-40° to +60°C</td>
<td></td>
</tr>
<tr>
<td>AC Current</td>
<td>-25° to +55 ° C</td>
<td></td>
</tr>
<tr>
<td>Datalogger</td>
<td>extended range from -55°C to +85°C</td>
<td></td>
</tr>
</tbody>
</table>

Current efforts are investigating applications of MISER technologies to a newer generation of RWS, such as the Arctic variant of the Army Force Provider system (FP MISER kit). The FP MISER kit is designed for use in deployable Arctic environments, tactical situations with priority on reducing heating, ventilation, and air conditioning (HVAC) loads, while maintaining a light weight, small size, and high degree of mobility. The system must be able to be installed and removed in extreme cold temperatures, potentially with high winds, and withstand being struck with airborne debris common in the High-Arctic rocky terrains.

Additionally, this effort will adapt the MISER technology as a ground insulation system as a thermal break or protective layer for the permafrost preservation. The ground insulation system will be designed to maintain the permafrost structural integrity by keeping the soils below freezing temperatures with deployment during the summer months and removed during the winter months. The ground cover extends completely under and several feet outward from the structure on the ground surface around the shelter, manually placed during the onset of the melting season and
removed once the freezing season. During the winter the ground panels will be lifted and added as under-floor insulation and also as additional insulation on the roof.

![Example instrumentation layout as shown on the exterior of the MINATOR. T = temperature, T/Rh = temperature with relative humidity, t = interior wall temperature, d = datalogger, ac = AC current sensor, T1 window, T2 mid-wall, T3 ceiling, T4 floor, T5 near door, T6 in front of heater, T7 exterior1, T8 exterior2, T/Rh1 = temp/rh combo mid ceiling, T/Rh2 = temp/rh combo corner ceiling, ts1 = outside footprint, ts2 = inside footprint, t1 = interior wall temp, ac1 = ac current number needed.](image)

**Figure 4.** Example instrumentation layout as shown on the exterior of the MINATOR. 

**Test Plan**

**Instrumentation:** Various sensors are installed in each type of shelter system to monitor performance during the testing period, at a minimum through one winter. These measurement systems are flexible and rugged enough to withstand the testing conditions at the test site. The sensors include temperature, temperature with relative humidity, moisture (i.e., wetness), energy usage and snow loading (Table 2); all of which will be attached to a datalogger for continuous recording. The sensor layout for each shelter is similar to Figure 4, with multiple placements in each module. Inside and outside the shelter, temperatures are measured with thermistor probes that can be affixed to the walls, floor, and ceiling and roof. Interior humidity will be measured with combination air temperature and relative humidity sensor. The subsurface condition below the modules is monitored with thermistor strings installed in temperature wells.
SUMMARY

To address infrastructure technology knowledge gaps for expeditionary shelters requirements in Arctic conditions, we recently launched an Arctic Infrastructure Research Group (AIRG) comprising over 50 DoD, DHS, and other Federal agency personnel, discussing needs and pathways forward to meet the expectations for all types of Arctic infrastructure. We are working collectively with different agencies in efforts to share information on these different shelters and identify knowledge and technology needing further study. Field demonstration at the Cold Regions Research Engineering Laboratory (CRREL) Permafrost Research Tunnels, and Farmers Loop sites in Fairbanks, AK, commenced in winter 2019-2020 and 2020-2021 to test expeditionary shelters against the requirements. In parallel, three systems currently are tested for energy efficiency, robustness through the winter season with repeated use and foot traffic, air leakage, water leakage, humidity control, and snow loading. One of the Force Provider Arctic Variant modules is completely covered with the MISER insulated wrap and is being evaluated for overall energy reduction as well as the materials or system performance for arctic conditions. Expected innovations will include the use of new materials, insulation wraps, or kit components to improve thermal efficiency, fuel efficiency, net-zero designs, re-usability, and modular systems to modify the shelter specifically to the need.

REFERENCES


ABSTRACT

In the spring of 1975 a geotechnical investigation was accomplished at the proposed building site and included six test holes drilled to a depth of 10.7 meters into warm permafrost. Ground temperature measurements were taken in January 1976 at each test hole. Based on this investigation the decision was made to support the elevated structure on modified H-piles driven to a depth of 9.1 meters. Details of the installation of the foundation piles and a subsequent limited monitoring effort are presented in a prior 2004 technical report by the authors. The primary interest in monitoring the foundation ground temperatures has been to again observe temperature change below the relatively undisturbed ground surface. On July 23, 2020, some 45 years later, the authors had the opportunity to again revisit the site and obtain current ground temperatures at limited accessible pile locations. Although some ground cooling was observed in 2004, it appears that (while still frozen) some slight warming is now occurring. Ideally, evaluation of conditions should be made in the fall in order to better assess full summer warming impacts. Unfortunately, access to most pile locations was limited by “hoar frost” accumulation or other obstructions in the pile angle openings that equipment was not available to allow us to penetrate. Also, the observed past desiccation of the undisturbed thin surface organic mat must have had some influence on reducing ground warming at this site. An assessment of long ago design concerns, as related to current climate warming impacts, are also reviewed.

INTRODUCTION

The Ahtna Corporation Glennallen Facility (Figure 1 and Figure 2) has provided a unique opportunity to review foundation subsurface ground temperature conditions beneath an elevated structure not having a supplemental cooling system. Because of the “warm” permafrost condition the design concept focused on minimizing any surface disturbance of the organic mat by installing the driven piling while the mat was still in a frozen state. Details on the site conditions, pile installation and prior ground temperature measurements are provided in a previous paper presented by the authors (Rooney & Riddle, 2004). This recent site visit (July 23, 2020) allowed us the opportunity to gather current ground temperatures and observe ground surface conditions nearly a half century after the initial investigation.

At the time this project evolved, design criteria for climate projections typically relied on projecting recent past weather history (perhaps last 30 years) forward. At that time concerns with climate warming impacts were being expressed thus it was decided that provisions for potential ground cooling could be required and the angle irons (Figure 3) were attached to the H-piling to allow for future implementation, should it be required. Now with the limited information obtained, we can at least identify the current conditions that indicate some very slight warming overall with more warming at the south end of the structure, where some ground disturbance has occurred due to the addition of a footing foundation which supports a small addition.

Information presented in this paper is based on data personally obtained by the authors with support from R&M Consultants, Inc. and Arctic Foundations, Inc. It was hoped that access to more
pile locations could be attained but only a few more piles were accessed and funding to access more would be necessary. Hopefully, a research project can be initiated in the future as we now design for climate warming impacts foundations supported on permafrost.

Figure 1. Site Location Map.

Figure 2. View of Ahtna Corporation Facility, July 23, 2020.
CURRENT SITE CONDITIONS

Some changes at the site have occurred over the years but mostly at the above mentioned small addition at the south end of the building. Asphalt pavement has been placed near the building perimeter at a few locations and handicap access facilities, supported on footing foundations have been added. The underlying building piling support system does not appear to have had any disturbance from water leakage other than minor effects related to the subsequently installed perimeter footing supports. Pile, test hole and measurement locations are shown in Figure 4.

Figure 4. Thermistor, Pile and Test Hole Location Plan. Temperature at first freezing point depth is also shown.

It was interesting for us to return to the site and observe the thin organic mat and the amount
of desiccation that has occurred. At first, it was thought that in the earlier cooler period for the building, the structure itself was enhancing cooling of the shaded foundation without snow cover, but with climate warming wonder if the dried organic mat is limiting winter ground cooling. Also, while reviewing pile locations, it appeared that some seasonal increase in the thawing active layer depth subsidence was evident on the piling, as opposed to seeing normal piling uplift markings when subjected to seasonal frost heaving.

![Figure 5. Measuring ground temperatures at a pile location. Cuttings from pre-drilling pilot holes prior to driving the piles are evident along with visible shrinkage cracks in the organic mat.](image)

The effort to obtain current ground temperature measurements was limited by the need to minimize the thermal impact on accessing the pile channel angles and it appears that some greater effort will be required to do so and would need more time to allow ground temperatures to re-adjust before taking measurements. There is still an opportunity for such a much more needed effort to do so, considering all the permafrost foundation design concerns that we are now facing. It is hard to understand why so little effort has been made to evaluate other such structures located in marginal warm permafrost conditions.
So at what point does a structure located in “warm” permafrost regions, such as this facility, need to consider implementing a cooling system to retard loss of piling ad-freeze bond and potential creep as phase change occurs with subsequent thawing? This project was developed with concern for some climate warming impact but not to the extent that has occurred since 1976. Current ground temperatures appear to be warming close to those measured in 1976, except for the south section of the structure where thawing now appears to extend to at least one-half the pile length.

EXISTING GROUND THERMAL CONDITIONS

Ground temperatures at the site were made on July 23, 2020 before a planned fall effort that would allow better identification of full active layer depth. The current gathered information has at least provided some insight into slight changes that have occurred since 2004. Access to former pile temperature measurement locations plus several others was obtained. A photo of actual measurement equipment and pile access is shown in Figure 5. Current ground temperature data was retrieved from limited locations on all sides of the building. Most significantly, there appears to be a slight warming close back to the original site conditions found in 1976. One exception was found at the section of the structure where thawing of the warming active layer has extended to a
depth of 5.5 meters. At this point in time, the concern now becomes how much of the thawed active layer will impose an additional downdrag load on the remaining marginally supporting frozen pile?

Figure 7. View of ground conditions below building in June 2004.

The following is a review of ground temperature data gained to date. This effort has been made out of personal interest without research grants but with support from Arctic Foundations, who assisted us in attempting to access the frozen closed pile angle openings. It now appears that additional such access will require some thawing effort that will then need time for the ground temperatures to stabilize.

Of a total number of over 50 piles, only seven pile locations were found to be accessible. This included three on the building’s east side (Piles A-2, A-11 and A-13), two in the center (Piles B6 and B-8) and two on the west side (Piles C-6 and C-15). Pile A-11 had temperatures that were recorded at a depth of approximately 5 to 6 meters at -0.2 °C in January, 1976 and at -0.7 °C in June, 2004. The temperature at this depth in July 2020 was measured at -0.3 °C the same as for Pile A-13. Pile A-2 had a temperature of -0.5 °C at the same depth.

Piles B-8 & B-9, located along the center column row, had temperatures of -0.8 °C at 6.5 meters and -0.7 °C at 5.6 meters, respectively in June 2004. In January 1976 temperature measurements at adjacent pile locations B-7 and B-9 were recorded at -0.7 °C and -0.6 °C at a similar depth. Nearby pile B-13 had a slightly warmer temperature of -0.2 °C at the same depth. Measurements taken in July 2020 at Pile B-8 were found to be -0.5 °C at similar depths.

On the building west side Pile C-5 was found to have a temperature -0.4 °C at just over the 6 meter depth. At Pile C-15, near the south end of the structure, frozen ground was not reached until a depth of 6 meters. This is a significant warming and may be due to the southern sun exposure. Unfortunately, other piles in this area could not be accessed. A plot of pile thermistor readings at selected locations for 1976, 2004 and 2020 are shown in Figure 6.
CONCLUSIONS

As observed at some pile locations, evidence of active layer deepening and resulting ground thawing subsidence was evident. Photos of ground surface conditions below the building in 2004 and 2020 can be seen in Figures 7 and 8. Deeping of the thawing active layer then places an additional load on the remaining supporting frozen pile segment that is now losing adfreeze bond support strength as the frozen ground now warms. With the current observed slight warming impact at the site, it appears that some additional effort may be required to monitor ground temperature conditions and to review and assess the potential need for installing ground cooling devices to retain necessary structural ground foundation support.

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Infrastructure’s Adaptation to Climate Change at the Russian Cold Region’s Territories

Irina V. Chesnokova, D.Sc.1; Alexandra A. Popova, Ph.D.2; Dmitrii O. Sergeev, Ph.D.3; and Gennadii S. Tipenko, Ph.D.4

1Water Problems Institute, Russian Academy of Sciences. E-mail: ichesn@rambler.ru
2Northern Survey Co., Russia. E-mail: PopovaAA@northernsurvey.ru
3Permafrost Laboratory, Institute of Environmental Geosciences. E-mail: sergueevdo@mail.ru
4Permafrost Laboratory, Institute of Environmental Geosciences

ABSTRACT

The apparent trends in average annual ground temperatures are different at different depths. This is due to the multidirectional influence of climatic factors and territorial differences in the vertical distribution of temperatures in rocks. Temperature trends at different depths are used in conjunction with the time indicators of the onset of geocryological events to draft the climate adaptation programs. Adaptation is planned at three main levels. At the national level, zoning of the Arctic is carried out according to the predicted timing of the permanent transformation of geocryological conditions. At the regional level the facilities and responsible companies must be determined. At the local level the engineering protection programs must be developed for each facility under construction or in operation. This makes it possible to take into account the background influence of climatic changes and separate these changes from the influence of the engineering structures.

INTRODUCTION

Regional and microclimatic features of climate change have led to a varying response of the temperature regime of permafrost in different cold regions of Russia, including the sectors and landscapes of the Arctic. Climatic changes in the Arctic are proceeding at a fairly high rate. To assess the risks of changes in the Arctic systems, an assessment of the current and future state of permafrost is of particular importance.

A significant part of the territory of Russia is affected by dramatic climate changes, and the consequences of these changes have a significant impact on the socio-economic development of the country, living conditions and human health, as well as on the state of the economy. According to many years of observations by the Federal Service of Russia for Hydrometeorology and Monitoring of the Environment, the average annual air temperature at the Earth’s surface in the Russian Federation since the mid-1970-s has been increasing by an average of 0.47°C per 10 years, which is over 2.5 times the growth rate of average global air temperature, which is 0.18°C per 10 years [Report…, 2017].

Under these conditions, the adoption of measures to adapt to climate change is absolutely necessary. The ongoing climate change in Russia creates new opportunities for the country’s economy, the use of which also relates to the field of adaptation. What does Russian National Plan mean? This means that economic and social measures have been identified that will be implemented by federal and regional authorities in order to reduce the vulnerability of the Russian population, the economy and natural objects to the effects of climate change, as well as to take advantage of the opportunities arising from such changes.

The solution of the problem using geocryological forecasting as the scientific basis for reducing economic losses associated with anthropogenic impact on permafrost is an interesting
scientific problem, which we have devoted our research to.

In the context of adaptation activities, hazard is defined as a change in the state and/or seasonal variability of permafrost, which is associated with the development of processes that have a physical impact on protected assets (infrastructure facilities, means of production, systems for providing services, ecosystems and natural resources).

The most common hazards arising from climate change include a decrease in the bearing capacity or strength of soils, differential movement of surface and buried structures, gravitational displacement of the soil, and ice formation. The existing or predicted danger associated with the vulnerability of the protected assets, entails the emergence of risk, which should be expressed already in value terms.

INPUT DATA

The main research areas are located in the European sector of the Russian Arctic (Vorkuta region), in Western Siberia (Yamal Peninsula), and in Eastern Siberia (Northern Transbaikalia, Chara region). This makes it possible to assess different climatic conditions for the transformation of permafrost at different latitudes and altitude zones.

For a short-term assessment of the geocryological consequences of climate warming, the data from the monitoring of the permafrost zone are most informative. At present, the continuous permafrost zone appears quite stable in the modern conditions of a changing climate. But climate warming in the future, combined with intense technogenic development, poses a serious long-term danger to the functioning of natural and man-made landscapes in cold regions and the Arctic zone of the Russian Federation.

![Figure 1. Apparent dynamics of mean annual temperatures in “Most-1” site (GTN-P code RU 54 04_0012).](image)

The apparent trends in average annual ground temperatures vary at different depths. If we try to judge the change in ground temperature regime based on data from a specific depth, we will get an incomplete idea of the geocryological conditions. For example, at Most-1 station in Eastern Siberia, according to data obtained from a depth of 5 meters, it seems that the permafrost has...
eventually become colder since 1990, and according to data from a depth of 19.5 meters it is warming up (Fig. 1).

Such a contradiction is apparent and is associated with complex patterns of formation of short-term temperature trends as a result of differences in the dynamics of climate components (thermal resistance of snow cover, surface moisture, and wind).

The deep temperature field of the permafrost also has a strong effect on short-term trends in the temperature regime of permafrost. Different vertical temperature distributions will lead to different permafrost reactions to the same climate change (Fig. 2).

Differences in temperature distribution over depth are associated with the history of the geocryological development of the permafrost in the Pleistocene. The past cold epochs formed a minimum temperature, the presence and depth of which depended on paleo-geographic conditions, thermophysical properties of rocks, and hydrogeological regime. For example, on the Yamal Peninsula, in some landscapes, the minimum ground temperature is located at depths of approximately 40-80 meters.

Modern climate change warms the upper horizons of the ground and the rate of such warming is significantly slower in cases where the temperature minimums in massifs below the depth of zero annual amplitude are very cold (Fig. 2, borehole P_641_2).

The linear temperature distribution along the depth of the frozen strata will lead to its faster warming under climatic or technogenic impacts from the ground surface (Fig. 2, borehole P_1532_Malozemelskaya).

In areas with multi-layer permafrost, the response of the temperature field to the climate signal will be even more complex (Fig. 2, borehole P_1544_Kolvinskaya).

![Figure 2. Different actual vertical temperature distribution will lead to various permafrost reaction to the similar climate change.](image-url)

Thus, to understand the reasons for the different reactions of permafrost to climatic changes, it is not enough to study and model the conditions of heat transfer through the surface. Information is needed on the depth distribution of temperatures and the geocryological history in the
Pleistocene.

Monitoring data obtained as a result of the implementation of the GTN-P program make it possible to calibrate and verify mathematical models of the temperature regime and form a geocryological forecast for the developed landscapes. Such models use the global climate models CanESM and MIROC as input information, as well as regional climate models using the latter data as lateral boundary conditions. The lower boundary conditions are set in the form of the position of the permafrost base and a conditionally constant heat flow, the value of which is set averaged for the tectonic blocks of the accepted structural geologic zoning. Special attention is paid to data on salinity, gas content and other factors of formation of the temperature field of the permafrost.

Control points for carrying out geocryological forecasting are selected taking into account the following criteria. The point:
- is typical for a certain zone of the permafrost distribution;
- is representative for a fairly large area;
- has data available necessary for mathematical calculation for the entire depth of the spread of permafrost and cooled rocks;
- has the actual or planned economic activities in the adjacent territory.

Each layer of permafrost (“Cryolithological Element”) is characterized by a set of the ground properties:
- total moisture content by weight;
- density;
- porosity;
- initial freezing temperature;
- coefficient of thermal conductivity;
- volumetric heat capacity;
- dependence of unfrozen water content on temperature.

DATA PROCESSING

The predicted trends in temperature changes at different depths are used in conjunction with the time indicators of the onset of geocryological events to draft programs for the adaptation of economic activities. An example of such a forecast indicator is the date of a stable transition from merging to non-merging permafrost (Fig. 3).

Non-merging permafrost means the formation of a permanently thawed layer between the permafrost table and the active layer. The existence of such a layer changes the bearing capacity of soils, creates conditions for frost heaving, and changes the hydrologic regime.

In addition to the temperature regime, the results of the geocryological forecast provide information on the geometry of the frozen massif, the boundaries of which change due to climatic influences. The horizontal boundaries of frozen massifs, as a rule, are not provided with sufficient information for forecasting; therefore, the authors focused on forecasting vertical boundaries, which include the top and bottom of permafrost strata. This indicator seems to be the most capacious and the authors extrapolate its values from the points of the geocryological forecast to the adjacent landscapes.

For adaptation purposes, the authors propose to use a new indicator of the state of permafrost soils: the average integral annual content of liquid water in a ten-meter soil layer. A value of 0 for this indicator corresponds to completely frozen soil, which, apparently, occurs only in deep space, and a value of 10 corresponds to constantly thawed soil.

In reality, the processes of seasonal freezing lead to temporary freezing of the upper horizons
and some cooling of the lower layers of permafrost, and a shift in the equilibrium between the content of ice and unfrozen water in the pore space.

The meaning of this indicator is the ability to assess the bearing capacity of the soil based on the phase state of moisture, which depends on temperature, salinity and gas content. The problem is that both the first and second characteristics change over time. Note also that up to now the bearing capacity was considered very simplified in the standards.

The adaptation is planned at three main levels. At the national level, zoning of the Arctic is carried out according to the timing of the transformation of geocryological conditions. The zoning is based on modeling the temperature regime of permafrost for the most representative landscapes. For each region, the timing and nature of a significant transformation of geocryological conditions are determined. In each region, objects and companies responsible for their adaptation are determined.

An adaptation program is drawn up for each facility under construction or in operation. The program is based on data from a local geocryological forecast, which uses the results of a previously performed national level forecast.

In the global view, the infrastructure includes two major types [Paving…, 2010] – 1) Social infrastructure: these projects include buildings and/or other facilities required to provide socially significant services to the population - medical and educational institutions, housing, etc., and 2) Economic Infrastructure: Projects that provide economic growth or sustainability to a region.
Infrastructure can be classified as individual, municipal, or national. Individual level corresponds to points on the geocryological maps. Individual objects are the subject of local permafrost forecast. It includes private houses and critical elements of industrial infrastructure.

The municipal level corresponds to the shapes and polygons on the map and needs to show the zone of the impact. Some areas have cumulative effects on permafrost from the ensemble of buildings and roads. Municipal level is the middle scale.

Third class are the large elements of national infrastructure such as pipelines and railroads. They are corresponding to lines on the map with the buffer zone of the impact. They have different characteristics in their different parts depending on the landscapes and climate zones. Therefore, the linear objects must be segmented in the procedures of the risk estimation.

Similar approaches are planned to be tested at the enterprises of the GAZPROM company [Osipov et al., 2019].

CONCLUSION

Adaptation is implemented in the form of special programs for improving management at the national and regional levels and in the form of special programs for engineering protection at the local level. The presented approach to the use of small-scale modeling data in local models makes it possible to take into account the influence of background climate change and separate these changes from the influence of an engineering structure.

Surface studies are not enough for adaptation programs. Monitoring observations and modeling of the entire permafrost layer are required.

The most complex adaptation programs will be formed for extended linear structures, which are associated with the variety of landscape conditions because of their geometry and location.

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REFERENCES


Improving Construction and Performance of a Runway in Nuiqsut, Alaska

Doug P. Simon, P.E., M.ASCE1; Jeremy R. Dvorak, P.E., M.ASCE2; and Erik J. Jordt, P.E. M.ASCE3

1HDL Engineering Consultants, LLC, Anchorage, AK. E-mail: dsimon@hdlalaska.com
2HDL Engineering Consultants, LLC, Anchorage, AK. E-mail: jdvorak@hdlalaska.com
3HDL Engineering Consultants, LLC, Anchorage, AK. E-mail: ejordt@hdlalaska.com

ABSTRACT

Airports provide the only means of reliable access to many arctic communities in Alaska and are essential for goods, transportation, and medical care. Despite being surfaced with gravel, the runways must provide reliable access year-round. Many of the runways in the North Slope region were constructed in the 1980s, when conventional engineering design indicated 1.3-meter to 2-meter embankments were adequate to protect the runways from settlement of the underlying permafrost. Global climate change has resulted in thaw instability of many runways. Short, wet, cold construction seasons make construction of arctic runways challenging. In addition, gravel surfacing can be difficult to produce locally and is generally of poor quality. This paper presents the solutions used to address the challenges for rehabilitating the gravel runway in Nuiqsut, Alaska. Insulation was installed in the runway embankment to limit the potential for future global warming to thaw the underlying permafrost. Geotextiles accelerated drainage during reconstruction of the embankment fill over the insulation. Dust control additives were blended into the gravel surfacing to increase strength and performance. This paper also presents lessons learned through design and construction. Recommendations are provided for design of future runways, and the value of the insulation, wicking geotextile, and dust control blending is discussed.

INTRODUCTION

Nuiqsut is located 27 kilometers (km) from the Beaufort Sea and 96 km west of Deadhorse at 70°12’ north latitude and 151°00’ west longitude (Figure 1). A seasonal, industry ice road connects the community to the road system in the late winter, but the airport provides the only year-round access. The airport runway is approximately 1,400 meters long, 30 meters wide, and is classified as Airplane Design Group II.

Nuiqsut is located in the arctic climatic zone north of the Brooks Range Mountains and south of the Arctic Ocean. The zone is characterized by long, cold, windy winters and short, cool, cloudy summers. Historical climate summaries for Nuiqsut indicate the average temperatures range from -26.9° Celsius in February to 9.9° Celsius in July. Historical climate summaries for Kuparuk, located approximately 53 km northeast of Nuiqsut, indicate the average annual precipitation is approximately 9.6 centimeters (cm) with an average annual snowfall of 84.1 cm (Alaska Climate Research Center, 2020).

CONSTRUCTION HISTORY

The runway was constructed in the early 1980s using hydraulically placed sands over tundra. The surface of the runway consisted of 30 cm of gravel. The design included a 2% slope from the center to promote drainage.

The subsurface conditions below the runway embankment consist of the tundra organic mat underlain by ice rich soils and ice that could settle if thawed. The at that time state-of-the-
construction indicated that the typical 1.3-meter to 2-meter thick embankments would provide sufficient thermal protection to prevent thaw settlement.

Figure 1. Project Location.

Over time, the crown of the runway disappeared due to routine grading maintenance, snow removal activity, and/or possible thaw settlement of underlying permafrost. Airport inspections conducted in 2015 and 2016 noted surface water collecting at the runway centerline. The trapped water caused the surfacing material to become soft, rutted, and unsafe for aircraft operations that led to periodic runway closures.

This paper illustrates some of the challenges encountered during design and construction and the lessons learned. The authors also provide an opinion on the value of insulation, wicking geotextile, and blending of dust palliative into the surface course.

Figure 2. Ponding along runway centerline.
DESIGN

The design needed to address the following key challenges identified early in the project development process.

**Lack of local gravel sources:** There are no acceptable gravel sources near Nuiqsut, and the nearest gravel pits were approximately 53 km away and only accessible by industry ice road in late winter. In addition, importing aggregates during winter conditions introduces excess moisture into stockpiles.

**Thermal stability:** Thirty-eight borings were drilled to evaluate the embankment soils and the underlying permafrost conditions. PVC was installed in seven borings to accommodate calibrated temperature probe cables to measure soil temperatures at 61-cm vertical intervals. The soils encountered and temperatures recorded confirmed that the underlying permafrost was thaw unstable and additional analysis was required to evaluate the design options.

Models of the airport embankments were developed using Temp/W, a finite element program, to evaluate how the embankments may perform in the future given current and predicted air temperatures. The models were used to:

- Predict the depth of thaw due to predicted changes in air temperature;
- Evaluate the use of insulation to limit the depth of thaw;
- Evaluate the potential for raising the grade to protect the underlying permafrost; and,
- Quantify the potential risks of thermal design option.

The models predicted that thawing temperatures would penetrate through the embankment and organic layer into the underlying permafrost if left unprotected.

**Aircraft safety during construction:** Designing a thermally stable runway required excavations that were deeper than is acceptable for aircraft adjacent to the excavation. In addition, the runway needed to remain open during the day to allow access for cargo, passengers, and emergency response if needed.

![Figure 3. Sketch of runway design.](image)

**Resilience:** The rehabilitated runway needed to provide year-round access for the 20-year design life.

The runway rehabilitation design called for 10.2 cm of rigid insulation installed approximately
100 cm below finished grade. The design required a wicking geotextile over the insulation followed by placement and compaction of the excavated embankment material, and surface application of EK35®, a dust palliative produced by Midwest Industrial Supply Inc. Figure 3 provides a schematic of the design cross section.

The design included 10.2 cm of expanded or extruded polystyrene rigid board insulation with a compressive strength of 275.8 kPa at 10% deformation and R-value of 0.8 °C·m²/W to reduce the heat transmitted into the underlying permafrost. The contractor chose to use expanded polystyrene board insulation for the project. Sheets were overlapped to provide horizontal and vertical offset of the seams. The insulation extended beyond the traveled surface to the edge of the embankment to reduce the potential for thaw settlement at the edge of the runway.

The design called for crushed aggregate surface course (CASC) meeting Alaska Department of Transportation & Public Facilities requirements. Maintenance and snow removal operations tend to move gravel from the crown toward the edge lights over time. Increasing the thickness of the gravel at the crown made the runway more resilient to material loss over time. Thinner layers of CASC were used on the taxiway and apron.

Concerns regarding thermal stability of the underlying permafrost during construction and the relatively short construction season (approximately 100 days) led to the accommodating excavation, placement of the insulation, and backfilling while embankment materials were still frozen. Therefore, the design included wicking geotextile above the insulation and below the fill to promote drainage during thawing. Wicking geotextile mostly consists of a woven geotextile that has unique hydrophilic and hygroscopic wicking yarns that provide enhanced drainage along the plane of the geosynthetic (Nicolon Corporation, 2015).

The design included surface application of EK35® at a minimum rate of 1 liter (L) per 1.5 square meters (m²) – a technique common in the region to limit the loss of fines.

### CONSTRUCTION SEQUENCE

The runway was rehabilitated through a multi-phased project that consisted of three separate contracts. Table 1 summarizes the phases and timeline. The electrical improvements are not discussed further in this paper.

<table>
<thead>
<tr>
<th>Phase</th>
<th>Improvements</th>
<th>Timeline</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Improvements to the generator and electrical equipment buildings</td>
<td>April 2017 to March 2018</td>
</tr>
<tr>
<td></td>
<td>CASC Procurement</td>
<td>October 2017 to April 2018</td>
</tr>
<tr>
<td>II</td>
<td>Insulation, embankment, CASC placement</td>
<td>Summer 2019</td>
</tr>
<tr>
<td></td>
<td>Dust palliative placement</td>
<td>Summer 2020</td>
</tr>
</tbody>
</table>

**Phase I – CASC Procurement:** The design required 52,600 metric tons of CASC to resurface the runway, taxiway, and apron. The selected bid for this project included a unit price of $72.75 per metric ton of CASC delivered and stockpiled in Nuiqsut. The supplier used a material source in Kuparuk River Unit oilfield approximately 53 km from Nuiqsut. This phase also included costs for mobilization/demobilization and a use fee for the industry ice road. The total cost for CASC procurement was approximately $5.7 million.

**Phase 2 – Runway Rehabilitation and Airport Improvements:** Phase 2 improvements primarily included installing insulation board, wicking geotextile, common fill, CASC, and a dust...
palliative material. The contractor started excavating the runway embankment (full width) to a depth of approximately 100 cm in June of 2019. Rigid insulation was installed at the bottom of excavation across the entire width and length of the runway. Installing insulation during summer months adds risk of trapping moisture and heat beneath the insulation. However, the thermal analysis predicted that summer installation would not significantly impact the short-term or long-term thermal stability of the embankments if the length of exposure was minimized. The specifications required insulation be installed within 48-hours after excavation to limit heat gain into the embankment.

Wicking geotextile was placed over the insulation board along the entire width and length of the runway. The geotextile was installed with the long axis of the fibers perpendicular to the runway to promote drainage toward the embankment edge. The geotextile was overlapped a minimum of 30.5 cm to ensure coverage.

Excavated runway embankment material (common excavation) was used for the majority of the backfill over the geotextile and insulation. The first lift of material over the insulation was 60 cm loose depth to protect the insulation from damage during compaction. Subsequent lifts were limited to a maximum of loose thickness of 20 cm to achieve proper compaction. CASC was then placed on the runway surface approximately 30-meters wide and full length. The CASC was compacted to a minimum of 95% of the theoretical maximum density as determined by the modified proctor (ASTM D1557).

![Figure 4. Insulation and geotextile placement.](image)

Runway construction was phased to allow for half-length runway operations during the day. The west half of the runway was rehabilitated between June and July of 2019. Construction switched to the east half of the runway in July 2019. As construction progressed, the CASC toward the bottom of the stockpile became wetter, and later in the season rain events further hampered placement and compaction.

The contractor was unable to complete construction in 2019 and concerns arose about ability of the CASC to support air traffic. During the winter of 2019-2020, the dust palliative application was redesigned from a surface application at a rate of 1 L per 1.5 m² to blending 1 L per 1 m² into the upper 7.6 cm. Additional dust palliative was shipped to Nuiqsut while the industry ice road was open in spring of 2020. Dust palliative work began in June of 2020 and was completed in July of 2020.
CHALLENGES

There are many challenges that are typical of construction in the arctic, North Slope region of Alaska. The following sections discuss the key challenges for the Nuiqsut runway project.

**Crushed Aggregate Surface Course**: The contractor produced the CASC by crushing a gravel material and blending it with a silty sand material. The contractor relied on crushing of fine gravel to achieve the desired fracture count and blending of silty sand to achieve the minimum fines requirement. This process resulted in a product in which the fines content was directly linked to sand portion of the gradation and where the fracture count was difficult to achieve.

During the crushing operations, testing indicated the material being produced did not meet the specified requirements. The contractor took corrective actions and re-processed the material through a profiler and the crushing plant before delivery to Nuiqsut. After the corrective actions, the average of the test results fell within the required specification, but multiple tests had individual sieves that were out of specification. The average fracture count was 77% compared to the required 75%, while the average gradation is shown in Figure 5.

![Figure 5. CASC Gradation](image)

**Stockpile Conditions**: As described in previous sections, the contractor produced and hauled CASC to Nuiqsut during the winter of 2017-2018. During thawed conditions in late spring of 2019, the contractor was able to pull thawed material from the top and outside of the stockpile and place it on the runway in thawed conditions, as required by the design. As the construction season progressed, the contractor exposed the middle of the pile and encountered frozen aggregate. The exposed frozen material began to thaw and saturate the remainder of the stockpile.

The frozen or partially frozen material in the center of the pile was the last material placed on the eastern half of runway and in general had a higher moisture content than the material placed earlier in the project. The elevated moisture content contributed to the challenges experienced near the end of the construction season when the contractor had difficulty meeting the minimum compaction requirements on the taxiway, apron, and a portion of the runway.

**Weather**: The North Slope generally has a short construction season. Low temperatures and rain became more prevalent near the end of the construction season. The low temperatures and wet
weather, in conjunction with the challenges described above, resulted in the contractor having difficulty meeting the minimum compaction requirements on the taxiway, apron, and a portion of the runway. Rain events late in the construction season contributed to the surface course on the east half of the runway becoming loose and muddy, making air traffic operations difficult. The east half of the runway surface was closed to air traffic late in the 2019 season until the CASC froze and could support air traffic.

**EK35 Blend:** Concerns arose about the inability to achieve firm, stable conditions on the taxiway, apron, and portions of the runway near the end of the 2019 construction season. HDL worked with the dust palliative manufacturer to evaluate the performance of the untreated CASC as well as the performance of the CASC blended with dust palliative. CBR testing was performed on the following combinations, and the results are provided in Table 2:

- Untreated surface course;
- Topical application of 1 L per 1 m²;
- Blended application of 1 L per 1 m² to a depth of 7.6 cm;
- Blended application of 1 L per 1 m² to a depth of 10.2 cm;
- Blended application of 1 L per 1 m² to a depth of 7.6 cm at 2% above optimum moisture content to represent the anticipated condition of the surface course;

<table>
<thead>
<tr>
<th>Material</th>
<th>Moisture Content</th>
<th>Initial CBR</th>
<th>CBR @ 30 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated</td>
<td>Optimum</td>
<td>23.4</td>
<td>N/A</td>
</tr>
<tr>
<td>Topical</td>
<td>Optimum</td>
<td>23.6</td>
<td>26.6</td>
</tr>
<tr>
<td>Blended to 7.6 cm</td>
<td>Optimum</td>
<td>27.8</td>
<td>32.0</td>
</tr>
<tr>
<td>Blended to 7.6 cm</td>
<td>Optimum + 2%</td>
<td>19.0</td>
<td>27.0</td>
</tr>
<tr>
<td>Blended to 10.2 cm</td>
<td>Optimum</td>
<td>25.4</td>
<td>34.2</td>
</tr>
</tbody>
</table>

Blending dust palliative to a depth of 7.5 cm was selected to balance performance and cost. Figure 6 shows the proctor curve for the blended material.

![Figure 6. Modified proctor test results for blended material](image_url)

The contractor initially used a soil reclaimer, a water truck, a road grader, and a steel drum compactor to blend dust palliative into the CASC.
Initial blending applications showed signs of CASC segregation (Figure 9) and corrective actions were taken.

As work progressed along the east half of the runway, the compacted, blended material did not behave as anticipated. The blended CASC appeared to be on the dryer side of the optimum
moisture. Follow-up discussions with the manufacturer indicated that the dust palliative could be absorbed into the mineral particles rather than create bonds between particles if the material was below optimum moisture content. The dust palliative process was re-evaluated, additional moisture was added to the blended portion of the runway and taxiway, and additional palliative was applied to the surface.

Use of additional dust palliative on the east half of the runway and taxiway required modification of the application rate throughout the remainder of the runway and apron. Dust palliative was surface-applied only.

**Equipment Challenges:** Equipment failures or breakdowns can cause significant challenges in remote locations because replacement equipment cannot be easily mobilized to the site. A large excavator was damaged during construction of the runway. This resulted in a decrease in production rate and the inability to condition the stockpile for a period of time, which ultimately contributed to challenges encountered late in the construction season.

The steel drum rollers picked up the reconditioned, blended CASC. The sensitive nature of the blended material in conjunction with elevated moisture contents created a tacky substance that would easily stick to other objects.

**LESSONS LEARNED / CONCLUSIONS**

**Insulation:** One concern with placing insulation in an embankment is the potential for damage during construction. The potential for damage appears to be primarily a function of the soils, the thickness of fill over the insulation, and the energy of the compaction equipment. Due to the potential for frozen material, the minimum thickness of embankment material over the insulation was 61 cm prior to compaction with a Caterpillar CS76 vibratory roller. No damage occurred during construction.

**Wicking Geotextile:** In arctic environments, excess moisture enters the embankment soils and gravel surfacing through capillary action and precipitation. In addition, excess moisture may be added to the gravel during winter shipment as snow and ice are picked up with the gravel. The excess moisture makes general construction difficult with compaction being the greatest challenge. Wicking geotextile was included in the design to help control moisture during construction. The geotextile also provides long-term drainage and additional strength to the structural section.

**CASC:** A quality surface course material will compact into a firm and unyielding surface when constructed properly. Two of the critical material properties of a quality CASC are the fracture and fines content. The angular particles interlock and provide the strength necessary for a runway surface. The fine component of the material helps bind the material together. Quality CASC material is difficult to produce in the North Slope region of Alaska.

**Conditioning Stockpiles:** During the initial phases of runway construction in Nuiqsut, the contractor was able to utilize the outer layers of the stockpile that had thawed and drained. After the thawed material had been removed from the stockpile, underlying material was still frozen and contained excess ice and snow that saturated the soils as it thawed. The contractor was forced to transport portions of the stockpile to other staging locations to properly thaw and drain. The additional conditioning process caused delays in construction and ultimately pushed work into the rainy months that typically occur in late summer.

Large material stockpiles will not thaw to their core during a typical summer on the North Slope and require significant effort to condition. Material on the surface of the stockpile that is in workable condition should be removed as soon as practical to allow for underlying materials to thaw and drain. Stockpile management should begin as soon as practical to maximize utilization.
of the drier spring season.

**EK35:** During final inspection, HDL collected limited modulus data along the runway and apron using a Dynatest 3032 lightweight deflectometer (LWD) with 150 mm plate, 10 kg drop weight, and 36 cm drop height. Table 3 provides the measured modulus along the runway and apron.

<table>
<thead>
<tr>
<th>Test</th>
<th>Location</th>
<th>Gravel Thickness (cm)</th>
<th>Treatment</th>
<th>Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Runway East</td>
<td>23</td>
<td>EK35 blended, compacted, two additional surface applications at rate of 1 L/2.5 m²</td>
<td>170</td>
</tr>
<tr>
<td>2</td>
<td>Runway East</td>
<td>23</td>
<td></td>
<td>131</td>
</tr>
<tr>
<td>3</td>
<td>Runway East</td>
<td>23</td>
<td></td>
<td>63</td>
</tr>
<tr>
<td>4</td>
<td>Runway East</td>
<td>23</td>
<td></td>
<td>91</td>
</tr>
<tr>
<td>5</td>
<td>Runway East</td>
<td>23</td>
<td></td>
<td>92</td>
</tr>
<tr>
<td>6</td>
<td>Runway West</td>
<td>23</td>
<td>Condition surface; topically apply at rate of 1 L/1.5 m²</td>
<td>88</td>
</tr>
<tr>
<td>7</td>
<td>Runway West</td>
<td>23</td>
<td></td>
<td>71</td>
</tr>
<tr>
<td>8</td>
<td>Runway West</td>
<td>23</td>
<td></td>
<td>140</td>
</tr>
<tr>
<td>9</td>
<td>Runway West</td>
<td>23</td>
<td></td>
<td>131</td>
</tr>
<tr>
<td>10</td>
<td>Runway West</td>
<td>23</td>
<td></td>
<td>137</td>
</tr>
<tr>
<td>11</td>
<td>Runway West</td>
<td>23</td>
<td></td>
<td>127</td>
</tr>
<tr>
<td>12</td>
<td>Apron</td>
<td>10</td>
<td>Condition surface; topically apply at rate of 1 L/1.5 m²</td>
<td>91</td>
</tr>
<tr>
<td>13</td>
<td>Apron</td>
<td>10</td>
<td></td>
<td>97</td>
</tr>
</tbody>
</table>

The average modulus for the three areas were 109 MPa, 116 MPa, and 97 MPa. The average values were typical for gravel surface course.

The modulus data indicates that the treatment process brought the east portion of the runway up to a comparable performance level as the west portion. The authors expected the east portion to have higher average modulus values than the west but the data appears to indicate otherwise. Additional data would be needed to be certain whether the apparent difference is statistically significant and confirm how the stiffness changes over time.

The use of rubber tire compactors (pneumatic rollers) would be able to perform without picking up material at the surface and would allow for compaction efforts to continue regardless of elevated moisture content.

The authors provide the following opinions.

- While expensive, insulation is relatively light and logistically easier to deliver to remote communities than large quantities of fill. It remains the most economical option for adapting many projects for global climate change.
- As constructed, the blended material was hydrophobic and it was difficult to add additional water and recondition. The soils should be conditioned to optimum moisture prior to dust palliative application.
- The wicking geotextile was more expensive than other geotextile options but functioned well. Since the embankment materials were not as wet as expected, it is difficult to know
if the wicking geotextile was worth the additional cost. However, we recommend its use in situations where embankment drainage is critical.

- Longer, narrower sections of runway were more workable than shorter, wider stretches of runway for dust palliative blending. The road grader was able to make longer passes with the blade and could properly mix the material.
- It is our opinion that using a crushed rock source for future projects will minimize the challenges during the manufacturing process and will provide an overall higher quality product.

ACKNOWLEDGEMENTS

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REFERENCES


Permafrost Degradation Effect on Seismic Response of Bridge Pile Foundation along Qinghai-Tibet Railway

Xiyin Zhang, Ph.D.\(^1\); Xingchong Chen\(^2\); and Jiada Guan\(^3\)

\(^1\)Associate Professor, School of Civil Engineering, Lanzhou Jiaotong Univ., Lanzhou, Gansu, China (corresponding author). E-mail: zhangxiyin@mail.lzjtu.cn; zhangxiyin@lzb.ac.cn
\(^2\)School of Civil Engineering, Lanzhou Jiaotong Univ. E-mail: chenxingchong@263.net
\(^3\)School of Civil Engineering, Lanzhou Jiaotong Univ. E-mail: 1021343656qq.com

ABSTRACT

With climate warming, widespread permafrost degradation is found worldwide in recent years. For permafrost regions located in earthquake belts, i.e., the Qinghai-Tibet Plateau permafrost region in China, permafrost degradation effect on seismic performance of the bridge pile foundation is worthy of research. In this study, a pile-soil interaction model considering thermal-mechanical effect is presented and applied to a case study of the bridge pile foundation along Qinghai-Tibet Railway. Seismic responses of the bridge pile foundation are analyzed with consideration of the thawed permafrost. Numerical results show that thawed permafrost can influence the lateral displacement, shear force, and bending moment of the bridge pile foundation under seismic actions. The freeze-thaw state of the active layer should be considered when analyzing the permafrost degradation effect on seismic responses of bridge piles. The seismic safety of the existing bridge pile foundations along Qinghai-Tibet Railway is evaluated. It is recommended that seismic design of bridge with pile foundation in permafrost region should consider the thermal-mechanical effect of pile-soil system and the permafrost degradation effect.

INTRODUCTION

Permafrost underlies approximately a quarter of the exposed land area of the Northern Hemisphere (Zhang et al. 2008). Some of permafrost regions are in earthquake belts and prone to strong and high frequent earthquakes, i.e. Qinghai-Tibet region in China (Li et al. 2009), Alaska region in America (Yang et al. 2011), and Baikal–Amur region in Russia (Belash and Uzdin 2020). Pile foundation has been widely applied for engineering structures in permafrost region (Che et al. 2014; Nixon and McRoberts 1976; Targulyan et al. 1994), its seismic response is closely related to the pile-surrounding permafrost conditions (Wu et al. 2012). With climate warming, widespread permafrost degradation is found worldwide in recent years (Vasiliev et al. 2020; Wagner et al. 2018). From two major earthquakes in Alaska, i.e. M9.2 Alaska Earthquake in 1964 (Ross et al. 1969) and M 7.9 Denali fault earthquake in 2002(Kayen et al. 2004), it is found that the freezing of ground crust had a great influence on bridge pile foundation behavior during earthquakes (Yang and Zhang 2017). A certain amount of uncertainties will exist in seismic performance of bridges with pile foundation in degraded permafrost regions. However, the potential effects of permafrost degradation induced by climatic warming on seismic response of the bridge pile foundation has not been given attention in the past. In this study, a pile-soil interaction model considering thermal-mechanical effect is presented based on the Japanese railway seismic design code. Because nonlinearity of the component is considered and expressed by the moment curvature relationship in this code (Zhang et al. 2020). A bridge pier with pile foundations along Qinghai-Tibet railway is selected as a case study. Numerical analysis is conducted to investigate the thermal-mechanical effect on the seismic response of bridge pile foundation under permafrost degradation.
NUMERICAL SIMULATION METHOD

A simply supported beam bridge with a span of 32m in Qinghai-Tibet permafrost region is selected as research object in this study, as shown in Figure 1. In permafrost region, elevated pile caps located above ground surface are always used to minimize thermal disturbance to permafrost (Figure 1). It will reduce the lateral strength of the pile foundation when the pile cap is not embedded into soils, which can lead to the potential for unexpected damages under seismic loading (Wang et al. 2016).

Figure 1. Bridge with pile foundation in Qinghai-Tibet permafrost region

A pier with pile foundation of this bridge is selected as reference case to establish the FE model with assistance of MIDAS software. The geometrical sizes of each component of the bridge pier with pile foundation are shown in Figure 2 (a). The dimension unit is meter (m) and the weight unit is ton (t) in this study. The diameter of the bridge pile is 1.15m with a longitudinal reinforcement ratio of 1.2%, 40 longitudinal steels with the diameter of 20mm are uniformly distributed around the circumference. The diameter of the hoop reinforcement is 8mm with the spacing of 200 mm. The concrete cover is 50mm. In FE model, the superstructure weight is simulated as a lumped mass at the top of bridge pier, it is suggested to be 403t for 32m simply supported beam bridge for railways. Pile-soil interaction model of bridge pile was provided in the Japanese railway seismic design code (Institute 1999; Luo 2005), as shown in Figure 2 (b). In this model, soil resistance is assumed to be elastic and ideally plastic. Coefficients of the lateral resistance, side friction and tip resistance of the surrounding soil are defined as functions of modulus of foundation deformation and pile diameter.

These coefficients need to be modified if considering the thermal-mechanical effect. The p-y approach models the lateral and vertical foundation flexibility with distributed p-y springs and associated t-z and q-z springs (Xie et al. 2017). In FE model, the Generalized dynamic Winkler model is applied to simulate the soil-pile interaction system (Allotey and El Naggar 2008). The “p-y method” has been confirmed to be effective in predicting the dynamic response of piles to the lateral spreading of the seismic effect in frozen region (Yang et al. 2013; Zhang et al. 2012). The mechanical properties of frozen soil are related to temperature and have significant effect on the pile-soil interaction in permafrost region (Yang et al. 2015). Therefore, p-y curves for lateral soil spring are both correlated to soil depth and temperature. A p-y curve for frozen silt constructed by Li and Yang (2017) is applied in this study. Thawed permafrost is considered by soil spring coefficient by different temperatures.
NUMERICAL SIMULATION RESULTS

The seismic motion input is considered to be the most uncertain quantity involved in the evaluation of the seismic response of structures (Viti et al. 2019). Because of the lack of strong seismic motion records in permafrost region, the artificial seismic wave record is generated based on historical earthquakes in Qinghal-Tibet plateau region (Cui et al. 2010; Wu et al. 2010). Strong seismic motion with return period of 2475 years is adopted in this study, the exceedance probability of the seismic motion is 2% in 50 years (Zhang et al. 2017). Figure 3 shows the acceleration time histories of the seismic motions, and Figure 4 shows the 5% damped acceleration response spectra of the seismic motions.

Figure 3. Acceleration time histories of the input seismic motions

In this study, four thermal cases have been analyzed, i.e. cold condition before and after permafrost degradation, warm condition before and after degradation. Here, the active layer is 2m depth, which will be frozen in cold condition while unfrozen in warm condition, the depth of...
thawed permafrost is assumed as 8m in this study. Figure 5 shows the sketch of the four thermal cases.

![Figure 4. The 5% damped acceleration response spectra of the input seismic motions](image-url)

![Figure 5. Sketch of four thermal cases](image-url)

One of a pile is selected as an example, and the envelopes of the lateral displacement, shear force, and bending moment of the pile with depth under different cases are analyzed. In these figures, positive values of depth represent above the ground surface, and negative below.

Figures 6(a) and (b) show the lateral displacement of the bridge pile under cold and warm condition. The lateral displacements under warm condition are larger than that under cold condition, it is about 2 times large in pile top. This is mean that the frozen active layer can confine the lateral deformation effectively and consistently decrease the lateral displacement of bridge pile. From Figure. 6(a), the lateral displacement firstly decreases with soil depth, then increases slightly and decreases to nearly zero eventually. In Figure. 6(b), the lateral displacement decreases monotonously with soil depth. The lateral displacements increase after permafrost degradation both in cold and warm condition. It can be found that the permafrost degradation has larger effect on pile displacement under cold condition than warm condition, and the influence depth is about 10m. It means that the permafrost status below 10m-depth has little influence of the lateral displacement during earthquakes.
Figure 6. Lateral displacement of bridge pile

(a) Cold condition  (b) Warm condition

Figure 6. Lateral displacement of bridge pile

Figure 7. Lateral displacement of bridge pile after permafrost degradation

The lateral displacement of bridge pile after permafrost degradation under warm and cold condition in Figure 6 are picked out and plotted in Figure 7. Combined with Figures 5 and 6, it is found that the status of active layer has more pronounced influence on the pile lateral displacement.
than the thawed permafrost. Therefore, it is essential to consider the active layer status when evaluating the permafrost degradation effect on seismic responses of bridge piles under seismic actions.

Figures 8(a) and (b) show the shear force of the bridge pile under cold and warm condition. From Figure 7, the maximum shear force occurs above the ground surface, which means that shear failure is prone to appear at the exposed pile shaft for elevated pile cap widely used in permafrost region. Owe to the freeze and thaw of the active layer, the maximum shear force under cold condition (Figure 7a) is larger than that under warm condition (Figure 8b). Certainly, the unfrozen active layer in the warm condition largely absorbs seismic energy by larger lateral displacement, but when the active layer become frozen in the cold condition, the lateral displacement will be confined (Figures 5a and 5b). It can be found that the shear forces of the pile will be reduced if the surrounding permafrost is thawed, and the reduced effect is more noticeable under warm condition.

![Shear force of bridge pile](image)

**Figure 8. Shear force of bridge pile**

The shear force of the bridge pile under cold and warm condition after permafrost degradation in Figure 8 are picked out and plotted in Figure 9. We can find that the frozen active layer under cold condition will significantly change the shear force distribution of the bridge pile. Because constraints of the pile change from rigid frozen active layer to unfrozen layer due to thawed permafrost, the shear forces change more sharply with depth under cold condition than warm condition.

Figures 10(a) and (b) show the bending moment of the bridge pile under cold and warm condition. Similar as changing of the shear force, the frozen active layer also increases the maximum bending moment of the bridge pile. The peak value of the bending moment is reduced when permafrost degradation occurs. The peak value of the bridge pile reduces from 1581.6 kN·m
to 1513.7 kN·m under cold condition, reduced only 4.4%, while it reduces from 1227.2 kN·m to 1026.1 kN·m under warm condition, reduced 16.4%.

Figure 9. Shear force of bridge pile after permafrost degradation

Figure 10. Bending moment of bridge pile
The bending moment of the bridge pile under cold and warm condition after permafrost degradation in Figure 10 are picked out and plotted in Figure 11. The different status of the soil layer can change not only the value but also the location of the peak bending moment in different conditions. The location of the peak bending moment under warm condition is lower than that cold condition. The depth of the peak bending moment is about 1 m under cold condition and about 3 m under warm condition. It is known that the thawed permafrost weakens the soil strength and rigidity, but the thawed permafrost can play a role in passive energy dissipation during earthquakes, compared with the frozen soil layer in the cold condition.

![Figure 11. Bending moment of bridge pile after permafrost degradation](image)

**SEISMIC SECURITY DISCUSSION**

The pile-soil interaction can greatly affect the dynamic behavior of bridge piers (Spyrakos 1990). The pile foundations and their arrangement affect the distribution of ductility demand of the bridge pier (Kappos and Sextos 2001). Therefore, the seismic responses of the bridge pier are discussed here.

Figures. 12 (a) and (b) show the time history curve of acceleration at the pier-top under cold and warm condition. From Figure. 12, it is shown that permafrost degradation has slight influence on acceleration of the bridge pier. However, when the status of active layer changes, the acceleration at the pier-top will be changed.

As seen in Figure 13, the peak acceleration of the pier-top decreases when the active layer is in unfrozen state (warm condition), from 11.8 m/s² to 9.5 m/s². During the time history, the accelerations of the pier-top vary in non-monotonic way, which increase at some time and decrease at other time.
Figures 14 (a) and (b) show the time history curve of bending moment at the pier-bottom under cold and warm condition. From Figure. 15, it is shown that the bending moment at the pier-bottom has a slight decline under permafrost degradation. When the active layer is in unfrozen state (warm...
condition), the peak bending moment of the pier-bottom decreases significantly. This indicated that the bridge pier is more liable to collapse under cold condition than warm condition.

![Figure 14. Time history curves of bending moment at bridge pier-bottom](image)

(a) Cold condition

(b) Warm condition

Figure 14. Time history curves of bending moment at bridge pier-bottom

![Figure 15. Time history curves of bending moment at bridge pier-bottom](image)

(a) Cold condition

(b) Warm condition

Figure 15. Time history curves of bending moment at bridge pier-bottom

CONCLUSION

To evaluate the effect of permafrost degradation on the seismic responses of the bridge pile in
permafrost regions, a numerical model of a bridge pier-pile-soil system is established based on the Qinghai-Tibet railway. Some conclusions can be drawn:

9) The numerical results show that permafrost degradation can influence the lateral displacement, shear force, and bending moment of the bridge piles under seismic actions. Generally, permafrost degradation reduces the constraints of the frozen soil to pile foundation, which cause an increase in the lateral displacement and decrease in the internal forces.

10) The freeze-thaw state of the active layer should be considered when analyzing the permafrost degradation effect on seismic responses of bridge piles under seismic actions. The permafrost degradation has different impact on lateral responses under different conditions.

11) The permafrost degradation has slight impact on the acceleration of the bridge pier with pile foundation, but the active layer status will significantly change the time history characteristics of acceleration at the bridge pier-top. The bending moment at the pier-bottom indicated that the bridge pier is more liable to collapse under cold condition than warm condition.

In this study, we only analyzed the lateral responses of the bridge pile in permafrost region with consideration of permafrost degradation. Additionally, the permafrost degradation can cause settlement of the bridge pile, which also can influence the seismic performance of bridges with pile foundation. So further research in this area would be needed.

ACKNOWLEDGEMENTS

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REFERENCES


Prediction of Climate Change Impact on a Highway in Warm Permafrost

Yue Zhao1; Zhaohui (Joey) Yang, Ph.D.2; Haibo Liu Ph.D.3; and Changlei Dai, Ph.D.4

1Graduate Student, Univ. of Alaska Anchorage, Anchorage, AK. E-mail: yzhao8@alaska.edu
2Univ. of Alaska Anchorage, Anchorage, AK. E-mail: zyang2@alaska.edu
3Columbia Univ., New York, NY. E-mail: hbl6804@gmail.com
4HeiLongJiang Univ., Harbin, HB. E-mail: daichanglei@126.com

ABSTRACT

Climate warming is causing a widespread impact on the built infrastructure, such as roads, airports, and pipelines, and threaten their safe operation in the broad cold regions. Such problems are particularly severe in areas where ice-rich and thaw-unstable warm permafrost exists extensively. This paper presents the preliminary results of a case study on a highway in Bethel, Alaska, located in deep, warm permafrost in western Alaska. The climate change influence on the thermal state of warm permafrost and its potential impact on the built infrastructure are assessed. We first analyzes the characteristics of the near-surface air temperature predictions from 31 climate models in the fifth phase of the Coupled Model Intercomparison Project (CMIP5) for the next century. The air freezing and thawing indices are evaluated from the climate modeling results and compared with historical data. A thermal model of a selected ice-rich soil profile is built and used to assess the ground temperature variation and permafrost degradation during the next century. Subsequently, permafrost thaw settlement is predicted, and their potential impact on the built transportation infrastructure is discussed.

INTRODUCTION

Permafrost is rock, sediment, or other earth material that has a temperature that remains below 0°C for two or more years. Its thickness increases during colder climatic periods and decreases during warmer periods. In Alaska, on the south of the Yukon River and the south side of the Seward Peninsula, the mean annual surface temperature of discontinuous permafrost are typically warmer than -2°C (Osterkamp and Romanovsky 1999). When the surface temperature is well below 0°C, such as -6°C, permafrost is considered stable. However, if the temperature is close to 0°C, the permafrost is considered warm and fragile. Warm permafrost may thaw in the case of thermal disturbance, and its occurrence and distribution are very sensitive to climate change.

Bethel, AK, is located in the zone of discontinuous warm permafrost area. Price (1972) reported that permafrost thicknesses near Bethel range from 184 m on uplands to 13 m beneath the Kuskokwim River, which was also reflected in the permafrost map prepared by Jorgenson et al. (2008). The large variation in the permafrost thicknesses may be due to local ground water and surface conditions. Under a warming climate, shallow permafrost may thaw and result in widespread ground surface subsidence in areas where near-surface permafrost is ice-rich, affecting drainage patterns, ecological relationships, and the built infrastructure. This paper attempts to assess the climate change influence on the built infrastructure in warm permafrost areas and a highway in Bethel, Alaska, was taken as an example. Air freezing and thawing indices, and thaw depth are investigated.

The mean annual near-surface air temperature (NSAT) data for the next century predicted by in the Coupled Model Intercomparison Project Phase 5 (CMIP5) (Taylor et al., 2012) were retrieved and analyzed to obtain the air freezing index (FI) and thawing index (TI). These indices
were compared with historical data. A thermal model of a selected soil profile with shallow ice-rich permafrost is presented and used to predict the ground temperature evolution, active layer thickness change and ground settlement during the next century.

STUDY SITE

Bethel, Alaska (60.78 N,161.48 W), the hub of over forty Alaska Native villages in southwest Alaska, is located on the alluvial plain of the Kuskokwim River. The city has a subarctic climate with long cold winters and short, mild summers, and a mean annual surface temperature of -0.3 °C based on weather data of 2020. This area is underlain with discontinuous permafrost comprised of recent river deposits, including sands, silts and organics. The study site, i.e., Chief Eddie Hoffman Highway, is the main road connecting the Bethel airport with the rest of the city, as shown in Figure 1.

Deep wells drilled in this area for water exploration were summarized by Feulner and Schupp (1964) and shown in Figure 1. Permafrost at well 2 extends to a depth of 184 m. Waller (1957) reported that a well drilled in downtown Bethel by the Kuskokwin River indicates that permafrost reaches a depth of 115 m. Based on cross section A-A’ from Feulner and Schupp (1964), the permafrost thickness varies from 184m in the west to 115m to the east, and the estimated depth of permafrost at TH 17-24 is 120 m. Dorava and Hogan (1995) reported the deep soils mainly silty sand and sandy silt. Geotechnical investigations were conducted along a segment of the highway to characterize subsurface conditions as part of a major highway repair and pavement preservation project by the Central Region Materials Section of the State of Alaska Department of Transportation and Public Facilities in October 2017 (AKDOT&PF, 2019). Soil were found to be poorly drained in this area. Local soils primarily consist of silt, sand, gravel and organic material. More details regarding the soils will be presented later in this section.
**Table 1. CMIP5 model predictions used in this study**

<table>
<thead>
<tr>
<th>Institute</th>
<th>Institute ID</th>
<th>Model Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Commonwealth Scientific and Industrial Research Organization (CSIRO) and Bureau of Meteorology (BOM), Australia</td>
<td>CSIRO-BOM</td>
<td>ACCESS1.0 ACCESS1.3</td>
</tr>
<tr>
<td>Beijing Climate Center, China Meteorological Administration</td>
<td>BCC</td>
<td>BCC-CSM1.1</td>
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<tr>
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<td>GCESS</td>
<td>BNU-ESM</td>
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<td>CanESM2</td>
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<td>NCAR</td>
<td>CCSM4</td>
</tr>
<tr>
<td>Community Earth System Model Contributors</td>
<td>NSF-DOE-NCAR</td>
<td>CESM1(BGC) CESM1(CAM5)</td>
</tr>
<tr>
<td>Centro Euro-Mediterraneo per I Cambiamenti Climatici</td>
<td>CMCC</td>
<td>CMCC-CESM CMCC-CM CMCC-CMS</td>
</tr>
<tr>
<td>Centre National de Recherches Meteorologiques / Centre Europeen de Recherche et Formation Avancees en Calcul Scientifique</td>
<td>CNRM-CERFACS</td>
<td>CNRM-CM5</td>
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<td>Commonwealth Scientific and Industrial Research Organization in collaboration with Queensland Climate Change Centre of Excellence</td>
<td>CSIRO-QCCCE</td>
<td>CSIRO-Mk3.6.0</td>
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<tr>
<td>EC-EARTH consortium</td>
<td>EC-EARTH</td>
<td>EC-EARTH</td>
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<td>NOAA Geophysical Fluid Dynamics Laboratory</td>
<td>NOAA GFDL</td>
<td>GFDL-CM3 GFDL-ESM2G GFDL-ESM2M</td>
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<td>NASA Goddard Institute for Space Studies</td>
<td>NASA GISS</td>
<td>GISS-E2-H GISS-E2-R</td>
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<tr>
<td>Institute for Numerical Mathematics</td>
<td>INM</td>
<td>INM-CM4</td>
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<tr>
<td>Institute Pierre-Simon Laplace</td>
<td>IPSL</td>
<td>IPSL-CM5A-LR IPSL-CM5A-MR IPSL-CM5B-LR</td>
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<tr>
<td>Japan Agency for Marine-Earth Science and Technology, Atmosphere and Ocean Research Institute (The University of Tokyo), and National Institute for Environmental Studies</td>
<td>MIROC</td>
<td>MIROC-ESM MIROC-ESM-CHEM</td>
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<td>Atmosphere and Ocean Research Institute (The University of Tokyo), National Institute for Environmental Studies, and Japan Agency for Marine-Earth Science and Technology</td>
<td>MIROC</td>
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<tr>
<td>Max-Planck-Institute for Meteorology (Max Planck Institute for Meteorology)</td>
<td>MPI-M</td>
<td>MPI-ESM-MR MPI-ESM-LR</td>
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<tr>
<td>Meteorological Research Institute</td>
<td>MRI</td>
<td>MRI-CGCM3 MRI-ESM1</td>
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<tr>
<td>Norwegian Climate Centre</td>
<td>NCC</td>
<td>NorESM1-M</td>
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</table>
METHODS

Climate Inputs: The historical NSAT data set recorded at the Bethel airport station dating back to 1950 was obtained from the National Oceanic and Atmospheric Administration (NOAA). For future NSAT scenarios, this study used the CMIP5 project’s prediction, which is a standard experimental framework for studying the output of coupled atmosphere-ocean general circulation models (Taylor et al. 2012). The CMIP5 project provided predictions for two-time scales: near-term (until 2035) and long-term (until 2100 and beyond). The data is available from the new Earth System Grid Federation (ESGF) peer-to-peer(P2P) enterprise system.

Table 1 lists a total of 31 CMIP5 climate models covering the Bethel region using the Representative concentration pathways (RCP) 8.5 scenario (PCMDI 2012). Note that the RCP 8.5 is considered the most probable scenario according to current consensus regarding greenhouse gas emission levels (IPCC 2014). The data spans 94 years, from 2006 to 2100. The models’ output in three types of calendar formats was normalized to the 365-day calendar, ignoring leap years. Once normalized, the average daily NSAT is determined, followed by the average monthly NSAT and the annual mean NSAT.

Freezing and Thawing Indices: The freezing index (FI) and thawing index (TI) are important parameters for engineering design in cold regions. The FI (or TI) is to measure the combined duration and magnitude of below-freezing (or above-thawing) temperatures occurring during any given season (Andersland and Ladanyi 2004). The usual design value is determined by the average air freezing (or thawing) index of the three coldest winters (or warmest summers) during the most recent 30 years of record. If 30 years of record are not available, the index for the coldest winter (or warmest summer) during the most recent 10-year period may be used.

The air FI and TI were calculated from the NSAT obtained from historical and CMIP5 data. First, the mean monthly NSAT was determined for each month except the changeover month. Note that any spring or autumn month that includes a seasonal maximum or minimum is called a changeover month (Andersland and Ladanyi 2004). Then, the mean monthly NSAT value was multiplied with the days of each month to find the monthly contributions. Next, the daily NSAT data was used to calculate the exact values for degree-days of freezing and thawing for the changeover months. And the monthly contributions of freezing degree days (thawing degree days) and the freezing degree days (thawing degree days) for the changeover months within a year were summed to arrive at the air freezing (thawing) index for that year.

Soil Profile: A total of 31 boreholes which depths from 5.6 m to 9.1 m were drilled to investigate the subsurface conditions (AKDOT&PF 2019). Figure 2 presents the soil moisture content variations with depth for boreholes drilled along the study site. A profile along the study site with shallow ice-rich permafrost was selected to demonstrate the impact of climate change on the built infrastructure. Specifically, the one with the largest thickness-weighted average moisture content was used to select an ice-rich profile. Among the available borehole data, TH 17-24 has a weighted moisture of 131.1%, the distributions of surface temperature, and its soil layers are shown in Figure 2. The measured mean surface temperature for the highway is about -0.5 °C, which is considered the warm permafrost. The observed range of active layer thickness was 3.05 m to 4.42 m in 2017.

Thermal Modeling: A 1D-TEMP/W finite element thermal model (GeoStudio Manual 2014) was generated based on the geotechnical data to assess the impact of climatic conditions on ground temperature variation. Soil layers and their index properties from the shallow borehole TH 17-24 were combined with soil information from the deep well reported by Feulner and Schupp (1964) to establish the soil profile for model construction. Note that no index properties were available.
from the deep wells; instead, deep permafrost testing data from Kannon (2019) was referenced in estimating the index properties. Table 2 lists the soil layers and their thermal properties. For each soil layer, the volumetric heat capacities of soil layers at both frozen and unfrozen states were calculated by the weighted average method (Andersland and Ladanyi 2004). The thermal conductivity values for both frozen and unfrozen states were determined based on the charts provided in Johansen (1977). The average daily NSAT was used as the model input on the ground surface-air boundary. For this study, the ground cover was asphalt pavement, and the pavement-air interface was modeled by using a thawing n-factor (nt) of 1.45 and a freezing n-factor (nf) of 0.5 (Andersland and Ladanyi 2004). The initial ground temperature conditions was assumed to decrease from -3°C at the top to 0°C at the bottom of the permafrost linearly. At the bottom permafrost, a geothermal gradient of 2°C/100 m was applied.

The thermal status was modeled from 2000 to 2017 based on recorded daily average NSAT data and from 2018 to 2099 based on predicted NSAT data. The model results with the historical data agree well with the observed active layer thickness (AKDOT&PF 2019) and the ground temperature distribution observed in 2017, which serves as the validation of the thermal model. The analysis time step was set to one day to provide daily data for 365 days to analyze temperature changes with depth over time.

![Figure 2. Moisture content distribution vs. depth, temperature distribution vs. depth, and soil profile for ice-rich borehole at the study site.](image)

**Predicted Strain:** Nelson et al. (1983) compiled an extensive database of thaw-consolidation strains of shallow permafrost samples collected along the Trans Alaska Pipeline and proposed an empirical model for estimating thaw settlement using simple index properties. This equation is presented below:

\[
\varepsilon_{TC} = A_1 n^2 + A_2 n + A_3 \frac{n^2 w}{S} + A_4 \frac{n}{S} + A_5 \frac{n}{w} + A_6
\]  

(1)
where: $\varepsilon_{TC}$ is the predicted thaw-consolidation strain, $n$ is the porosity, $\omega$ is the gravimetric moisture content, $S$ is the degree of saturation, and $A_1$ through $A_6$ are regression coefficients. They obtained regression coefficients for different soil types, which are listed in Table 3 for completeness. This model is applicable for predicting strains across a shallow depth of about 15 m. This model was adopted in this study as thaw consolidation model was not available for deep permafrost thaw. For silt layer, a simple model was developed to determine thaw strain; for this model strain was only dependent on dry density (Kannon 2019). Paper combines these two methods to predicted strain. Table 4 lists the predicted thaw-consolidation strain values for each layer of the study site.

The settlement, $\delta$, was determined by accumulating the incremental settlement occurred in each time interval, according to Eq. (2):

$$\delta = \sum \gamma_i \Delta \xi_i$$

where $\gamma_i$ and $\Delta \xi_i$ are the thaw-consolidation strain and predicted thaw penetration increment at the $i$th soil layer within each time interval, which is five years in this study. Note that the model was not adjusted for thaw consolidation at each time interval.

### RESULTS AND DISCUSSIONS

**Mean Annual Near-Surface Air Temperature:** The observed mean annual NSAT from 1950 to 2020, and the mean annual NSAT predicted by the 31 CMIP5 climate models from 2006 to 2099 are depicted in Figure 3. The trend of the historical data, or a line of best fit of the historical data, and the average of the mean annual NSAT from all models are also presented for comparison. Although substantial variation exists in the annual NSAT predicted by models worldwide, their
average aligns with the historical trend surprisingly well. One can see a consistent warming trend in the mean annual NSAT from 1950 to 2020, increasing by almost 0.4 °C per ten years in Bethel, Alaska. The climate models reflect the worst-case scenario emission and predicted accelerated warming for the next century, with the predicted mean annual NSAT increasing from 0 °C in 2020 to 6.1 °C in 2099, or 0.74 °C per ten years.

Table 4. Predicted thaw-consolidation strain for each soil layer

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Depth of Soil Layer (m)</th>
<th>Soil Type</th>
<th>w (%)</th>
<th>e</th>
<th>n</th>
<th>S (%)</th>
<th>Predicted strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0-0.6</td>
<td>SP-SM</td>
<td>3.7</td>
<td>0.33</td>
<td>30</td>
<td>0.25</td>
<td>0.03</td>
</tr>
<tr>
<td>2</td>
<td>0.6-2.1</td>
<td>SM</td>
<td>16.1</td>
<td>0.72</td>
<td>60</td>
<td>0.42</td>
<td>7.2</td>
</tr>
<tr>
<td>3</td>
<td>1.0-2.3</td>
<td>Pt</td>
<td>138.4</td>
<td>2.08</td>
<td>100</td>
<td>0.67</td>
<td>20.6</td>
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<tr>
<td>4</td>
<td>2.3-2.5</td>
<td>SM/OL</td>
<td>58.4</td>
<td>1.28</td>
<td>100</td>
<td>0.56</td>
<td>8.6</td>
</tr>
<tr>
<td>5</td>
<td>2.5-3.8</td>
<td>Pt</td>
<td>70.4</td>
<td>1.06</td>
<td>100</td>
<td>0.51</td>
<td>3.6</td>
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<tr>
<td>6</td>
<td>3.7-4.4</td>
<td>OL</td>
<td>84.0</td>
<td>1.85</td>
<td>100</td>
<td>0.65</td>
<td>17.8</td>
</tr>
<tr>
<td>7</td>
<td>4.4-6.1</td>
<td>SP</td>
<td>22.5</td>
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<td>75</td>
<td>0.44</td>
<td>3.3</td>
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<tr>
<td>8</td>
<td>6.1-35.0</td>
<td>SM</td>
<td>20.0</td>
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<td>65</td>
<td>0.45</td>
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<td>9</td>
<td>35.0-47.0</td>
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<td>0.74</td>
<td>85</td>
<td>0.43</td>
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<td>10</td>
<td>47.0-120.0</td>
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<td>0.82</td>
<td>65</td>
<td>0.45</td>
<td>8.9</td>
</tr>
</tbody>
</table>

Figure 3. Historical and CMIP5 climate models predicted mean annual NSAT results from 1950 to 2100 at the study site.

Freezing and Thawing Indices: The variation of historical and predicted FI and TI vs. time for the study site is depicted in Figure 4. Substantial scattering exists in the historical FI and TI, even if changeover month data are used for calculation. However, the predicted FI and TI are less noisy since they were evaluated by using the average of 31 models. As shown, the FI has been in
a downward trend. And this trend is predicted to continue in the next century, dropping from 1328 \(^o\)C-days in 2021 to 231 \(^o\)C-days by 2099, or a reduction of almost 83%. As a result of the predicted warming, the number of days below freezing decreases, and that above freezing increases leading to larger TI. The TI is predicted to increase from 1585 \(^o\)C-days in 2021 to 2450 \(^o\)C-days by 2099, or 55%.

**Active Layer Thickness:** The 1D-Temp/W (GeoStudio Manual 2014) was used to predict how the ground temperature changes with climate change in the next century. The results were used to determine the thaw depth of permafrost, or the thickness of the active layer. Temperature data was extracted from 62 locations along a vertical line extending to 120 m deep at any given time step. And a MATLAB code was developed to process and interpolate the raw data to provide a contour map of the temperature field within each year.

Figure 4. Historical and Predicted Freezing and Thawing Indices.

Figure 5 shows the temperature fields for selected years with a zero-degree isotherm identified for thaw depth detection. It is seen from Figure 5 that the depth of thaw increases steadily. The thaw penetrates to 33.3 m deep by 2050, and most of the warm permafrost in the model is predicted to thaw by the end of this century. It should be noted that this prediction is for the conditions at the study site, where the surface has asphalt pavement, and if the predicted air temperature is reliable. It should be noted that in other part of the town where intact tundra exists, the depth of permafrost extends up to 184 m and it may not degrade as severely as the study site. Figure 6 depicts thaw penetration variation with time. The observed active layer thickness from 2017 was shown in this figure for reference. This figure shows that the thaw penetration steadily increase with time.
Figure 5. Evolution of ground temperature with time as predicted by thermal modeling.

**Thaw Settlement:** Thaw settlement is just one of the many consequences due to warm permafrost degradation. Figure 7 visualizes how the ground settlement and its annual rate vary with time due to the thickening of the active layer. With 2020 as the reference year, the ground surface of an ice-rich site is predicted to settle by 2.5 m in 2050, and up to 7.5 m in 2099. The rate exhibits an upward trend, indicating accelerated settlement with time. A smooth curve was fitted with the settlement rate data, which shows that, on average, the settlement rate increases from 19.3
mm/yr in 2020 to 35.7mm/yr in 2050. As mentioned before, the prediction model used in this study is applicable to a shallow thaw depth of about 15 m. Therefore the predicted thaw settlement is valid until around 2050. As the thaw depth deepens, settlement due to consolidation caused by self-weight will increase, and an appropriate thaw-consolidation model should be used to evaluate the settlement.

Figure 6. Variation of predicted active layer thickness with time for Bethel, Alaska.

Figure 7. Predicted settlement and settlement rate at the study site due to degradation of permafrost beneath a paved highway.
Impact on Build Infrastructures: Extensive settlement and distresses and damage have been occurring to the built infrastructure and buildings in the study area, as illustrated by two examples in Figure 8. The example on the top shows extensive differential settlement along Chief Eddie Hoffman Highway, where excessive settlement occurred in pockets with ice-rich permafrost at shallow depths and in segments where culverts existed. This damage prompted a recent major rehabilitation of this highway (DOT&PF 2019). The example at the bottom of July of 2018 depicts the excessive settlement along the pathway and the tilting flagstaff by the U.S. Post Office building. It is noted that differential settlement and pavement cracking also occurred in the parking lot but not shown in the photo. Due to the increasing concerns of damage to the buildings and other infrastructure, foundation cooling measures such as thermosyphons have been installed at various locations. If the climate warms as predicted, the ground settlement is predicted to continue to occur during this rest of this century. How we can more efficiently adapt to climate change and mitigate its associated hazards to the built infrastructure is a big question for engineers to ponder.

Figure 8. Observed damage due to permafrost thaw in Bethel, Alaska. L: Differential settlement on Chief Eddie Hoffman Highway (AKDOT&PF 2017). R: Settlement in the walkway and tilting flagstaff at the U.S. Post Office (Courtesy of R. Mitchells, Golder Associates, Inc.)

SUMMARY

This paper attempts to assess the climate warming effects on warm permafrost and its impact on the built infrastructure for this century. It first presents the climate-model predicted near-surface air temperature of Bethel, Alaska. The temperature data from 31 models in the Coupled Model Intercomparison Project (Phase 5) were analyzed to predict how the freezing and thawing indices will change in the coming years. Moreover, Chief Eddie Hoffman Highway in Bethel, Alaska, was taken as an example to predict how the depth of thaw and the ground surface settlement change with time. On average, the mean annual near-surface air temperature is predicted to increase 0 °C in 2020 to 6.1 °C in 2099, or 0.74 °C per ten years, which is faster than the historical warming rate of 0.40°C per ten years observed form 1950 to 2020. By the end of this century, the FI is predicted to drop by almost 83%, and the TI will increase by 55%, due to warmer winters and longer summers.

For the study site, where the surface tundra and organic layer is replaced with asphat pavement, the thaw is predicted to penetrate to 33.3 m deep by 2050, associated with up to 2.5 m in ground settlement in ice-rich sites. The model predicts that most of the warm permafrost will thaw at the specified surface and air temperature conditions, although there are substantial amount of uncertainty associated with the air temperature prediction. It is essential to monitoring the ground thermal status and ground settlement and analyze the performance of existing foundation cooling
measures to derive strategies for efficient mitigation of the associated hazards. Future efforts should also consider the effects of precipitation and vegetation changes associated with climate warming.

REFERENCES


Rethinking Water and Sanitation in Challenging Environments: Lessons Learned from Installing Portable, Adaptable, Mid-Tech Household Systems

Kaitlin J. Mattos¹; John Warren, P.E., M.ASCE²; Mia Heavener, P.E., M.ASCE³; and Karl Linden, Ph.D., M.ASCE⁴

¹Civil, Environmental, and Architectural Engineering Dept. and the Mortenson Center for Global Engineering, Univ. of Colorado, Alaska Native Tribal Health Consortium. E-mail: kaitlin.mattos@colorado.edu
²Alaska Native Tribal Health Consortium, Division of Environmental Health and Engineering. E-mail: jwarren@anthc.org
³Alaska Native Tribal Health Consortium, Division of Environmental Health and Engineering. E-mail: mia.heavener@anthc.org
⁴Civil, Environmental, and Architectural Engineering Dept. and the Mortenson Center for Global Engineering. E-mail: karl.linden@colorado.edu

ABSTRACT

Permanent water and sanitation infrastructure faces major technical and economic challenges in cold region communities because of the threats of freeze-thaw cycles, permafrost instability, and a changing climate. As a result, thousands of households suffer health and wellbeing consequences because they live without basic access to clean water, safely managed sanitation, and appropriate hygiene. In response, the Alaska Native Tribal Health Consortium has spent five years developing, piloting, and deploying portable, adaptable, mid-tech household water and sanitation systems in rural Alaskan communities that lack piped infrastructure. These portable alternative sanitation systems (PASS) work with natural freeze-thaw cycles to help households manage potable water and human waste. They can be adapted to various modes of operation based on end-user preferences and environmental conditions. These systems require little training and technical expertise to operate and maintain and can be easily moved to new locations if households have to relocate. End users have demonstrated that PASS units can be successful at providing incremental improvements in water and sanitation services to households if they are appropriately designed, installed, and supported. We evaluated function, use, and adoption of PASS units over the first year after their installation. We discuss lessons learned from deploying these innovative mid-tech systems in houses, such as the need to develop basic technical installation and operation guidance and socially appropriate trainings to ensure success of the technology. These lessons can be used to support the development of new types of adaptive and resilient infrastructure with low environmental impacts for underserved communities.

Keywords (3-5): water and sanitation, environmental engineering, low-impact development

INTRODUCTION

Globally, 2.1 billion people lack access to safe drinking water and 4.4 billion people lack access to safely managed sanitation (World Health Organization Joint Monitoring Program 2017). In isolated rural areas and many cold climate communities, centralized and high-tech water and sanitation systems are too expensive (in capital and long-term operating costs) or lack the required operation and maintenance capacity to be sustainable (Hickel et al. 2017; US Arctic Research Commission 2015). Piped water and sewer systems are also beginning to be acknowledged as environmentally wasteful in terms of net energy and mixing of waste products that precludes...
resource recovery (Andersson et al. 2016).

In cold climates, water and sanitation infrastructure construction is challenging because of continuous and discontinuous permafrost, a changing climate that leads to ground instability on the surface and subsurface, degradation of permafrost that leads to challenges servicing infrastructure, and high costs of maintenance and risks for users (Smith and Low 1996). Piped water and sanitation services are non-existent or failing in thousands of rural Alaskan households in over 30 communities (Alaska Department of Environmental Conservation 2017) due to the widely-distributed household units, small population size and relative isolation of villages, and the challenging and costly engineering conditions in cold climates (US Arctic Research Commission 2015). Furthermore, small communities with very small local economies and lack of economies of scale often have a very hard time funding construction, operation and maintenance for centralized water, wastewater and accompanying infrastructure (such as roads, hauling vehicles, waste lagoons and water pumps, Adank et al. 2018). In some recent cases, piped infrastructure funding from outside sources has been denied because the entire community urgently needs to relocate away from environmental hazards (erosion, storm surges, Thomas et al. 2013) that are linked to a changing climate (Marino and Lazrus 2015).

At the other end of the infrastructure engineering spectrum, low-tech systems, such as countertop water treatment systems and pit latrines, often target a minimum level of service and may not provide all of the desired health benefits (Rosa and Clasen 2017), as defined by the United Nation’s Sustainable Development Goals (Rosa 2017). The lack of adequate access to water and sanitation infrastructure for communities in rural Alaska has been linked to higher rates of respiratory, skin and gastrointestinal infections (Gessner 2008; Hennessy et al. 2008; Thomas et al. 2013). Emerging “mid-tech” water and sanitation systems that are adaptable to user needs, resilient to the changing environment, and require only simple operation and maintenance will move communities closer to health and environmental standards.

With the goal of reducing the technical challenges of cold climate water and sanitation infrastructure, engineers have developed and tested a variety of innovative and experimental interventions in rural communities, such as incinerating toilets, composting toilets, household greywater reuse systems, insulated pump-and-haul tanks, and “freeze-proof” septic tanks (Hickel et al. 2017; US Arctic Research Commission 2015). Although many of these interventions represent promising technical solutions to rural water and sanitation issues, most lacked end-user input and social acceptance of the technology. There is a growing acknowledgement in water and wastewater engineering that sound technical infrastructure alone is insufficient for adapting to human needs and providing a long-term solution (Andersson et al. 2016; Eichelberger 2014; Jepson et al. 2017; Kaminsky and Javernick-Will 2012; Murphy et al. 2009). Therefore, the implementation of new interventions must critically evaluate technical and social components of long-term sustainability of the infrastructure. Most of these alternative interventions have been deployed on a household scale and, as a result, must address the significant hurdles of human behavior, perceived costs/benefit/risk, occasional neglect, and low-skill operation and maintenance (Burleson et al. 2019; Montgomery et al. 2009; Murphy et al. 2009). They further must be carefully vetted with end users before implementation (Murphy et al. 2009) to ensure that the start-up effort required for a household to adopt a novel intervention is worth the benefits gained (Burleson et al. 2019). Otherwise, these systems will end up being misused or unused and will not fulfill their intended goal of improving water and sanitation access.

Because of the downsides to traditional piped infrastructure in rural communities, the inadequacies of low-tech options, and the need for adaptive strategies in infrastructure engineering
to respond to climate change, the Alaska Native Tribal Health Consortium (ANTHC) created a mid-tech water and sanitation unit, the Portable Alternative Sanitation System (PASS), to address household needs. These units have been piloted in multiple communities and must demonstrate long-term sustainability in order for homeowners to experience the benefits of services, for funders to support additional projects, and for the technology to be considered for use in other communities in-need. This study evaluates the sustainability and implementation of PASS over the first year after installation in the home to determine initial function, use and adoption rates and their implications for longer-term sustainability.

Sustainability is often discussed in terms of the “triple bottom line” of economic, social and environmental considerations (Elkington 1998). Within the context of water and sanitation infrastructure in developing communities, this triple bottom line can be assessed as infrastructure and operation/maintenance that can be financed for the full design life, that is acceptable to targeted end users, and that protects the environment (Kaminsky 2015; Kaminsky and Javernick-Will 2012). Kaminsky further proposed “technical performance” as a fourth pillar for infrastructure sustainability in 2015. In rural Alaska, where population densities are extremely low and dilution may very well be a sufficient treatment method to reduce human impact on the natural environment, the environmental sustainability of water and sanitation is rarely discussed. While the PASS may represent a significant step toward increased environmental sustainability compared to traditional piped water and wastewater systems because it uses minimal water and energy and creates minimal waste while providing for basic needs, a full environmental assessment of the PASS should be the subject of further study and is not discussed here. Economic sustainability is also excluded from this analysis, because of the specific funding mechanisms that establish water and sanitation infrastructure in rural Alaska Native communities. Briefly, state, and federal government agencies have legal agreements with tribes to provide health services and infrastructure, so the vast majority of capital, operation and maintenance expenses related to water and sanitation infrastructure are not the responsibility of individual homeowners receiving the systems. This paper therefore examines sustainability from the perspective of the social and technical pillars.

We define “long-term sustainability” as use, function, and adoption of PASS units over the first year after installation. Use and function were chosen as two critical components of long-term sustainability because they play important roles in the provision of water and sanitation services (Davis et al. 2019; Kaminsky and Javernick-Will 2012; Montgomery et al. 2009). If units are not in use due to unmet social needs such as lack of ownership, cultural acceptance or capacity, then the system will not improve access to water and sanitation in the home (Andersson et al. 2016; Murphy et al. 2009). If units do not function due to environmental or technical engineering issues, then water and sanitation services are not being provided (Kaminsky and Javernick-Will 2012). We expect function and use to be consistent with one another within specific households at a certain point in time, unless technical or social inputs are missing (Kaminsky 2015). We also evaluated the adoption of new technologies, a measure that incorporates use, function, and the degree to which previous technologies (e.g. unsafe water storage containers or “honeybucket” bucket latrines) are replaced by the new system. We use one year as the minimum time period for follow-up in cold regions because of seasonal changes in the environment and human activities that could have an impact on infrastructure. Although one year is not sufficient to determine long-term function, use and adoption of technology (Mac Mahon and Gill 2018), it is a good indicator of whether a technology makes it through initial seasonal challenges and may continue into the future.
Figure 1: A Portable Alternative Sanitation System is a household water, sanitation and hygiene unit developed by the Alaska Native Tribal Health Consortium in 2015 to provide mid-tech, adaptable and portable options to Alaska Native communities without access to piped water and sanitation in their homes. (Figure not drawn to scale. Image courtesy of ANTHC. Used with permission.)

METHODS
Technology

ANTHC developed PASS (Figure 1) in 2014 to provide an alternative mid-tech option for basic water and sanitation to unserved rural households in cold regions. Water is self-hauled to the home from treated or natural sources or supplied by a rain catchment system. The water system uses an electric pump to push water through a household treatment system containing a Cryptosporidium-rated cartridge filter and an activated carbon filter followed by manual chlorination by a leur-lock chlorine injection point. Treated water is stored in either a 50-gallon or 100-gallon tank which feeds a single low-flow handwashing sink in the home. Drinking or cooking water can be accessed through a spigot directly on the tank. The PASS toilet consists of a waterless urinal and urine-diverting dry toilet as an alternative to the typical honeybuckets or pit latrines (“outhouses”) used for human waste. Greywater from the sink and separated urine from the toilet and urinal are disposed of in an onsite underground seepage pit. A self-regulating 5-watt per foot electric heat trace is installed adjacent to the drain lines to the seepage pit to keep liquid in the line from freezing during the winter months (typically September to April). Feces and toilet paper are collected in a bucket inside the toilet, dried with an electric ventilation fan and disposed of at a landfill or in a burn barrel. The PASS can be divided into two components: the water component consists of the water treatment system, storage tank, handwash sink, and greywater discharge, and the toilet component consists of the urine-diverting dry toilet, urinal, ventilation fan, and urine discharge.
The mid-tech design of PASS incorporates several features targeted for success in remote, rural Alaskan communities. The underground seepage pit is located within the active soil layer and was engineered to work with the natural freeze/thaw cycles of the ground to ensure appropriate treatment and drainage, reducing the need for highly engineered treatment systems or septic tank maintenance. Operation and maintenance of PASS was designed to be simple enough that homeowners would be able to keep it running and do repairs on their own with minimal expertise and easy-to-access parts and tools. The system was also designed to be adaptable, so homeowners can add features and use it in a way that best reflects their household’s needs. For example, PASS has two modified discharge modes. In the event of a freeze-up or clog in the drain lines, the sink and urine drainages can be switched to drain into containers that are manually emptied, to reduce the loss of use of the rest of the system during extreme cold weather or blockages. User-centered design concepts (such as collaborative innovation processes) were employed during a pilot phase, and subsequent models incorporated a sturdier toilet design, reduced barriers to proper operation and maintenance, and increased adaptability. For example, the toilet can be changed from a urine-diverting dry toilet, to a bucket latrine with ventilation (“vented honeybucket”) by removing the separating piece. This allows households that are uncomfortable with the separator to still make use of the improved and ventilated toilet. Finally, the system involves minimal permanent infrastructure (e.g. underground pipes) and is relatively portable, so households can move it within their existing home or communities that may need to relocate due to the threats of climate change can move their water and sanitation system to a new home.

Having a PASS unit in the home is meant to improve health through treatment of natural water sources, safe storage of water, handwashing with flowing water, safer greywater disposal, and safer human waste management. However, supporting infrastructure within the community is needed to ensure that PASS can be operated and used. PASS is capable of treating natural water sources to United States Environmental Protection Agency primary drinking water quality standards (US Environmental Protection Agency 2009). A centralized watering point can further improve access and decrease time for households to haul water to their homes, although in-home water treatment is still important because water can be contaminated during transport in open haul containers. A community laundry and shower facility (called a “washeteria” in rural Alaska) is also necessary to provide access to hygiene activities that would otherwise require large quantities of water hauled to the home. Communities need an appropriate landfill to dispose of partially dried human waste and a method of transportation to haul water and waste to and from the home.

Installation and support

In 2015, nine PASS units were installed in volunteer households as part of a pilot project in a Northwest Arctic Alaska Native community that lacked piped water and sanitation. The units were evaluated after the first year. They were then re-designed in 2016-2017 to reduce complexity, improve durability and to make the systems more adaptable to alternative modes in case of winter operation challenges. In 2017-2018, the pilot homes had their units upgraded if they desired. In 2018, 10 new units were installed across three communities in the Interior region and one community in the Yukon-Kuskokwim Delta (Southwest) region as a second-phase pilot to see whether the unique soil and weather conditions would support the mid-tech design. In 2019, the finalized design was installed in 39 homes in one Northwest Arctic community and one Yukon-Kuskokwim Delta community. In total, 58 PASS units have been installed in six villages in rural Alaska. Major projects to install PASS in additional unpiped homes in three communities are planned for 2020-2021. More unpiped communities are becoming interested in financing PASS...
units as an intermediate step until pipes can be installed in future years.

PASS units installed between 2015-2019 were constructed through ANTHC with federal and state funds allocated through the Indian Health Service’s Sanitation Deficiency System database. Thus, homeowners paid no capital costs for the installation and were only responsible for minor operation and maintenance costs, including the cost (if applicable) of water; transporting water to the home and waste away from the home; electricity demanded by the pump (during tank filling), the heat trace in winter, and the ventilation fan; and consumables such as replacement filters or chlorine.

Because PASS is a unique system, ANTHC staff engaged with homeowners through community meetings and explained with diagrams and 3-D models what PASS was, how it differs from a traditional water and sanitation system, and what is required for operation and maintenance. After construction of PASS, ANTHC staff visited households to train residents on the daily use and care for the unit. Homeowners were also provided with starter kits of bleach, plastic 1mL luer-lock syringes, chlorine test strips, extra filters, a long-form operation manual and a visual quick-start guide. On subsequent visits, homeowners were offered additional retraining if needed, and problems were documented and troubleshooted. ANTHC conducted regular follow-up and warranty assistance for one-year following installation and continued to follow-up informally in subsequent years when feasible.

Data collection and analysis

To collect information on function, use and adoption of PASS units, ANTHC staff (including authors Mattos and Heavener) conducted semi-structured interviews by phone and in person at every follow-up visit and whenever they were in contact with homeowners to determine how PASS units were perceived, used and functioned in the home. Interviews were planned to be conducted at 1-, 3-, 6- and 12-months after installation during home visits, but due to the remoteness of communities and difficulty of travel, many of these interviews were pushed forward or backward or were conducted by telephone when in-person visits were not possible. In some cases, homeowners were not available or declined to participate in interviews.

Homeowners or heads of household were asked whether each component was working and if they had had any mechanical or operational challenges since the last interview. If challenges were mentioned, staff asked whether and how they were resolved. Interviewers were typically in a position to assist homeowners with repairs or report warranty needs, so homeowners may have been incentivized to report all challenges and malfunctions during interviews. Homeowners were also asked how they use the PASS unit, what they thought of the unit overall, and what other water storage, handwashing, and toilet fixtures they used in the home in addition to or instead of PASS.

Data from interviews was qualitatively coded into categories of function, use, and adoption. Function is defined as mechanical and technical soundness. Function measures whether systems were operational and whether needed repairs were completed with available resources and support. Use is defined as the component being regularly put to its designed purpose, solely or in conjunction with other household fixtures (e.g. supplemental water storage containers, handwash basins, honeybuckets or outhouses). Use measures end user acceptance, ease of adjustment to the new system, and preference for PASS over prior water and sanitation practices and fixtures. Function and use codes from each interview were summarized into ordinal values of high, moderate, or low at three time points – during February (usually the coldest month) of the first winter that the unit was installed (usually 1-3 months after installation), approximately 6 months after installation, and approximately 12 months after installation.
Adoption is defined as end-user acceptance and near-continuous long-term use. End-user acceptance encompasses social criteria such as comfort, dignity, and preferences. Near-continuous long-term use requires that systems be sufficiently funded, maintained and functioning at their full technical capacity, and used (Eder et al. 2015). The specification of “near-continuous” is included to account for seasonal changes of occupancy of homes and adaptation to the constantly changing environmental conditions at high latitudes. Adoption measures the degree to which households replace their previous water and sanitation fixtures with their new PASS unit. Data from each interview reflecting adoption was summarized into four ordinal classes of “full”, “high”, “partial”, and “rejector”. Full adopters used the PASS water system without supplemental water storage, used their PASS handwashing sink in place of their handwash basin, and used the PASS toilet as their sole toilet. High adopters used the PASS water system for some water uses but still made use of supplemental water storage and used modified discharge modes of urine and greywater occasionally. Partial adopters used the PASS water or toilet components occasionally or for specific uses but heavily supplemented their water with other water storage and made use of honeybuckets or outhouses commonly instead of the PASS toilet. Households were also considered partial adopters if they adopted either the water component or the toilet component, but not both. Rejectors (non-adopters) either uninstalled both the water and toilet systems or kept them in their home completely unused.

RESULTS AND DISCUSSION

Of the 58 households who received PASS units between 2015-2019, 32 households in five communities gave written consent for interviews and participated in the research described here. Household size ranged from 1-13 occupants. Figure 2 shows combined use and function of a subset of PASS units over their first year after installation. Although consistent data from many of these households is available more than four times in the year, some homes were unavailable to discuss their systems with interviewers more than once or twice. Some communities had challenges with communications (e.g. phone lines were down) or were inaccessible for in-person visits (e.g. flights were infrequent and prohibitively expensive), and thus those households received less support and less frequent follow-ups for their systems.

Water components (including the water treatment system, storage tank, handwashing sink and greywater discharge to pit) were more likely to stay functioning and in use over time than toilets, possibly because the water component required less behavioral adaptation compared to traditional water tank and sink systems, while the urine-separating dry toilet was a significant change from honeybuckets, outhouses and flush toilets that users had experienced before, and had a number of new features to navigate for trouble-shooting. It was common for function and use to change over the year, for example if there was a technical issue that took the homeowner or ANTHC some time to address (e.g. Figure 2, H11), or if there were freeze-up issues during winter but the system returned to normal functioning when temperatures rose (e.g. Figure 2, H9). Households most frequently used the water component for treating rainwater, handwashing, and occasionally to provide drinking or cooking water, although PASS units were rarely the exclusive use of potable water in most homes. Major functional challenges for the water component included seasonal freeze-up of the drain line or failing to turn on the heat trace in winter. Use challenges for the water component included households disliking the low flow of the sink and residents forgetting or not knowing how to fill the tank or clean the system.
Figure 2: Function and use of the toilet and water components of PASS for a subset of sixteen households (H1 through H16) over the first 13 months after installation.

The toilet component was either used in place of the honeybuckets that some households used previously or used as a secondary toilet to outhouses during inclement weather or late at night. Major functional challenges for the toilet component included leaking connections, ventilation issues, and seasonal freeze-up of the drain lines resulting in clogs or slow drainage. Use challenges for the toilet component included residents disliking the urine-separator and people finding the toilet cumbersome or less comfortable than their previous toilet. Several homes were also...
displeased to receive a system that was not as convenient or simple as piped water and flush toilets, despite attempts by ANTHC staff to explain the system and manage expectations before PASS was installed. Function and use of the units were highly heterogeneous and dependent on the preferences, time, and technical capacity of the individual homes. Some homeowners were determined to keep their system working despite challenges, while others preferred to return to their prior water storage and toilet systems if they had issues with PASS.

Table 1: Function and use of toilet and water components of PASS units during 1st winter, 6-months, and 12-months after installation. Cells show number (N) and percent (%) of homes reporting low, medium, or high function and/or use at each point in time. Some percentages add to 101% to accommodate rounding.

<table>
<thead>
<tr>
<th>Water component (total N = 31)</th>
<th>Toilet component (total N=32)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1&lt;sup&gt;st&lt;/sup&gt; winter</td>
</tr>
<tr>
<td>N</td>
<td>%</td>
</tr>
<tr>
<td>Function</td>
<td>Low</td>
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<td>Med</td>
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<td>Function and use</td>
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<tr>
<td></td>
<td>Med</td>
</tr>
<tr>
<td></td>
<td>High</td>
</tr>
</tbody>
</table>

Function and use of PASS units

Between half and two-thirds of PASS water and toilet components were high-functioning and in high use at each time point (first winter, 6-months, and 12-months), however, individual households fluctuated up and down in both function and use over time. When both function and use were considered together (e.g., a component had to be both functioning and in use), only half of the toilet components and one third of water components were high-functioning and highly used after one year (Table 1). The largest differences in function and use levels were observed between 6- and 12-months after installation, when 16% of homes shifted down from high (-13%) and up from low (-3%) function and use, into the medium function and use category for their water component. After the first year, more than half of households (52%) found a moderate use for the PASS water in their home, often in modified discharge mode or alongside supplemental water storage containers and practices, and a smaller number kept it functioning and in high use (32%). For the toilet component, a 16% shift in function and use was also observed in that time period, with households moving out of the medium category and into either the high (+6%) or low (+9%) categories. This indicates that the water component served a moderate use for homes, but the toilet was an “all or nothing” transition. Households either loved or hated the toilet, and rarely used the adapted vented honeybucket mode. At the end of one year, 84% and 69% of homes had medium or high use of the water component and toilet component, respectively. There was some failure in
We are able to assess whether technical function or social acceptance (use) was the fail point for PASS units by evaluating when the function of a household’s unit and the household’s use of the unit are mismatched. For the 32 homes presented here, level of function matched the level of use – meaning that as long as it was functioning properly, people still used it – in 45% of water components and 69% of toilet components after 12 months (Table 2). In these cases, the technical and social inputs into the system were balanced.

In cases where function exceeded use, the technology is lacking social inputs, such as priority addressment or habit building, that lead to it not being used to its full potential. This means that people were not using the toilets even when they were working. In these cases (35% of water systems and 16% of toilets), it is likely that the system was over-engineered or too new and foreign for what people needed, wanted, and expected. For example, some households did not learn about all of the features of their PASS unit, and therefore were not using them fully.

Alternatively, when use exceeded function (19% of water systems and 16% of toilets), it was an indicator that people were still choosing to adapt the system and use it, despite mechanical challenges. This may also mean that the system was under-engineered for people’s needs or technically insufficient for households. For example, in the winter, households were frequently using the toilet component in adapted modes due to cold weather mechanical issues that were often related to design or construction issues. Technical improvements could have improved the level of function to match the level of use.

### Table 2: Function levels compared to use levels for household PASS water and toilet components at three points in time after installation.

<table>
<thead>
<tr>
<th></th>
<th>Water component (total N = 31)</th>
<th>Toilet component (total N=32)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st winter</td>
<td>6 months</td>
</tr>
<tr>
<td>Function &gt; Use</td>
<td>7 (23%)</td>
<td>8 (26%)</td>
</tr>
<tr>
<td>Function = Use</td>
<td>18 (55%)</td>
<td>18 (55%)</td>
</tr>
<tr>
<td>Function &lt; Use</td>
<td>7 (23%)</td>
<td>6 (19%)</td>
</tr>
</tbody>
</table>

**Adoption**

Four homes (13%) were full adopters of PASS: using the system as their sole water storage, toilet, and hygiene fixtures. Two of the four also made significant modifications on their own to enhance their water access at multiple fixtures (e.g. kitchen sink, shower) in the home. Twelve households (38%) were high adopters of PASS: using the water tank fully for bathroom hygiene but keeping other water storage containers for other purposes and occasionally using the system in a modified discharge mode. Eleven homes (34%) were partial adopters: using the water and toilet components occasionally but heavily supplementing with other fixtures and practices. Of these, two adopted the water component but fully rejected the toilet component because they had too many challenges with it. Three others fully rejected the water component because it was too complicated, and they preferred their current practices for handling water. Five additional homes (16%) fully rejected the entire PASS unit: two uninstalled the system and three left it in the home but completely out of use. Thus, despite mechanical challenges and diverse end-user preference for each of the components, the PASS unit as a whole did fulfill an unmet need for additional water, sanitation and/or hygiene infrastructure in 84% of unpiped households at the end of the first year.
Lessons learned from PASS

Because PASS is a novel technology being implemented in established communities, households experienced various social and technical challenges with the systems. Since units were rolled out in phases over several years and in different communities, program staff had the unique opportunity to gain feedback from homeowners and project personnel and improve the implementation at each step: recruitment, design, construction, and follow-up support. The overall approach for improvements was to discuss challenges among the multi-disciplinary team and evaluate potential technical and social solutions before making changes.

After the first year of the project, staff received significant feedback about the end-user’s impression of the system. Several comments were incorporated into technical design and construction changes, such as building a sturdier toilet, removing a waste storage tank that would often leak, and adding an electric heat trace to keep drain lines from freezing during the winter. Some feedback would have resulted in too complex or costly technical solutions, so social solutions were undertaken (this technique was also observed by Kaminsky [2015] in latrine repair projects). For example, when households expressed disappointment that the system was not a flush toilet as they had expected, engineers did not replace the dry toilet with a flush toilet. Instead, project staff attempted to manage household expectations in future installations, by bringing scale models around so homeowners could visualize the system better, and by placing the statement “This is not a flush toilet” prominently on informational materials. Staff had to communicate and have repeated conversations among themselves and with households to ensure that information was being shared and understood.

PASS also faced challenges with operation and maintenance requirements, which were new to homeowners compared to the low-tech infrastructure they were accustomed to. Again, some of these challenges were solved with technical approaches, such as configuring the filters in such a way that all parts of the system could be easily accessed and cleaned. ANTHC also included a start-up kit of necessary materials, such as cleaning brushes, bags, gloves, soap, and a drain snake, so that households were prepared for maintenance and cleaning activities from the beginning. Simple, visual training materials were developed to guide homeowners through tasks to enable proper operation and to encourage routine minor maintenance. Training was a time- and energy-intensive job that required repeated communication with multiple residents within a household to disseminate information. Hiring local employees within the communities to conduct training was the most efficient way to communicate and build PASS capacity.

In order to ensure technical sufficiency, the PASS team had to come up with basic technical standards for design and installation, and conduct training and inspection oversight from a distance. This was a particular challenge for PASS, and likely for other mid-tech systems, because the technology seems simple and intuitive, but there are nuances to the installation that are required for proper operation. For example, a low-watt heat trace was used to reduce electrical cost to homeowners while preventing freeze-ups. However, the low wattage meant that the heat trace conduit had to be laid perfectly flush against the drain lines to keep them from freezing in the winter, when temperatures would often reach -30° F. When foam insulation created a 1-inch gap between the heat trace conduit and the drain line, the drain lines would freeze and back up into the home. This minor issue may not have been catastrophic for more complex and robust piped systems, but it was a major challenge for PASS. Not only was it a technical vulnerability, but homeowners who had to deal with freeze-up issues were sometimes apt to reject the system after the first winter. To prevent this, project staff and engineers created detailed installation and inspection checklists for each home. Checklists were simple so that staff at any level of expertise...
could evaluate construction work and note additional needs. They also gave staff a concise way to communicate about technical standards. The recruitment of staff who stayed with the PASS project for multiple years also helped to streamline all steps of the project and maintain institutional knowledge.

The portable and adaptable design of PASS is an elegant and necessary response to the challenge of engineering appropriate technology for underserved communities. However, because the unit looks simple and feels like an incremental improvement in water and sanitation for households, many homeowners did not understand or take advantage of the multiple modes of operation. For example, some households moved or remodeled their home, and just left the PASS unit behind, unused. Others experienced pipe freeze-ups and put their old honeybuckets back in front of their PASS toilet, instead of converting the PASS toilet into a modified discharge mode. Project staff attempted different ways to describe these various features in training. At first, the options were not emphasized, so many homeowners didn’t know they existed. Then they were emphasized but too many extra parts were involved in performing the conversion, so the options were not used. Then the parts and training were provided and emphasized, but homeowners were overwhelmed by all of the information. Finally, and after several years, the PASS program has produced sufficient supporting materials and has developed community capacity so that homeowners have options to get support and assistance when they need it. Thus, the initial homeowner training is just the starting point for homeowners to learn about their system. This approach appears to contribute to the success of PASS. For the most recent installation of systems in 2019, 50% were functional (9/18), 38% had only minor or intermittent problems (7/18), and only two out of 18 toilets (11%) were rejected (replaced with honeybuckets) throughout the first winter.

Levels of adoption may be lower than expected for PASS because of the principles of technology diffusion (Straub 2009). For example, many technology adopters may be swayed by the desire to conform to the majority (Burleson et al. 2019), which requires that the technology reach a point where it is widespread in the community. Further, PASS communities are generations deep in their water and sanitation practices without much formal engineered infrastructure. Households have adapted their lifestyles over hundreds of years to their current behaviors. An incremental improvement like PASS may have more challenges to overcome to be fully adopted under those circumstances than a high-tech system, like piped water and flush toilets. One critical challenge is whether PASS is being viewed as a positive improvement in water and sanitation infrastructure or a temporary stop-gap measure that is barely tolerable. The first few PASS communities were just reaching the point where the majority of homes have the system in 2020, six years after the technology was developed. Thus, continued evaluation of PASS adoption in the coming years may show different results than those reported here.

PASS project staff were still trouble-shooting design, installation, training, and end user acceptance challenges in 2020. The system is constantly undergoing minor changes as manufacturing and program implementation attempt to scale-up. However, the whole team has been trained to think about both social and technical options for all challenges. Mid-tech water and sanitation infrastructure projects are likely to need this degree of hands-on implementation in cold climate communities in the future because of the environmental, financial, and historical challenges facing water and sanitation projects.

LIMITATIONS

This study evaluated PASS units at 32 out of the 56 systems installed in five out of six
communities. A greater number of homes would have led to additional insight into technical and social challenges with PASS. Household interviews were the primary method of data collection. Data may be skewed towards or away from specific concerns because of homeowner availability and willingness to discuss issues with interviewers, and additional methods of data collection (such as structured observations or objective sensor data) would have been valuable. This analysis also stopped after approximately 12 months of follow-up with each household because of limited research resources, although longer-term analysis may yield different results. Long-term follow-up on novel technologies for years is needed to fully vet these types of systems. Further analyses not reported here are being conducted to evaluate the commonalities between households that adopt or reject PASS units and individual components to improve the implementation of PASS and other interventions in the future.

CONCLUSION

Mid-tech water and sanitation systems may be able to fill a major need in underserved communities in cold regions. Current health and wellbeing inequities between piped and unpiped communities are growing. In rural Alaska, funding gaps between what is needed and what is being allocated to improve infrastructure in remote communities are also growing (Griffith, and Black 2014). Some communities may be waiting for years to receive fully piped water and sanitation systems. Others will have to relocate due to environmental threats and will have no infrastructure for some period of time. Mid-tech systems may provide some of the health and hygiene benefits of piped infrastructure if they are properly installed and used. If mid-tech options are to be pursued, they must be appropriately piloted. Planning, design, construction, knowledge transfer, capacity building and communication are all critical components. Function, use, and adoption of systems should be evaluated for at least one year to determine technical feasibility and social acceptance of the systems. We provide evidence that evaluating these indicators of use are superior to simply evaluating mechanical functionality, which can be misleading if appropriate social inputs are missing from the implementation.

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Permafrost Test Sites: A Summary of Alaskan Pipeline Industry Efforts in Addressing Frozen Ground and Related Technical Issues

James W. Rooney, P.E., M.ASCE

1Retired; JF Rooney, LLC, Anchorage, AK. E-mail: j.rooney@me.com

ABSTRACT

This paper is an attempt to identify and provide a brief summary of some industry efforts that occurred during the late 1960s on through the mid-1980s, that I was involved with or aware of. All of the test sites were focused on evaluating terrain conditions and assessing potential pipeline impacts that would be involved while dealing with frozen ground conditions. Efforts included various organizations and participation by government agencies that occurred during both the Alyeska Pipeline Service Company (APSC), Arctic gas pipeline, and Alaska northwest natural gas transmission system early project activities. There were at least 11 test sites having locations in various parts of Alaska. These included Barrow, Prudhoe Bay, Prospect Creek, Hess Creek, the Fairbanks area, and Glennallen. They addressed concerns relating to thermal modeling of a hot oil 48-inch pipe buried in frozen ground, thaw settlement/ frost heave effects, trench excavation methods, and testing vertical support solutions for the designated above ground pipeline segments.

INTRODUCTION

During the late 1960’s as the north slope of Alaska oil discovery occurred, there was a need to quickly respond to potential options on how to develop the oil field infrastructure and ship oil from Prudhoe Bay to mainland USA. Pipeline technology in dealing with permafrost and frozen ground was very limited and it was apparent that the industry was not prepared for addressing the complex conditions involved. In 1969, access to the north slope was limited to aircraft and a more than 350 mile overland winter “haul road” trail that was blazed over the frozen terrain by the Alaska Department of Highways, in order to improve access for necessary equipment to get to Prudhoe Bay.

Several options were considered for exporting the oil. These included overland routes, either west or south and questionable seasonal tanker shipping from Prudhoe Bay. Eventually, the proposed route over Atigun Pass in the Brooks Range on down to Valdez was selected. At that point, it became apparent the pipeline route location would become a very critical effort and route selection would require major investigations to locate both the pipeline, pump stations and the required construction support road.

COLD PIPE TEST SITE AT BARROW, ALASKA-1968/69-TAPS:

In 1968 an initial effort was made to access and evaluate large diameter pipeline constructability issues in permafrost terrain and a test site was constructed at Barrow (Utqiagvik), Alaska. This study was focused on trench excavation and pipe placement techniques. The study was conducted by the University of Alaska-Fairbanks (UAF) Environmental Engineering Laboratory for TAPS.
With the need to verify thermal modeling capability for a hot oil buried pipeline, a 800-foot long 48-inch diameter pipe was installed in a uniform silt frozen ground site on the UAF west campus (Figures 1 and 2). Hot air was utilized to simulate anticipated pipeline temperatures and the instrumented site was monitored to record consequent thaw bulb development. This allowed refinement of the thermal modeling for evaluating potential effects of a buried hot oil pipeline.
The study was performed under the direction of UAF Prof. Hal Peyton, who was then working as an independent consultant for TAPS and also working with Exxon’s research group to assist in verifying their thermal modeling effort.

![Figure 3. Photo of Gas Arctic Cold Pipe Test Site at Prudhoe Bay, Alaska.](image)

In anticipation of the potential for transporting natural gas from Alaska through Canada, the Gas Arctic System was formed to evaluate the feasibility of such a project. As part of the Gas Arctic System’s test of the performance of a large diameter gas pipeline under various soil and climatic conditions, a test site facility was constructed at Deadhorse near Prudhoe Bay on the Copper River Basin near Glennallen, Alaska. The study was performed under the direction of UAF Prof. Hal Peyton, who was then working as an independent consultant for TAPS and also working with Exxon’s research group to assist in verifying their thermal modeling effort.

![Figure 4. Photo showing large diameter drilling equipment used for deep holes excavated in the Copper River Basin near Glennallen, Alaska.](image)
Alaskan Coastal Plain (Figure 3). Emphasis was given to constructability issues, including trenching excavation techniques, pipe bedding and backfill needs and assessing soil/pipe interaction. The study was directed by Battelle Columbus Laboratories for Gas Arctic Systems.

GLENNALEN DEEP HOLE SPECIAL SOIL PROFILE STUDY, JUNE, 1970-TAPS

Initial concepts for burial of a hot oil pipeline in permafrost were met with much concern and the need to assess viable construction options had to be determined. Evaluation of frozen ice-rich soil conditions for major segments of the pipeline route became an early effort. The Copper River Basin was a significant large segment of the pipeline route that burial was of concern. The extent of frozen ground ice distribution in the basin needed to be evaluated for all to understand. So, this ‘Special Study’ was initiated to identify subsurface frozen soil conditions along this problematic long section of the route, all of which became above ground pile supported pipeline.

Four 36-inch diameter holes were excavated to a depth of 50 feet in the frozen lacustrine deposit. The purpose of the study was to further evaluate subsurface soil and permafrost conditions in those areas where significant ice distribution and varied orientation had been observed in recovered soil samples. This effort assisted in confirming the need for an above ground pile supported pipeline through the length of this frozen lacustrine deposit. Large diameter drilling equipment used for this study is shown in Figure 4.

Figure 5. Photo showing ice formation and distribution in the frozen clay Copper River Basin soil.

VSM PILE TEST SITE AT CHENA HOT SPRINGS ROAD, FAIRBANKS, ALASKA – 1972/73-TAPS

In an effort to evaluate pile type and installation techniques in permafrost for the proposed above ground vertical support members (VSM), a test site was installed along Chena Hot Springs Road near Fairbanks, Alaska. Both H and pipe piles were evaluated using percussion, vibratory
and pre-drilling techniques. Ad-freeze bond to the piles was assessed for both the driven and slurry-back piles. The study was performed by the Arctic Civil Engineering Section of TAPS with the assistance of their consulting support group.

Figure 6. Thaw Settlement Test Site layout plan

Figure 7. Cross-section of insulated heat pad utilized to thaw the ground to a depth of 30 feet.
THAW SETTLEMENT TEST SITE AT PROSPECT CREEK, ALASKA-1972/74-TAPS

In order to ascertain in situ performance of a specific frozen soil formation, a test site was installed and monitored at Prospect Creek, Alaska. Information with regard to thaw induced ground settlement was an essential requirement in defining acceptable criteria for burying the pipeline in permafrost. The test site consisted of a 50-foot wide by 200-foot section placed in frozen silty granular soil (Figure 6). The instrumented site was artificially thawed to a depth of 25 feet by placement of an electrically heated “hot pad” covered by insulation (Figure 7).

Results of the testing were utilized to confirm consolidation strain values obtained in the laboratory and to investigate the anticipated influence of soil arching. This study was performed by the Arctic Civil Engineering Section of TAPS with the assistance of their consultant support group.

Figure 8: Aerial view of thaw settlement test site on February 26, 1973 (Note that heated area has subsided below the surrounding ground).

Figure 9: Aerial view of Tanana Uplands Test Site Under Construction. (Note the patterns in the snow perpendicular to embankment centerline, indicating where survey personnel obtained ground surface elevation contour data.)
WORK-PAD TEST SITES AT CHENA HOT SPRINGS ROAD AND GLENNALLEN, ALASKA-1972/73-TAPS

Work pad test sections were constructed on “warm” ice-rich permafrost terrain and were operated over two full seasons. Both locations were instrumented in order to allow evaluation of loading and seasonal effects. A total of 18 different work pad test sections, utilizing various construction techniques, materials and varying fill thicknesses, were incorporated into each of the two sites. This study was performed under the direction of the Arctic Civil Engineering Section of TAPS with the assistance of their consultant support group.

![Figure 10: Photo of failed work pad.](image)

SLOPE THERMAL EROSION TEST SITE AT HESS CREEK, ALASKA-1973/74-TAPS

This test site was designed and constructed for the purpose of evaluating frozen soil cut slope erosion in ice-rich silty soil using different slope treatments intended to reduce adverse ground thawing impacts. The site was located near Hess Creek along the Dalton Highway (Figure 11). Potential erosion problems, such as ground subsidence, downslope movement, re-deposition of thaw slope material, headward erosion and gullying were evaluated during the two-year test period. Various test site details are shown in Figures 12 and 13.

Results from the test site study demonstrated that thermal degradation impacts could be
reduced by effective slope treatments. This study was performed under the direction of the Arctic Civil Engineering Section of TAPS with the assistance of their consultant support group.

Figure 12. Cross-section of some of the proposed slope treatments.

Figure 13. Photo of selected slope treatments just after placement in 1973.

In 1984, ADOT&PF performed a research review of slope conditions at this test site and the
results were presented in ADOT/PF Report No. FHWA-AK-RD-85-02 (Mageau and Rooney, 1984). Observations indicated that surface treatments with higher insulation properties can reduce thermal erosion in ice-rich soil for a limited number of thaw seasons.

Figure 14. Photo of test site treatments taken in 1984. Slightly over 10 years after installation.

Figure 15. Chena Hot Springs Road test site, photo was taken in 1979.

GAS PIPELINE TEST SITE AT CHENA HOT SPRINGS ROAD, FAIRBANKS, ALASKA-1978 TO 1982-ANNGTS

As an effort to further understand buried large diameter cold pipeline performance in
permafrost terrain, the ANNGTS project team constructed a major test site on Chena Hot Springs Road. The purpose of the tests was to evaluate the performance of a buried 42-inch diameter chilled steel pipe when subjected to variable ground conditions having soil thaw and frost heave forces.

**GAS PIPELINE UPLIFT/SETTLEMENT TEST SITES AT LITTLE SALCHA, LIVENGOOD, TANANA RIVER, SWEETWATER AND WISEMAN, ALASKA-1981/82-ANNGTS**

To obtain a better understanding of soil/pipe interaction and uplift pressures, the ANNGTS project team constructed five test sites within the Alaska segment of the proposed route.

**TRENCH STABILITY/BLASTING TESTS AT PRUDHOE BAY AND SLOPE MOUNTAIN, ALASKA-1981-ANNGTS**

In an attempt to obtain a better understanding of trench stability and blasting effects in complex permafrost terrain, two test sites were established by the project team at the above locations. The effort was focused on trench excavation and working through highly variable frozen ground conditions, including massive ice and ice wedge polygons. The study was performed under the direction of the Engineering Section of ANNGST with the assistance of their consultant support group.

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Alyeska’s 40-Plus Years of Experience with Heat Pipes on the Trans-Alaska Pipeline System

Larry Mosley¹; John Zarling, Ph.D., P.E.²; Frank Wuttig, P.E.³; and Charles Schulz, P.E.⁴

¹Alyeska Pipeline Service Company, Fairbanks, AK, USA. E-mail: Larry.Mosley@alyeska-pipeline.com
²Zarling Aero and Engineering, Fairbanks, AK, USA. E-mail: zae@gci.net
³Alyeska Pipeline Service Company, Fairbanks, AK, USA. E-mail: Frank.Wuttig@alyeska-pipeline.com
⁴Alyeska Pipeline Service Company, Fairbanks, AK, USA. E-mail: Charles.Schulz@alyeska-pipeline.com

ABSTRACT

Heat pipes (thermosyphons) were installed in the vertical support members (VSMs) of the Trans-Alaska Pipeline System where the pipeline is elevated in warm non-thaw-stable permafrost areas. More than 124,000 heat pipes with pure anhydrous ammonia, NH₃, as the working fluid were installed during pipeline construction in the mid-1970s to thermally stabilize the permafrost surrounding the VSMs. Shortly after pipeline construction, non-condensable-gas, NCG, began to occur in some of the heat pipes, affecting their performance. Alyeska conducted an extensive research effort to identify the root cause for the occurrence of NCG and performed a test program on the degradation in heat pipe performance with the build-up of NCG. Two procedures have been used to repair underperforming heat pipes due to NCG issues: (1) Using “getter devices” and (2) recharging the heat pipes with carbon dioxide, CO₂. Using getter devices was not a permanent solution to the NCG problem, and experience has shown CO₂ recharging to be a successful repair option. This paper describes 40-plus years of heat pipe experience for an aboveground pipeline system in permafrost, several heat pipe options Alyeska considered prior to construction, and the choice and development of the heat pipes that were used on the pipeline. This paper also describes how Alyeska has managed NCG issues that developed after construction to ensure integrity of the aboveground system and the innovative use of heat pipes as thermometers to monitor end-of-thaw-season ground temperatures at the base of thermal VSMs as an additional integrity management tool.

Keywords: Thermosyphon Performance, Non-Condensable-Gas in Thermosyphons, Trans Alaska Pipeline System

HEAT PIPES AND THERMOSYPHONS

Heat pipes consist of a sealed tube or pipe charged with a pure working fluid, i.e., a liquid with no contaminants or with a fixed chemical composition throughout. The charge of working fluid is such that there is pool of liquid inside the heat pipe with the remainder of the pipe’s volume filled with vapor. The heated end of the heat pipe is typically referred to as the evaporator section and the opposite cooled end is called the condenser section. Heat pipes have a very high thermal conductance due to their operating characteristic of continuous vaporization and condensation of the working fluid. Technically speaking, heat pipes also have an internal capillary screen or mesh that returns the condensate to the evaporator section. If the capillary mesh is not present and the condensate is returned by gravity, then the heat pipe is referred to as a thermosyphon. The “heat pipes” used on the Trans Alaska Pipeline System, TAPS, are thermosyphons. In this paper we will
refer to the thermosyphons used on the TAPS, as “heat pipes”.

HISTORY

Development of the heat pipe started in 1831 by A. M. Perkins with the “Perkins Tube” which operated as a thermosyphon as it did not have a capillary wick (Wikipedia 2020). A General Motors engineer, R. S. Gaugler (Gaugler 1944), patented in 1944 the heat pipe with the inclusion of a capillary wick using any “volatile liquid” as the working fluid. The early 1960’s showed interest in heat pipes for space applications. Los Alamos National Lab was a leader in the development of heat pipes for this application with G. M. Grover (Grover 1966) receiving a patent for the heat pipe in 1966. He used water, sodium, and lithium as working fluids. In the mid 1950’s Erv Long of Anchorage, Alaska and an engineer with the US Army Corp of Engineers independently developed and patented the thermosyphon for permafrost stabilization, (Long 1965a). He initially used propane as a working fluid but subsequently switched to carbon dioxide.

DEVELOPMENT OF HEAT PIPES AND THERMAL VSMS FOR TAPS

North American’s largest oil field was discovered at Prudhoe Bay on Alaska’s North Slope in 1967 and, then with the signing of the Alaska Native Claims Settlement Act in 1971, the way was paved for the design and construction of TAPS. Construction was completed in 1977 with first oil flow occurring in June of that year. The chosen pipeline route from Prudhoe Bay to the ice-free port of Valdez is about 1,290 km (800 miles) in length with 680 km (420 miles) of pipeline built above ground. Early in the design, an H-type bent with two piles supporting a crossbeam on which the pipe was supported was the focus for the above ground pipeline in the non-thaw-stable permafrost areas.

To maintain the permafrost surrounding the piles, a cooling system was required. Pile cooling options considered were: air convection piles (Reed 1966, Reid 1974), two phase thermosyphons installed within the pile, thermosyphon piles and single phase thermosyphons installed in the piles, (Jahns et al. 1973, Waters et al. 1975).

In early 1972, Alyeska Pipeline Service Company, APSC or Alyeska, had discussions with Thermodynamics’ Inc. on their liquid convection pile, (Babb et al. 1971), Erv Long on his thermosyphon based “Long Thermopile” (Long 1965a, b), and Grumman Aerospace and McDonnell Douglas Astronautics Co., (Waters 1974, Waters et al. 1975, Grover 1975, Cady 1976) on the development of small diameter heat pipes (Cryo-anchor) for installation in pipeline support piles known as vertical support members or VSMs. In the end, Alyeska chose the McDonnell Douglas Cryo-anchor as the heat pipes used in the thermal VSMs for the TAPS pipeline. There were over 124,000 heat pipes installed during construction. Much of the development work performed on the heat pipes for the TAPS pipeline is described by (Galate 1975, 1976, Jahns 1973).

In the early 1970s, Alyeska established the Chena Hot Springs Road Test Site northeast of Fairbanks, Alaska for conducting pile testing, (Strandberg 2020). The site was underlain with thermally degrading permafrost and managed by R & M Geological Consultants. Air convection piles and piles with heat pipes were installed and instrumented with data recorded and subsequently analyzed. These data were used for the early validation of Esso Production Research Co.’s two-dimensional, non-steady-state finite element “Permafrost Simulator”, (Wheeler 1973, 1978) which incorporated a surface energy balance (Miller 1975). Heat pipes were installed in three piles in November of 1972. These thermal piles with heat pipes installed showed stabilization

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of the degrading permafrost.

In March of 1974, a prototype work pad was constructed along the present alignment of TAPS at the end of Love Road which intersects with Chena Hot Springs Road, (Strandberg 2020, Pearson 1977). This area consists of tussocks, mosses, sparsely occurring tamaracks and black spruce as surface vegetation and is underlain with permafrost. Later that year, in early October, fourteen VSMs were installed to support an above ground anchor (4 VSMs) and gate valve assembly (4 VSMs); two structural bents, Nos.183 and 184 (4 VSMs); and a dummy bent with VSMs Nos.13 and 14 (2 VSMs). This section of pipeline was incorporated into TAPS during construction in the following years (Cady 1978, Pearson 1977). Prototype heat pipes were installed in VSMs associated with bents Nos. 183 and 184 at the end of November 1974. Production heat pipes were installed in the aboveground anchor and gate valve assembly VSMs in March of 1975. No heat pipes were installed in the VSMs of the dummy bent. Figure 1 shows the VSMs for the dummy bent with temperature-controlled radiators and an infrared target radiator on the far left. These two VSMs were used as a non-thermal VSMs for thermal cooling comparison and in a surveillance experiment and have since been removed.

A weather station and numerous soil temperature thermistor strings were also installed in 1974 at this site. Alyeska’s Master Specification for thermistor strings requires YSI 44034 NTC thermistors or equivalent (5 kOhm at 25 °C, 16.33 kOhm at 0°C with +/- 0.1 °C interchangeability tolerance). Recorded data following the 1974-75 winter, showed end of summer ground temperatures surrounding the instrumented thermal VSMs had decreased about 1.1 °C (2 °F) compared to soil temperatures surrounding the dummy VSMs.

![Figure 1. Two VSMs that were a part of the dummy bent installed in October 1974 at the Love Road test site. Photo taken in 2005, however, these VSMs have since been removed.](image)

**TAPS HEAT PIPES**

TAPS’s heat pipes are installed in VSMs forming the bents supporting the above ground pipeline in areas with warm non-thaw-stable permafrost, Figure 2. VSMs with heat pipes installed
are referred to as thermal VSMs and provide cooling of the ground, (Heuer 1979). These heat pipes, charged with anhydrous ammonia, NH₃, were manufactured by the McDonnell Douglas corporation. TAPS heat pipes are made of mild steel with an inside diameter of 38 mm (1.5 inches). Outside diameter varies from 76.2 mm (three inches) starting at the top of the heat pipe extending downward from 152 mm to 304 mm (six to twelve inches) below the top of the VSM and then 50.8 mm (two inches) outside diameter to the bottom end of the heat pipe. Wall thicknesses are 19 mm (0.75 inches) and 6.4 mm (0.25 inches), respectively. Lengths of TAPS heat pipes vary from 8.5 m (28 feet) to 22.9 m (75 feet) in generally 0.91 m (three-foot) increments. Heat pipe lengths ranging from 8.5 m (28 feet) to 11.3 m (37 feet) received a 1.2 m (4-foot) long aluminum finned section installed on the condenser end and lengths ranging from 12.8 m (42 feet) to 22.9 m (75 feet) have 1.8 m (6-foot) finned sections installed. The 152 mm (6-inch) section extending above the fins was to serve as a reservoir for non-condensable gas, (Heat Pipes--Data Sheet C2, Cady 1978).

![Fig. 2. Thermal Vertical Support Member, VSM, (Heat Pipes--Data Sheet C2, Cady 1978)](image)

The end caps and two sections (50.8 mm or two-inch OD and 76.2 mm or three-inch OD) were friction welded in the manufacturing process. The internal surface of TAPS heat pipes was knurled to increase surface roughness for improved heat transfer. A tapered plug is installed just below the condenser end cap that was apparently part of the manufacturing process to charge the unit with the working fluid. A serial number is stamped on the condenser end cap with two of the digits of
the serial number indicating its length.

The finned sections are an aluminum extrusions that are hydraulically pressed onto the 76.2 mm (three-inch) OD section of the thermosyphon. There are 20 vertical fins with an outside diameter of 277 mm (10.9 inches). A high thermal conductivity grease is used to provide improved thermal contact and ease the installation process because of the interference fit. Exposed surfaces of the finned sections are anodized to provide a high emissivity surface in the infrared portion of the radiation spectrum.

The VSMs are 0.45 m (18-inch) outside diameter steel pipe with a nominal 9.5 mm (3/8-inch) wall thickness. After installation of the VSM, two heat pipes were installed diametrically opposite and within one inch of the internal VSM wall. The evaporator end of the heat pipe is typically at or within 0.91 m (three feet) of the bottom end of the VSM. The internal space between the heat pipes and VSMs were filled with a saturated sand slurry up to the ground surface to provide good thermal transfer between the heat pipes and the surrounding ground. Weep holes were drilled into the side of the VSMs near the ground surface to serve as a drainage path for water entering VSMs.

![Figure 3. Infrared image of NCG blocked and unblocked heat pipes.](image)

**NON-CONDENSABLE GAS ISSUE**

Shortly after construction of TAPS was completed in 1977, infrared surveillance of the heat pipes was initiated. This was conducted during cold spells when the units were working using helicopter-borne infrared video imaging and recording equipment to focus on the finned condenser sections. Flights were carried out during night-time hours to avoid daylight reflection interference from the finned condensers. These videos were later reviewed, and any issues observed by pipeline survey stations were recorded.
Figure 4. Molecular sieve/Pd-Ag diffuser to allow hydrogen to escape without leaking ammonia.

The 1980 infrared survey showed cold tops caused by non-condensable gas, NCG, formation on 6% of the heat pipes. It was reported in 1983, (Johnson 1983), that many of the heat pipes installed on TAPS showed some degree of NCG blockage. It was determined that the non-condensable gas was mainly hydrogen and likely due to corrosion. Corrosion of metals containing iron is referred to as rust and can occur by electrochemical or anaerobic processes. For example, in anaerobic corrosion of iron, the global reaction that produces hydrogen is Fe + 2H₂O → Fe(OH)₂ + H₂. When the heat pipe is active, upward flow of NH₃ vapor to the condenser section carries with it the NCG. The NH₃ condenses leaving behind a bubble of hydrogen blocking the upper end of the condenser section. Figure 3 shows an infrared image of blocked and unblocked heat pipes.

Alyeska contracted with several engineering consulting groups with respect to the cold topping issue including Exxon Production Research and Grumman Aerospace. Possible reasons for the occurrence of hydrogen were dissociation of NH₃, internal corrosion, or external corrosion with hydrogen diffusion through the thermosyphon wall. Samples of the gases inside the cold topped thermosyphons which were subsequently analyzed showed more moles of hydrogen than what would occur through disassociation of NH₃. This left the possibility of internal and/or external corrosion. Laboratory testing of the NH₃ in surplus heat pipes from the Alyeska’s Nordale yard east of Fairbanks did not detect non-condensable gases. So, the present thinking is that the source of the hydrogen is external corrosion and then hydrogen diffusion through the heat pipe wall.

GETTER PINS

In the early 1980s, Alyeska embarked on a program to accommodate the NCG issue. Several approaches to eliminate the NCG were investigated including: installing molecular sieves, Pd-Ag diffusers, to allow the hydrogen to escape without leaking ammonia, Figure 4; installing a valve to allow bleeding-off the hydrogen when the heat pipe is operating (there is no valve on the TAPS’s heat pipes); and installing “getter pins” containing a metal hydride, to absorb the hydrogen, Figures 5 and 6. In 1983, 20 prototype getter pins and 11 diffusers were installed. Four different getter
materials were tested with zirconium di-manganese, Zr-Mn₂, chosen for the production getter pins, which were supplied by Ergenics, Inc. Hydrogen dissolves in Zr-Mn₂ to form a solid solution. These were then installed above the finned section of the NCG blocked heat pipes. Beginning in 1984, 94 getter pins containing 33 grams of Zr-Mn₂ were installed and then from 1985 through 1988, 2,683 20-gram getter pins were installed. It was determined from the infrared surveys that many of the heat pipes with the 20-gram getter pins were showing NCG blockage with the conclusion that the getter devices had become saturated with hydrogen. Larger getter pins were developed containing 165 grams of Zr-Mn₂ and 100 were installed in 1992 and 1993.

![Figure 5. Experimental prototype getter device installed in 1983.](image)

![Figure 6. Production getter pin containing zirconium di-manganese, Zr-Mn₂, a metal hydride, installed to absorb the NCG, hydrogen.](image)

The 165-gram getter pins were 254 mm (10.0 inches) in length and 25.4 mm (1.0-inch) diameter and made of mild cold drawn steel. The last 50 mm (2.00 inches) of the pins were tapered down to 15.9 mm (0.625-inch) diameter at its end. A tapered hole was drilled into the section of heat pipe extending above the finned condenser section. The hole did not quite penetrate through the ¾-inch wall thickness of the heat pipe. The lead tape seal was removed from the tapered end of the pin exposing a semi-porous fiber plug and then the pin was quickly inserted into the hole in the heat pipe. A 10-ton hydraulic jack then pressed the pin through the remaining wall of the heat pipe resulting in an audible “pop” exposing the gas inside the heat pipe to the Zr-Mn₂ contained in the pin. There was a leakage factor associated with the installation of getter pins with about 7% of the getter repaired heat pipes having leaked and requiring the installation of heat pipe inserts based on the 1990 infrared survey. The getter repair program was discontinued in 1994, after installation of about 2,900 getter pins, because of leakage between of seated getter pins and heat pipes and hydrogen saturation of the Zr-Mn₂ in the getter pins.

Replacement heat pipes, known as heat pipe inserts, were supplied during pipeline construction and are identical to the standard TAPS heat pipes except below the 76.2 mm (3.0-inch) OD section, replacement heat pipes are 31.8 mm (1.25 inches) in outside diameter. To install these
replacements, the standard heat pipes were cut-off just below their finned section, the internal friction weld bead at the connection between the 76.2 mm (3.0-inch) and 50.8 mm (2.0-inch) outside diameters section was removed with a reamer, and then the replacement heat pipe was inserted. The 3.2 mm (0.125-inch) annular space was filled with a water-glycol solution.

A few of the heat pipe inserts have “jacked” upward following installation in 1990, Figure 7. Several others have failed due to being crushed by fluid pressure within the annular space caused by freezing of the antifreeze. Both failures were likely caused by using a weak water-glycol antifreeze solution during installation of the replacement heat pipes or dilution of the water-glycol due to precipitation.

Figure 7. Heat pipe insert that “jacked” due to water in the annular space freezing. Heat pipe insert “jacked” until it toppled.

NCG BLOCKAGE AND HEAT PIPE PERFORMANCE

In 1999, APSC decided to measure the degradation in heat pipe performance as a function of NCG blockage. The first experimental effort turned-out to be a false start. However, in 2000 four calorimeters were setup at the Doyon Industrial Facility where Alyeska has offices in Fairbanks, Alaska (Sorensen et al. 2003). These calorimeters consisted of insulated vertical large diameter pipes filled with a water/glycol antifreeze solution. Electric heating elements and a submersible pump were installed in the calorimeter pipe to heat and mix the antifreeze solution. Twenty-eight-foot-long heat pipes with four-foot finned sections were installed into the vertical insulated pipes as shown in Figure 8. Thermistors were installed to measure heat pipe evaporator temperature, well mixed water/glycol antifreeze temperature, and ambient air temperature. Wind speed was measured using an anemometer installed as part of the test apparatus. Power was measured to the heating elements and mixing pump with and without the heat pipes installed. Heat loss from the calorimeters without heat pipes installed for a measured antifreeze-air temperature difference
allowed the determination of the thermal resistance of the calorimeters. Total heat loss from the calorimeters when the heat pipes were installed was corrected for the calorimeter heat loss. The test program was carried out by injecting known amounts of hydrogen into the heat pipes and measuring their conductance degradation. Infrared images were recorded for each level of hydrogen injected to attain heat pipe conductance as a function of NCG blockage. In 2001 experiments were conducted replacing NH₃ with CO₂ as the working fluid. No measurable difference in heat pipe conductances occurred when using CO₂.

The correlation of conductance as a function of blockage was used as a grading procedure for identifying heat pipes in need of repair from the infrared surveys.

![Figure 8. Test apparatus used to measure NCG impact on heat pipe conductance. Heat pipes 28-foot long with 4-foot finned sections installed and instrumented.](image)

**NH₃ AND CO₂ REPAIRS**

In the summer of 2001, Alyeska began repairing heat pipes by recharging 70 of them with NH₃. The next summer 186 NH₃ repairs were performed and 671 heat pipes were repaired by recharging with CO₂. The cost and toxicity of the ammonia and the required steel TOR valve were the reasons for changing to carbon dioxide as the recharge fluid. Either five pound or ten-pound carbon dioxide canisters with known weights of carbon dioxide are discharged into the heat pipe under repair, depending on its length. The main steps in the process are to weld on a thread-o-let above the finned section, hot tap into the heat pipe and bleed-off the contaminated ammonia. At this point, the heat pipe is allowed to vent overnight and the evaporator to return to ground temperature. Next, it is evacuated and purged with dry nitrogen twice before a CO₂ CGA 320 manifold valve as shown in Figure 9 is installed, and a final evacuation is performed prior to recharge with carbon dioxide. Since 2002, over 52,000 heat pipes have been converted to CO₂, about 42 percent of the total heat pipes on TAPS.
HEAT PIPES AS THERMOMETERS

Because the working fluid, a mixture of liquid and vapor, is in the two-phase zone, temperature and pressure are not independent thermodynamic properties. Therefore, if the absolute pressure is measured at the CO₂ valve and this pressure is corrected for the hydrostatic head of CO₂ vapor above the liquid pool, then pressure at the base of the heat pipe is known. The temperature is easily determined based on this pressure from a pressure-temperature table for CO₂ Refrigerant 744—saturated liquid and vapor, (ASHRAE 2017). In 2003 a program was initiated of measuring the pressure on several hundred heat pipes at the end of summer before below freezing temperatures occur. These data have been tracked for the last 17 years in order to monitor permafrost temperatures at the base of these thermal VSMs.

SUMMARY AND CONCLUSIONS

Alyeska has more than 40 years of heat pipe experience operating an aboveground pipeline system in thaw unstable permafrost, providing an example and rich history for the design and management of similar systems in the future. Integrity management of the aboveground system has incorporated the experience and lessons learned from this rich history into current management practices. Our experience has shown:

- heat pipes charged with NH₃, like those used on TAPS, can accumulate non-condensable-gas in the top of the heat pipes, reducing their performance;
- how a large-scale experiment was used to determine the degradation in heat pipe performance as a function of NCG blockage;
- how underperforming heat pipes were repaired by replacing the NH₃ operating fluid with CO₂;
- innovative use of heat pipes as thermometers to monitor end of thaw season ground temperatures at the base of thermal VSMs as an additional integrity management tool.

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Slope Stabilization along a Buried Crude-Oil Pipeline in Ice-Rich Permafrost

Peppi Croft, P.E.¹; Oliver T. Hoopes, P.E.²; Frank J. Wuttig, P.E.³; Charles Schulz, P.E.⁴; and Wendy L. Mathieson, P.E.⁵

¹Shannon & Wilson, Inc., Fairbanks, AK, USA. E-mail: ppc@shanwil.com
²Shannon & Wilson, Inc., Seattle, WA, USA. E-mail: oth@shanwil.com
³Alyeska Pipeline Service Company, Fairbanks, AK, USA. E-mail: Frank.Wuttig@alyeska-pipeline.com
⁴Alyeska Pipeline Service Company, Fairbanks, AK, USA. E-mail: Charles.Schulz@alyeska-pipeline.com
⁵Shannon & Wilson, Inc., Seattle, WA, USA. E-mail: wlm@shanwil.com

ABSTRACT

The Trans-Alaska Pipeline System (TAPS) transports warm oil through a 48-inch diameter pipeline (mainline) 800 miles from Prudhoe Bay to Valdez, in Alaska. The system traverses continuous and discontinuous permafrost terrain and is supported aboveground or belowground, depending on subsurface conditions. The stabilization site is in discontinuous warm permafrost in the Copper River Basin in Alaska’s interior. The mainline at the site is buried in thaw-unstable, ice-rich permafrost and actively refrigerated to allow for animal crossings. Alyeska Pipeline Service Company (APSC) initially observed evidence of ground movement at the study site in 2012, threatening to expose the mainline. APSC developed and implemented stability mitigation measures to protect the mainline. This paper demonstrates how geotechnical hazards due to changing environment and permafrost conditions are being managed along TAPS and evaluates mitigation design by comparing predicted and observed subsurface conditions at the study site 3 years after mitigation.

INTRODUCTION

The Trans Alaska Pipeline System (TAPS) transports warm oil through a 48-inch diameter pipeline (mainline) 800 miles from Prudhoe Bay to Valdez, in Alaska (Figure 1). TAPS traverses three major mountain ranges, several active fault systems, more than 30 major rivers and streams (including the Yukon River), and continuous and discontinuous permafrost terrain. The mainline was constructed in the 1970s in a buried, belowground mode or supported aboveground, depending on subsurface conditions. The mainline was typically constructed aboveground in thaw-unstable permafrost terrain where excessive settlement was predicted and belowground in more stable permafrost, thawed ground, or bedrock. A few sections were constructed belowground despite the presence of thaw-unstable permafrost to accommodate animal migration, highway crossings, or rockfall and avalanche terrain. Alyeska Pipeline Service Company (APSC) insulated and refrigerates these sections to prevent permafrost degradation and excessive settlement.

The site is in a section of the Copper River Basin (Figure 1) where the a 1.8-mile-long section of the mainline is buried in thaw-unstable permafrost to allow for caribou crossings. The mainline in the special bury section is insulated and buried with underlying refrigerated circulating brine piping to maintain the integrity of the underlying thaw-unstable permafrost. Mainline refrigeration (MLR) units cool and circulate the brine. APSC has been monitoring the ground temperature and permafrost integrity along the special bury section using thermistor strings. APSC routinely monitors mainline curvature and depth of cover using inline inspection tools and settlement rods.
In 2012, APSC observed ground cracking and scarps over the mainline adjacent to a pre-existing thaw pond on the west side of the workpad and settlement along the east side of the workpad that had developed since construction (Figure 2). Instrumentation measurements from
2001 through 2016 indicated movement toward the thaw pond on the west side of the pad was related to a combination of permafrost thaw, groundwater seepage, and creep near the active layer/top of permafrost interface. APSC determined that continued slope movement could result in unacceptable risk to the mainline and chose to develop and implement slope stabilization measures to maintain long-term mainline integrity. APSC installed two inclinometer casings in October 2013 and began quarterly to annual visits to monitor and study the site in 2013.

APSC performed a desktop study to evaluate mitigation alternatives beginning in 2016 and used 2-dimensional (2D) and axisymmetric finite element thermal simulations to evaluate mitigation options. The selected mitigation was installed in Fall 2017 and consisted of woodchip surface insulation and an array of free-standing thermosyphons left over from construction to freeze-back the active layer, cool permafrost, and slow the movement. Post-installation slope inclinometer and thermistor string data indicate freeze-back and significant cooling of frozen soils in the treatment area, arresting slope movement adjacent to the mainline.

The paper demonstrates how geotechnical hazards are being managed along TAPS and how APSC is adapting to changing environment and permafrost conditions. This paper evaluates the thermal modeling approach and mitigation design by comparing predicted and observed subsurface conditions at the study site 3 years after mitigation installation.

SETTING

The study site is in the Copper River Basin in the southern interior of Alaska, approximately 170 miles south of Fairbanks and 100 miles north of Valdez. The area is part of the eastern Copper River Lowland which is a relatively smooth plain with little elevation changes. The plain is incised by steep-walled valleys of the Copper River and its tributaries. Most rivers head in glaciers in surrounding mountains and have braided upper courses. Lakes are abundant, and many lakes are encroached by irregular muskeg marshes (Wahrhaftig, 1965).

Glaciolacustrine (glacial lake) deposits, lacustrine deposits, lacustrine and organic deposits, and glacial till are typical surficial deposits. The depth to bedrock in the area is unknown and the basin is generally underlain by permafrost. The top of permafrost is typically within 5 feet of the surface in undisturbed areas and permafrost is generally 100 feet to 200 feet thick (Péwé and Reger, 1983). Vegetation in the area predominantly consists of black spruce, willows, and sphagnum moss. Standing water is common in low-lying areas and thermokarst depressions.

CLIMATE

The study site has a continental climate with long cold winters and short warm summers. Air temperature data from the Gulkana Airport (about 25 miles south of the study site) is available between March 1943 and 2020. For the record period between 1990 and 2019, the mean minimum air temperature in January is -9.8 degrees Fahrenheit (°F) and the mean maximum air temperature in July is 69.4°F. The Freezing and Thawing Indices for this period are 4,368 and 3,163, respectively.

Mean annual air temperatures have been increasing over the last century and climate modelers predict continued warming with shorter winters and longer summers.

INSTRUMENTATION

This paper includes references to measurements from the following instrumentation at the study site:
Three MLR monitoring thermistor strings installed in 2001 along a line perpendicular to the mainline at the northern end of the study site. Thermistor string readings have been taken manually on a quarterly to annual basis since 2001.

Two inclinometers installed along a line perpendicular to the expected movement for this study. Inclinometer casings were installed in 2013 and decommissioned prior to mitigation in 2017.

One inclinometer casing installed just prior to mitigation in 2017 to monitor post mitigation movement.

One thermistor string installed prior to mitigation in 2017 to monitor post mitigation ground temperatures. This thermistor string was installed in the area treated with woodchips and free-standing heat pipes (FSHPs) and is about 5 feet from the nearest FSHP. FSHPs consist of a single thermosyphon installed in a steel casing. Thermosyphons are passive cooling devices charged with a two-phase liquid that remove heat from the subsurface when air temperatures are colder than ground temperatures. The temperature cable is connected to a datalogger for relatively continuous temperature measurements.

Figure 2 shows a site plan with instrumentation locations.

SITE AND SUBSURFACE CONDITIONS

We describe site and subsurface conditions prior to the 2017 mitigation implementation below. The site was relatively flat with the exception of an approximately 10-foot-high slope between the workpad and a thaw pond to the west. A drunken forest with leaning trees was present along the eastern margin of the thaw pond, indicating thermal degradation of underlying permafrost.

Workpad fill at the site was underlain by 2 to 4 feet of sandy silt, with organics in the active layer, overlying frozen, ice-rich, glaciolacustrine silt and lean clay with 10 to 50 percent visible ice by volume and massive ice layers. Figure 3 presents a generalized subsurface profile of the site with selected inclinometer and thermistor string measurements.

Prior to mitigation, the depth to top of permafrost ranged from about 10 to 14 feet, and ground temperature monitoring data indicated the active layer depth was increasing with time. Permafrost outside of the influence of the active refrigeration lines was warm with temperatures ranging from about 31 to 32 °F; and permafrost temperatures closer to the refrigeration lines ranged from 26 °F to 32 °F. The target operating temperature for the brine lines is 17 °F.

Groundwater was perched above the permafrost, approximately 4 to 6 feet below ground surface (bgs). The groundwater level in the settlement area, east of the pipeline was higher than the water level in the larger thaw pond west of the workpad.

Ground movement was evidenced by tension cracks and scarps and was measured in the inclinometers. Tension cracks indicated downward and horizontal ground movement toward the thaw pond to the west. Figures 2 and 3 show approximate locations of tension cracks and scarps, and Figure 4 shows photographs of tension cracks in 2016.

Inclinometer measurements showed up to 5 inches of movement toward the thaw pond west of the mainline in a shear zone 7 to 12 feet bgs between 2013 and 2017. Quarterly measurements indicated seasonal movement rate variations, with higher movement rates in the freezing season and lower rates during the thawing season.
MOVEMENT MECHANISMS

Subsurface conditions at the site are complex. Based on thermistor string and inclinometer data, we interpret the following mechanisms contributed to ground movement at the site:

- Continued thaw settlement around the thaw pond adjacent to the workpad embankment west of the mainline.
- Groundwater seepage from east to west along the top of permafrost.
- Shear failure in thawed soils near the bottom of the active layer.
- Creep in warm permafrost and seasonally frozen soils.
- Active-layer movement due to freeze-thaw processes.

MITIGATION MEASURE EVALUATION

APSC determined that continued slope movement could expose the mainline and result in unacceptable risk to the mainline and chose to develop and implement slope stabilization measures to maintain pipeline integrity. APSC evaluated several mitigation measures and identified a combination of passive cooling by Free Standing Heat Pipes (FSHPs) and surficial insulation (woodchips) as the preferred strategy. The evaluation process included performing a combination of limit equilibrium slope stability analyses, finite element seepage analyses, and finite element thermal simulations. This paper focuses on the finite element thermal analyses.

THERMAL MODELING

We describe the software, approach, limitations, input factors, and analyses in the following sections.
Software and Approach: We performed transient thermal simulations using the finite element simulation packages TEMP/W (Geo-Slope International, 2016b). At the time of our analyses, TEMP/W was designed to solve axisymmetric and 2D, transient, heat-transfer problems with phase change. 3D modeling capabilities were not yet available for TEMP/W. The axisymmetric analyses modeled heat transfer due to conduction. 2D analyses modeled heat transfer due to groundwater convection and conduction by coupling SEEP/W and TEMP/W models.

Limitations: Thermal and seepage models are based on generalized representations of real-world conditions. Models have inherent limitations, including:

- Subsurface soil profiles are simplified representations based on conditions encountered during exploratory drilling at discrete locations.
- 2D thermal models are not designed to include the effect of vertical thermosyphons (FSHPs) and effects of thermosyphons were approximated by assuming horizontal thermosyphons instead.
- The 2D thermal models include effects of groundwater, mainline temperature, and brine-line temperature.
- Axisymmetric analyses model effect of one FSHP without groundwater, mainline temperature, and brine-line temperature.
- The combined cooling of several vertical FSHPs was not evaluated by these analyses.

Input Parameters: We developed soil parameters based on conditions encountered during exploratory drilling, index property soil testing, and published values. We estimated thermal conductivity values of the granular fill and silt after Johansen (Johansen, 1975). We ignored the presence of insulation around the mainline based on studies suggesting insulation properties have degraded significantly since construction (SSD, Inc. and J. A. Maple & Associates, 1998).
Figure 5. Selected 2D Simulation Results.

a) Pre-mitigation conditions simulation.

b) Woodchips and horizontal thermosyphons after 8-year simulation.

c) Woodchips and horizontal thermosyphons after 40-year warming trend simulation.
Boundary Conditions we applied in our models included:
- N-factor adjusted air temperature,
- Brine-line temperature,
- Pipe surface temperature,
- Geothermal heat flux, and
- Thermosyphons.

We based temperature boundary conditions on recorded air temperature, brine-line temperature, and pipe temperature data between 2007 and 2016. We fit an average annual temperature sinusoidal curve to the daily mean air temperature data and used typical published n-factors for different surfaces to estimate surface temperature conditions. We used air temperature data from the Gulkana Airport about 25 miles south of the study site.

Based on available data, we selected a constant brine line temperature of 17.5°F and developed a sinusoidal fit for available pipe temperature data from 25 miles south of the study site. We applied a constant geothermal heat flux of 0.325 BTU/(day ft²) at the bottom of the models. We assumed starting conditions were 20°F at the ground surface and 27°F near the bottom of the thaw pond and workpad fill. We developed design thermosyphons and unit conductance values using axisymmetric analyses (see below).

Model Run-Time: We performed axisymmetric and 2D simulations for the following two cases:
- **Case 1:** 40 years with average air temperatures and pre-mitigation conditions followed by 8 average air temperature years with mitigation measures in place.
- **Case 2:** 40 years with mitigation measures in place with a warming trend of 1°F per decade applied.

Axisymmetric Analyses for Vertical Thermosyphons: We used axisymmetric analyses to evaluate the effect of thermosyphons on soil thermal conditions at the site. We evaluated thermosyphons with various unit conductance (UC) values to identify a preferred thermosyphon unit conductance. The axisymmetric analyses modeled a slice through a vertically installed thermosyphon in a cylindrical coordinate system. The horizontal direction in the model represents the radial distance from the center of the heat pipe.

Based on our parametric axisymmetric analyses, we selected a design thermosyphon UC of 95 BTU/(days F° ft). This UC corresponds to a 4-ft-tall fin (condenser unit) and a 2-inch-diameter heat pipe (evaporator unit) embedded 23 ft below the ground surface. The total conductance capacity of the design thermosyphon is 48 BTU/(hour F°) which corresponds to 1,152 BTU/(day F°).

Two-Dimensional (2D) Analyses: We used 2D analyses to evaluate thermosyphon and insulation options and to show that thermal improvements would create a frozen mound west of the mainline that would increase shear strength and resist movement. Profile A-A’ in Figure 3 shows general subsurface conditions we used in the 2D analyses. The 2D analyses included convective heat transfer effects from groundwater flow between the settlement area and thaw pond east and west of the mainline, respectively.

To approximate the effect of the FSHPs in the 2D analyses, we used three horizontal thermosyphons spaced 8 feet apart in end view near 5 feet below the base of the wood chips. We evaluated buried expanded polystyrene (XPS) board insulation and woodchips for surface insulation. Simulation results indicated the woodchips and XPS board insulation provided similar performance.

Figure 5 presents selected results for 2D thermal simulations near peak thaw in late October:
• Figure 5a presents pre-mitigation conditions with thawed conditions between the ground surface and up to 12 feet bgs, which is consistent with pre-mitigation thermistor string measurements.
• Figures 5b and 5c present conditions 8 and 40 years after mitigation implementation, respectively. Mitigation measures in these analyses included 2.5 feet woodchip insulation at the surface and three rows of horizontal thermosyphons installed about 5 feet below the base of the woodchips west of the mainline.

MITIGATION DESIGN

Based on the thermal, seep, and slope stability analyses, APSC selected the following mitigation design:
• FSHPs with 4-foot-long fins installed in a triangular grid on 10-foot centers and embedded to 25 feet bgs.
• Woodchip insulation (3-foot-thick) at the ground surface. APSC selected woodchips insulation for the final design due to the ease of insulation and maintenance and APSC’s experience with woodchips insulation at other sites along TAPS.

Thermal simulations indicated this design is conservative. APSC selected this conservative design to reduce potential future maintenance and monitoring cost.

Figure 6. Site Photograph of Installed Mitigation Measures (in 2020).
MITIGATION CONSTRUCTION

APSC installed FSHPs and placed woodchips in October and November 2017, respectively. Additionally, APSC placed fill in the settlement area with standing water east of the drivelane to reduce the potential for warm groundwater flow under the workpad and across the buried pipeline. APSC installed 53 FSHPs to about 25 feet bgs. Figure 6 presents a photograph of mitigation measures at the study site in 2020.

OBSERVED AND PREDICTED MITIGATION PERFORMANCE

Post-mitigation construction temperature measurements indicate significant cooling. Figure 7 presents a contour plot of ground temperatures with depth and time, at a location about 5 feet away from the nearest FSHP. The top of the plot corresponds to the bottom of the woodchip insulation.

Figure 7. Temperature Contours with Time (about 5 Feet from Nearest FSHP).

Inclinometer and thermistor string measurements since mitigation installation in 2017 show the following:
- No inclinometer movement since August 2018. About 0.5-inch movement occurred 10 to 15 feet bgs between December 2017 and August 2018. The movement likely occurred in December and January, before the FSHPs had sufficiently cooled the permafrost to mitigate creep movement.
- Active layer thickness decreased from up to 14 feet bgs to 2 feet below the bottom of the woodchips insulation.
- Significant permafrost cooling up to 30 feet bgs.
- Permafrost temperatures near the previous active layer depth decreased from 31 to 25°F and have remained below 28°F since February 2018.

Figure 8 presents predicted (blue) and observed (green) temperatures near 7 and 10 feet bgs about 5 feet from the nearest FHSP. Predicted temperatures are based on axisymmetric analyses.
The generalized soil profile on the left shows depth of different soil units used for the thermal modeling and assumed FSHP embedment depth. Shear zone movement at this location occurred 7 to 10 feet bgs prior to mitigation installation.

![Generalized Soil Profile](image)

Figure 8. Simulated and Observed Subsurface Temperatures Near the Shear Zone (5 Feet Away from the Nearest FSHP).

Predicted winter temperatures near 10 feet bgs were between 26 and 30°F, while observed winter temperatures were 19°F in the first winter and 15°F and 14°F in subsequent winters. Predicted summer temperatures near 10 feet bgs were between 30 and 32°F, while observed summer temperatures remained below 28°F starting in 2018.

DISCUSSION

Post-construction instrumentation data between 2017 and 2020 show mitigation improvements are providing significant cooling and arrested slope movement at the study site. As expected, thermal simulations underpredicted the cooling capacity of the mitigation system as constructed due to the following:

- Axisymmetric FSHP analyses do not consider interaction effects between thermosyphons.
- Potential added cooling due to wind effect on thermosyphon condensers was ignored.
- The average effective spacing between FSHPs in the array is less than assumed in our analyses.
- The thickness of woodchips installed at the site was 3 feet instead of 2.5 feet as modeled.

At the time of the analyses, the thermal modeling software we used was not designed to model vertical thermosyphons in 2- or 3D space. The additive cooling effect of several thermosyphons could thus not be modeled using the software and several conservative assumptions were made. A conservative design approach at the study site was preferred by APSC to reduce the need for future maintenance and monitoring costs at the site.
CONCLUSIONS

The paper demonstrates how geotechnical hazards are being managed along TAPS with changing environment and permafrost conditions. We evaluated the thermal modeling approach and mitigation design by comparing predicted and observed subsurface conditions at the study site 3 years after mitigation installation. The mitigation measures achieved required cooling and arrested movement within several months. Observed subsurface temperature data show significantly more cooling than was predicted by thermal modeling, due to modeling limitations and design changes. APSC plans to continue recording subsurface temperature data at the site to monitor mitigation performance. Observed mitigation performance at this site will be available to guide design and reduce conservatism for future projects.

REFERENCES

Embankment Fill Slope Movement on Thaw Sensitive Permafrost: Movement Mechanisms and Thermal Conditions at Lost Creek along the Trans-Alaska Pipeline System; Lost Creek—Part 1

Peppi Croft, P.E.; Oliver Hoopes, P.E.; Frank Wuttig, P.E.; and Wendy Mathieson, P.E.

1Fairbanks, AK, USA. E-mail: peppi.croft@gmail.com
2Shannon & Wilson, Inc., Seattle, WA, USA. E-mail: oth@shanwil.com
3Alyeska Pipeline Service Company, Fairbanks, AK, USA. E-mail: frank.wuttig@alyeska-pipeline.com
4Shannon & Wilson, Inc., Seattle, WA, USA. E-mail: wlm@shanwil.com

ABSTRACT

The Trans-Alaska Pipeline System was constructed in the 1970s to transport oil through a 48-inch diameter pipeline (mainline) 800 miles from Prudhoe Bay to Valdez, Alaska. The system traverses continuous and discontinuous permafrost and is supported above-ground or buried, depending on permafrost and ground conditions. The Lost Creek site is in discontinuous, warm permafrost on a northwest-facing cut and fill slope. On the lower portion of the slope, fill was placed over frozen colluvium and ice-rich peat. The mainline at this site is supported aboveground on H-type bents, which consists of two piles supporting a crossbeam on which the pipe is supported. Since 1990, Alyeska Pipeline Service Company has observed signs of ground and pile movement. This paper presents our interpretation of geotechnical site conditions based on instrumentation monitoring and field observations. We developed mitigation options to control ground movement and present our approach in a separate paper within these proceedings.

Keywords: slope stability, thaw sensitive permafrost, pipeline

INTRODUCTION

The Trans Alaska Pipeline System (TAPS) transports warm oil through a 48-inch diameter pipeline (mainline) over 800 miles from Prudhoe Bay to Valdez, Alaska. TAPS traverses three major mountain ranges, several active fault systems, more than 30 major rivers and streams (including the Yukon River), and continuous and discontinuous permafrost terrain. The mainline is supported aboveground or buried, depending on permafrost and ground conditions. Thermal or non-thermal, 18-inch-diameter pipe piles, referred to as vertical support members (VSMs), support the aboveground mainline. Thermal VSMs are outfitted with two thermosyphons each, which passively remove heat from the subsurface during the winter months. Support systems were designed based on preconstruction subsurface explorations and terrain mapping. Support designs were adjusted as necessary based on the actual subsurface conditions encountered during construction.

The pipeline support system was designed to allow for longitudinal and lateral pipe movement at intermediate bents. Shoe supports connect to the pipeline and rest on crossbeams that are supported by two VSMs (Figure 1). Shoe supports are lined with Teflon-slide plates and are designed to move on the crossbeams to allows for pipe movement (e.g., due to thermal expansion and contraction of the pipe, hydraulic events, and seismic events).

The Lost Creek site is in warm discontinuous permafrost on a northwest-facing cut and fill slope with permafrost temperatures between 30 and 32 degrees Fahrenheit. The embankment fill is up to 45 feet thick and was placed over ice-rich (thaw unstable) peat and silt near the base of the
slope and frozen colluvium along the lower hillslope. The mainline is supported aboveground by thermal and non-thermal VSMs, which are embedded to 18 to 50 feet below ground surface (bgs).

Figure 1. Typical Intermediate Bent Support.

Alyeska Pipeline Service Company (APSC) actively monitors and maintains TAPS and unstable areas. Starting in 1990, APSC observed ground cracking and VSM movement at the site. In early 2000, APSC first observed and noted VSM downhill movement that required increased maintenance. APSC installed geotechnical instrumentation in 2006 and 2009 and advanced exploratory borings in 2010 to study movement mechanisms. Geotechnical instrumentation included three thermistor strings and three inclinometer casings. Geotechnical data indicates movement occurs in a shear zone in a layer of underlying frozen peat once representing the ground surface. APSC collaborated with University of Alaska Fairbanks researchers in 2017 to sample shallow peat deposits adjacent to the embankment for laboratory creep and thermal testing.

This paper summarizes movement mechanisms and thermal conditions at the Lost Creek site. APSC is planning to replace VSMs and install mitigation measures at this site in 2021. We present our approach for designing mitigation methods in a separate paper within these proceedings (Lost Creek – Part 2).

SITE SETTING

The Lost Creek site is approximately 55 miles northwest of Fairbanks, Alaska and 6 miles northeast of Livengood near Alaska Department of Transportation & Public Facilities Milepost 6 on the Dalton Highway (Figure 2). The site is part of the Yukon-Tanana Upland Physiographic Province which is comprised of a wide band of rounded, northeast-trending ridges, hills, and low mountains between the Tanana and Yukon Rivers. Deformed sedimentary and volcanic rocks underlie the northern portion of the Uplands which rise to elevations of 1,000 to 5,000 feet (Wahrhaftig, 1965).

Surficial deposits at the Lost Creek site consist of alluvial stream terrace deposits near the valley bottom and upland silt and colluvium along the hillside (Waythomas and others, 1984; Twelker and others, 2016). The Lost Creek area has not been glaciated. Permafrost in the Livengood area is discontinuous (Jorgensen and others, 2008) and periglacial mass wasting is
common at higher altitudes while valley bottoms commonly contain ice wedges (Wahrhaftig, 1965).

Figure 2. Trans Alaska Pipeline System and Site Location.

The Lost Creek valley is asymmetrical and consists of gently sloping south-facing slopes and relatively steep north-facing slopes. Vegetation on south-facing slopes consists of tundra and small black spruce trees rooted in ice-rich silt. North-facing slopes are characterized by relatively thin silt deposits over shallow bedrock. Bedrock at the site consists of fine grained, thinly bedded to massive chert.

CLIMATE

The Lost Creek site is in a continental climate zone, characterized by short warm summers and long cold winters. Air temperature data from a NOAA climate station in Livengood is available between 1998 and 2020, with 94 percent data coverage. The mean air temperature is -7.4°F in January and 59.4°F in July and the Freezing and Thawing Index are 5050 and 3178, respectively, for the period of record.

Figure 3 presents mean annual air temperature data with time for Fairbanks between 1970 and 2020 and for Livengood between 1999 and 2020. Mean annual air temperatures for Livengood are 2 to 3 degrees colder than for Fairbanks, however, both locations show a similar increasing trends.

SITE DESCRIPTION

From the Lost Creek valley, the mainline rises up over a thick fill and through a deep cut that was excavated up to about 60 feet below natural grade. Material from the cut was used to construct the fill slope and workpad below the cut. The fill is up to 40 feet thick. The approximate locations of these cut and fill slope areas are indicated with dashed lines in Figure 4. Workpad fill in the cut area is approximately 5 feet thick and appears to consist of spoils from the excavation. The workpad surface through the cut and upper portion of the fill slopes at about 3 Horizontal to 1
Vertical along the pipeline, with locally steeper areas.

![Figure 3. Mean Annual Air Temperature for Livengood and Fairbanks.](image)

**Figure 3. Mean Annual Air Temperature for Livengood and Fairbanks.**

![Figure 4. Site Overview and Instrumentation Cluster Locations (View Southeast).](image)

**Figure 4. Site Overview and Instrumentation Cluster Locations (View Southeast).**

**SUBSURFACE MATERIALS**

APSC recorded subsurface materials in four preconstruction borings, during VSM construction, and in geotechnical borings which were advanced in 2006, 2009, and 2010. Subsurface materials within the fill area generally consist of a granular workpad fill material underlain by a combination of organic silt, peat, massive ice, and a poorly sorted mix of silt, sand, and gravel over weathered bedrock. Embankment fill thickness increases from about 5 feet near the valley bottom to 40 feet near the top of the fill area. We interpret part of the embankment fill overlies a layer of colluvium, perhaps landslide debris, deposited before pipeline construction. The colluvium consists of silt with sand and gravel. The base of this material likely coincides with the failure surface we observe in the inclinometer casings today. The landslide colluvium is underlain by a peat layer that increases in thickness from about 1 foot under the colluvium to 9 feet in the valley bottom.
APSC installed three instrumentation clusters, each consisting of one inclinometer casing and one thermistor string. Figure 4 presents approximate locations of these instrumentation clusters.

- Instrumentation Cluster 1 is near the top of the fill slope in the workpad about midspan between VSM bents and between the pipeline and the drivelane.
- Instrumentation Cluster 2 is near the bottom of the eastern toe of the fill slope, east of the workpad and drivelane. The fill surface slopes toward the northeast at this location.
- Instrumentation Cluster 3 is near the bottom of the fill slope in the workpad about midspan between VSM bents and between the pipeline and the drivelane.

INCLINOMETER DATA

Measurements from inclinometer casings indicate movement parallel to the mainline along a
shear zone in ice-rich peat. Figure 5 shows cumulative displacement versus depth for the inclinometer near the bottom of the fill slope (Instrumentation Cluster 3) and the top of the fill slope (Instrumentation Cluster 1).

Figure 6 presents resultant displacements in the ice-rich peat for the three inclinometers. The shear zone movement rate does not show significant seasonal variations. Resultant movement in Cluster 2 is higher than in Clusters 1 and 3, likely due to the addition of lateral movement (transverse to the embankment) which occurs near the same depth at this location.

SUBSURFACE TEMPERATURE DATA

APSC installed two thermistor strings in 2006 and one thermistor string in 2009 and recorded manual measurements on an annual to quarterly basis since installation. Thermistor string measurements show warm permafrost underlies the site. The depth to permafrost varies longitudinally and laterally across the slope. The depth to top of permafrost generally increases with increasing fill thickness. Measured permafrost temperatures range from 30 to 32 degrees Fahrenheit (°F). Figure 7 presents thermistor string data from near the bottom (Cluster 3) and top of the embankment fill (Cluster 1). Temperatures in the upper portion of the permafrost at the bottom of the fill slope show an increasing trend since 2006 (Figure 8). The thermistor strings near the bottom of the embankment slope are within an area of embankment shoulder rotation described later.

Figure 7. Thermistor Data from Bottom (Left) and Top (Right) of Embankment Fill Slope.

VSM Tilt Data. We measured VSM tilt annually beginning in 2006 using a 2-foot digital level. Figure 9 shows relative VSM tilt magnitude and direction of VSMs in the embankment fill in 2020. Figure 10 presents VSM resultant tilt magnitudes with time by relative embedment and movement mechanism. VSM tilt magnitudes have increased up to 3.5 degrees since 2006.
SUBSURFACE CONDITIONS

We used information from the subsurface explorations, original construction records, and our
measurements to interpret a geologic profile of the slope. Figure 11 shows fill slope subsurface conditions along the mainline. The interpreted shear zone (dashed blue line) is based on inclinometer measurements and tension cracks near the top of the fill slope. We note that VSMs are embedded below or above the shear zone, which affects VSM tilt magnitude and shoe displacement.

![Figure 11. Sketch of Lost Creek Subsurface Geologic Profile – Fill Slope.](image)

**MOVEMENT MECHANISMS**

Data from geotechnical instrumentation, VSM tilt measurements, and our field observations indicate two movement mechanisms are acting on the embankment fill slope:

1. **Longitudinal (along the pipeline) movement due to creep along a shear zone in ice-rich peat in warm permafrost.** This movement is indicated by inclinometer measurements, transverse ground cracking across the top of the fill slope, VSM tilt of VSMs embedded below the shear zone, and relative shoe movement on the upper portion of the slope. The tips of some VSMs are founded in stable soil below the lower shear zone, while the upper portions of these VSMs are being pushed over by the moving soil mass causing rotational movement of these VSMs. The VSM tips of two bents (41 and 42) are embedded above the shear zone and are affected mostly by translational movement rather than rotational movement.
2. Transverse (lateral) slope movement in the embankment fill side slopes due to degrading permafrost and thaw settlement at the embankment toe and beneath the side slopes. The ground cracking and scarp formations in the workpad embankment are characteristic of lateral spreading and shoulder rotation associated with embankment construction on warm, thaw-unstable permafrost soil.

CONCLUSIONS

APSC identified the Lost Creek site as a maintenance concern with a slope stability hazard that could affect pipeline integrity if the instability is not mitigated. As part of their integrity program, APSC has been studying subsurface conditions, and movement mechanisms to develop risk mitigation strategies. Concerns for this site include:

- Embankment distress in the form of longitudinal ground cracking and scarps that continue to worsen with time.
- Both transverse and longitudinal slope movement mechanisms that are temperature- and climate-sensitive. Long term warming trends in air and ground temperatures increase the risk of accelerated frozen ground creep rates and permafrost thawing, further aggravating movement and increasing risk.
- Seismic activity that could reduce slope stability of thawed soils.
- Longitudinal slope movement due to permafrost creep inducing lateral loads on the VSMs which were not considered in VSM design.

To address these concerns, APSC is planning to replace tilting VSMs and mitigate the slope stability hazard at the site. We present planned mitigation strategies in a separate paper in these proceedings (Lost Creek – Part 2).

REFERENCES


Embankment Fill Slope Movement on Thaw Sensitive Permafrost: Combining Creep Testing and Thermal Simulations to Develop Mitigation Options at Lost Creek along the Trans-Alaska Pipeline System; Lost Creek—Part 2

Oliver Hoopes, P.E. 1; Peppi Croft, P.E. 2; Frank Wuttig, P.E. 3; Chuck Schulz 4; and Wendy Mathieson, P.E. 5

1Shannon & Wilson, Inc., Seattle, WA, USA. E-mail: oth@shanwil.com
2Fairbanks, AK, USA. E-mail: peppi.croft@gmail.com
3Alyeska Pipeline Service Company, Fairbanks, AK, USA. E-mail: Frank.Wuttig@alyeska-pipeline.com
4Alyeska Pipeline Service Company, Fairbanks, AK, USA. E-mail: Charles.Schulz@alyeska-pipeline.com
5Shannon & Wilson, Inc., Seattle, WA, USA. E-mail: wlm@shanwil.com

ABSTRACT

The Trans-Alaska Pipeline System was constructed in the 1970s to transport warm oil through a 48-inch-diameter pipeline (mainline) 800 miles from Prudhoe Bay to Valdez, Alaska. The system traverses continuous and discontinuous permafrost terrain and is supported aboveground or buried, depending on subsurface conditions. The Lost Creek site is a northwest-facing cut and fill slope in discontinuous, warm permafrost. The mainline is supported aboveground by vertical support members (VSMs). Since 1990, Alyeska Pipeline Service Company has observed signs of ground and VSM movement. Movement mechanisms at the site include longitudinal creep movement along a shear zone in ice-rich permafrost and transverse shoulder rotation due to thaw settlement. The movement causes lateral pile loading, VSM displacement, and embankment shoulder failure. This paper presents the methods we used to develop options to mitigate slope movement at the site. We present our interpretation of site conditions in a separate paper within these proceedings.

Keywords: Slope Movement Mitigation, Permafrost Degradation, Creep Movement, Passive Refrigeration, Woodchip Insulation

INTRODUCTION

The Trans Alaska Pipeline System (TAPS) transports warm oil through a 48-inch-diameter pipeline (mainline) 800 miles from Prudhoe Bay to Valdez, Alaska. TAPS traverses three major mountain ranges, several active fault systems, more than 30 major rivers and streams (including the Yukon River), and continuous and discontinuous permafrost terrain. The mainline is supported aboveground or buried, depending on subsurface conditions. Aboveground, the mainline is supported by an H-type bent with two piles supporting a crossbeam on which the pipe is supported. The vertical supports are either thermal or non-thermal 18-inch-diameter pipe piles referred to as vertical support members (VSMs). Thermal VSMs are outfitted with two thermosyphons each, which passively remove heat from the subsurface during the winter months. Thermosyphons consist of 2-inch-diameter, pressurized steel tubing filled with a two-phase fluid that passively extract heat from the subsurface when air temperatures are lower than subsurface temperatures.

The Lost Creek site is a combined cut and fill slope about 55 miles northwest of Fairbanks, Alaska near Milepost 6 on the Dalton Highway. The pipeline grade along the slope was achieved by excavating an approximately 60-foot-deep cut in the upper portion of the slope and using a portion of the excavated material to construct an up to 40-foot-thick fill embankment along the
lower portion of the slope (see Figure 1). The mainline at the site is supported aboveground by thermal and non-thermal VSMs, which are embedded to 18 to 50 feet below ground surface (bgs).

The fill slope has a long history of movement due to degradation of permafrost underlying the embankment shoulders and creep movement in frozen peat. Slope movement causes VSM loading, tilt/rotation, and downhill displacement, and extensive embankment cracking and displacement. The support structures were not designed to resist significant slope movement and repeated maintenance is required at the site. Typical maintenance includes adjusting and re-leveling crossbeams and support shoes, moving support shoes, adjusting the pipeline load on the crossbeams, and re-grading the drivelane and the embankment slopes. APSC expects continued degradation of the slope and support systems as the underlying permafrost continues to warm, thaw, and creep. APSC is concerned the integrity of the support system at this site is not sustainable over the future life of TAPS and plans to replace select VSMs and construct mitigation measures in the future.

We combined laboratory creep testing results, finite element thermal simulations, and slope stability design analyses to establish relationships between temperature, shear load, and creep rate to design a thermal mitigation system consisting of passive, free-standing heat pipe (FSHP) thermosyphons, thermal VSMs, and surface insulation. This paper summarizes our analyses methodology and results.

**THERMOSYPHONS AND FREE STANDING HEAT PIPES (FSHPS)**

A typical FSHP used by APSC consists of a thermosyphon installed in a steel casing installed in a drilled borehole. The annulus between the thermosyphon and steel casing is typically filled with sand slurry. The thermosyphon consists of a sealed pipe with an evaporator section (buried end) and a condenser section (typically fitted fins exposed to air). Thermosyphons are charged with a working fluid with a low boiling point (such as anhydrous ammonia or carbon dioxide). Figure 2 presents a schematic diagram of a vertical FSHP thermosyphon from Wagner (2014). Passive cooling occurs during the winter when the temperature at the condenser (air) is below the saturation temperature of the working fluid and the temperature along the evaporator (subsurface). Heat is absorbed from the soil and rejected to the atmosphere by convection. This passive cooling
cycle occurs throughout the winter months, creating “bulbs” of cooled ground which can grow over time.

![Figure 2. Passive Thermosyphon (from Wagner, 2014).](image)

SUBSURFACE CONDITIONS

The Lost Creek – Part 1 paper in these conference proceedings presents a detailed description of the subsurface conditions and our geotechnical interpretation for this site. We provide a brief overview of subsurface conditions below.

APSC performed subsurface explorations and installed instrumentation at the site in 2006, 2009, and 2010. In 2017, the University of Alaska, Fairbanks (UAF) researchers advanced four shallow borings using a handheld Snow, Ice and Permafrost Research Establishment (SIPRE) coring tool in the low-lying area south of Bents 43 to 42 to collect undisturbed frozen ground samples for laboratory creep testing. Figure 3 shows the approximate locations of the subsurface explorations performed at the site and bent locations.

The embankment fill material consists of sand and gravel with some silt. Embankment fill thickness increases from about 5 feet near the valley bottom to 40 feet near the top of the fill area. Part of the embankment fill overlies a layer interpreted to be landslide colluvium deposited before pipeline construction. A frozen peat layer underlies the landslide colluvium and embankment fill and increases from about 1 foot near the top of the fill area to up to 9 feet near the valley bottom.

The active instrumentation installed at the site, indicated in Figures 1 and 3, includes the following three clusters of inclinometers and thermistor strings:

- **Upper Cluster** (Inclinometer 06S-392-2; Thermistor string 35240) is in the upper portion of the fill area approximately in-line with the VSMs and midspan between bents.
- **East Cluster** (Inclinometer 09S-392-2; Thermistor string 35350) is in the lower portion of the fill area, at the northern (pipeline east) margin of the workpad and adjacent to the drivelane.
- **Lower Cluster** (Inclinometer 09S-392-1; Thermistor string 35230) is in the lower portion of the fill area, approximately in-line with the VSMs and midspan between bents.
Inclinometer measurements indicate downslope creep movement along the frozen peat layer. Thermistor measurements show the temperature of the peat shear zone ranges from 31.3 to 32.0 degrees Fahrenheit (°F). The inclinometers in the upper and lower instrument clusters show a distinct zone with longitudinal (along the pipeline) creep movement while movement in the east cluster is complicated by added lateral shoulder rotation and thaw settlement at similar depths as the creep movement. We consider inclinometer measurements from the upper and lower instrumentation clusters to be representative of the creep zone and used these two locations for the creep strain design analyses in the following sections.

**APPRAOCH**

The VSM support structures are not designed to resist lateral loads induced by slope movement. To reduce potential loads on replacement VSMs, APSC required that the mitigation measures limit VSM tilt to less than 3 degrees (°) over 40 years. We used the following approach to design the mitigation measures:

- Develop allowable shear strain rates that would achieve the VSM tilt design criterion.
- Establish a relationship between temperature, creep strain rate, and shear stress.
- Establish current in situ shear stresses within the shear zone.
- Determine the required level of cooling to reduce the creep rate in the slope to below the allowable strain rates.

We describe these steps in more detail in the following sections.
ALLOWABLE SHEAR STRAIN RATES

We developed the allowable shear strain rate using the following steps:
1. Obtain displacement rate \( (D) \) over the thicknesses of the shear zone at the lower (09S-392-1) and upper (06S-392-2) inclinometers (see Figure 4).
2. Calculate approximate plane strain shear strain rates \( (\dot{\varepsilon}_p) \) by dividing \( D \) by the thicknesses of the shear zones \( (P) \) for the lower and upper inclinometers, respectively \( \dot{\varepsilon}_o = D / P \).
3. Compare cumulative inclinometer shear zone displacements with VSM tilt records for nearby VSMs and develop slope movement to tilt conversion factors \( (S) \) in units of inch per degree) for each inclinometer.
4. Calculate allowable 40-year displacement ranges \( (d_a = S \times 3^\circ) \).
5. Obtain allowable shear strain rates \( \dot{\varepsilon}_a = \frac{d_a}{P \times 40 \text{ years}} \).
6. Convert \( \dot{\varepsilon}_o \) and \( \dot{\varepsilon}_a \) rates from plane strain to octahedral space by multiplying by a factor of \( \sqrt{2/3} \).

Figure 4. Observed Inclinometer Displacement Since September 2006.

CREEP STRAIN RATE – TEMPERATURE - SHEAR STRESS RELATIONSHIPS

The creep rate of frozen soil subjected to long term shear stress is strongly dependent on temperature. We developed creep strain, temperature, shear stress relationships for this site. We describe our methodology below.

Dr. Matthew Bray of UAF performed a series of transient creep tests (Darrow and Bray, 2018) on peat samples from the SIPRE borings adjacent to the embankment fill. Bray performed tests at different fixed temperatures and provided curves showing shear strain rate versus shear stress. Figure 5 presents results for seven of the tests on peat similar to that encountered in the shear zones. We understand Dr. Bray will publish the details of this laboratory test program in a future paper.
We applied loglinear fit lines (straight dashed lines in Figure 5) to extrapolate the lab data trends. Observed and allowable shear strain rates for the study site were slower than the laboratory tests were capable of measuring. The fit lines follow the form of a power function:

\[ \dot{\varepsilon} = A \sigma^n \]

where: \( \dot{\varepsilon} \) = Shear Strain Rate (1/hr), \( A \) = Empirical fit coefficient (equivalent to a viscosity), \( \sigma \) = Shear Stress (psi), and \( n \) = empirical fit coefficient.

Figure 6 presents laboratory creep test data centered on the observed shear strain rates (\( \dot{\varepsilon}_o \); shown as dashed horizontal lines) and allowable shear strain rate (\( \dot{\varepsilon}_a \)) ranges (horizontal yellow and magenta bands) for the study site.

Bray performed transient creep tests at fixed temperatures of 23°F, 27°F, 29°F, or 31°F (Darrow and Bray, 2018). We estimated creep – temperature relationships at intermediate temperatures by approximating linear relationships between the power function fit coefficients (\( A \) and \( n \)) and temperature by least squares regression (see Figure 7). Apart from one data point
(Sample S17 at 23°F), the $A$ and $n$ data exhibited trends with good matches to mathematical functions. We chose to exclude the Sample S17 data point because 1) we wanted to provide more accurate fits at higher temperatures for which shear strain rates are exponentially higher, and 2) we did not anticipate the need to cool the shear zone to temperatures below 27°F.

We plotted octahedral shear stress against shear strain rate and included temperatures observed in the shear zone (Figure 8). Based on the observed temperatures (31.5 and 31.3°F) and current average octahedral shear strain rates the average pre-mitigation in situ shear stress in the shear zone near the top and bottom of the embankment fill slope are 1.45 and 2.00, respectively.

We used the estimated in situ shear stresses depicted in Figure 7 in our subsequent shear strain rate and displacement analyses to design the proposed slope mitigation improvements. For these analyses, we assumed the shear stresses in the slope would not significantly change by cooling the ground.

**THERMAL SIMULATION METHODOLOGY**

This paper focuses on thermal simulations to evaluate FSHP layout. Our thermal simulation methodology consisted of:

1. One-dimensional (1D) modeling to calibrate surface n-factors for the workpad fill based on thermistor string measurements.

![Figure 7. Power Function Fit Parameters and Temperature Based on Creep Tests by Bray.](image)
2. Axisymmetric analyses of a single FSHP considering the full soil column (200-foot-deep bottom of model).
3. Plan view analyses for several FSHP layouts for the peat layer.
4. FSHP layout design analyses using the calibrated plan view analyses.

We performed transient thermal simulations using the finite element simulation package TEMP/W (Geo-Slope International, 2018). TEMP/W solves transient heat-transfer problems with phase change. We used the simulations to estimate ground temperatures over time for the existing conditions and for the proposed mitigation design alternatives assuming a 40-year design life.

![Figure 8. Estimated Current Shear Strain Rate and In Situ Shear Stresses.](image)

**MATERIAL PARAMETERS**

We used the following materials in our thermal models:
- Mineral soils (including workpad material, sand, and silt), chert bedrock, and massive ice;
- Peat;
- Woodchip insulation;
- Steel and sand slurry within the FSHPs.

We developed mineral soil parameters based on boring logs, index property laboratory testing, and published values. We used the simplified thermal and full thermal material models available through GeoStudio’s TEMP/W software. For the simplified thermal model, the frozen and unfrozen volumetric heat capacities, thermal conductivities, and volumetric water contents are fixed. For the full thermal model, thermal conductivity and unfrozen water content are temperature-dependent functions while heat capacity and volumetric water content remain fixed. We used the simplified thermal model for massive ice and the chert bedrock at the base of the stratigraphic column; we used the full thermal model for all other materials.

We estimated thermal conductivity values of the mineral soils (silt, sand, gravel) after Johansen (Johansen, 1975). We used published thermal parameters for chert bedrock from Cermak and Rybach (1982).

We developed thermal parameters for the Lost Creek peat from relationships published in Andersland and Ladanyi (2004) and from UAF laboratory test results (Darrow and Bray, 2018). We assumed a specific heat capacity of 0.4 for the peat.

We developed thermal parameters for the wood chip insulation using a combination of...
laboratory test results and published values. We conducted gravimetric moisture content analyses on one sample from another mitigation area along TAPS near Glennallen, and on six samples from Northland Wood in Fairbanks. These tests resulted in an average of about 100% gravimetric water content. We performed two dry density tests on Northland Wood samples and obtained an average of about 10 pounds per cubic foot (pcf). Using a gravimetric water content of 100% and a unit weight of 10 pcf, we derived the unfrozen and frozen thermal conductivities from relationships from McRoberts and others (1985). We calculated unfrozen and frozen volumetric heat capacity based on Lunardini (1981). We assigned the same unfrozen moisture content curve to woodchips that we assigned to peat.

BOUNDARY CONDITIONS

We developed the following boundary conditions for thermal modeling:

- Ground surface:
  - N-factor adjusted air temperatures based on historic climate data (1970 to 2017),
  - N-factor adjusted air temperature based on climate predictions (2017 to 2060),
- Geothermal heat flux,
- Thermosyphon using historic and predicted climate data for FSHPs.

We used historic climate data from an APSC monitoring station at Lost Creek established in 2009 and the National Oceanic and Atmospheric Administration (NOAA) Livengood station (USC00505534, NOAA, 2018).

We developed synthetic climate predictions using Scenarios Network for Alaska Planning (SNAP) data for the Livengood area at 10-minute resolution and for the RCP 6.5 scenario published by UAF. On average, this record constituted a warming trend with an increase in average annual air temperature of about 0.75°F every 10 years.

We ran 1D simulations from 1970 to 2017 with various \( n_f \) (freezing) and \( n_t \) (thawing) combinations and compared modeling results with observed subsurface temperature data from the Lost Creek site by computing mean square error (MSE) values for temperature predictions between 20 and 40 feet bgs (encompassing the area of interest around the shear zone). We selected the n-factor combination \((n_f = 0.6, n_t = 1.1)\) with the lowest MSE and assumed these n-factors were representative of workpad gravel and woodchip surface. We assumed n-factors of \( n_f = 0.25, n_t = 0.73 \) based on typical values published by Andersland and Ladanyi (2004) for tundra surface adjacent to the workpad.

We used subsurface temperature measurements to estimate the geothermal gradient at the site (about 0.014°F per foot) and calculated the geothermal heat flux based on thermal conductivity of the chert material. We set the boundary condition at the base of the 1D, axisymmetric, and 2D section models to a constant geothermal heat flux (0.315 British thermal unit per day per square foot) for both the initial condition and the consequent transient analyses.

THERMAL SIMULATIONS

We performed thermal simulations to evaluate FSHP layout and spacing using a combination of axisymmetric and plan view analyses. We established a steady transient condition by modeling historic temperatures from 1970 to 2017. We then simulated 2017 and 2060 using a series of synthetic sinusoidal temperature records incorporating a climate warming trend. The vertical sides of the model are no-heat-flow boundaries.
A single FSHP can be adequately modeled using axisymmetric analyses, however, axisymmetric analyses are not feasible to model additive cooling due to multiple FSHPs. Therefore, we modeled the effects of multiple FSHPs at the shear plane depth using plan view analyses. Plan view analyses on the other hand do not account for surface climate boundary and geothermal gradient effects.

To compensate for these limitations, we calibrated the plan view analyses by comparing the following analyses:
- Axisymmetric: 200-foot-deep axisymmetric analysis of a single FSHP with 3-foot-thick layer of woodchips at the surface, extending at least 30 feet beyond the FSHP axis.
- Plan view: quarter-section of equivalent peat thickness, external constant temperature
(31.9°F) boundary conditions, and internal zero heat flux/line-of-symmetry boundary conditions.

Limiting the lateral extents of the plan view model domain to 20 feet from the edge of the FSHP and reducing the unit conductance of the FSHP by 40% produced good agreement between the plan view and axisymmetric models.

Figure 11. Annual High Temperature Results for Plan View Analysis (along line 7.5 feet from FSHPs).

We developed equivalent layout patterns with 10-, 12-, and 15-foot (center-to-center patterns) with two and four longitudinal FSHP rows. Figure 9 presents two examples of plan view geometries. In these diagrams, the red line represents a fixed temperature boundary condition of 31.9°F set at 20 feet from the leftmost FSHP axis (blue half circles). Other edges in the model represent no-flow/line-of-symmetry boundary conditions. The upper diagram in Figure 9 represents two rows of FSHPs in a square pattern (one row on each side of the pipeline) with a FSHP spacing of 15 feet; the lower diagram represents four rows of FSHPs parallel to the pipeline.
with 15-foot triangular pattern spacing, two rows on each side of the pipeline.

**CREEP STRAIN ANALYSIS/DESIGN CALCULATIONS**

After performing these plan view layout analyses, we queried the finite element mesh temperature versus time results along lines transverse to the pipeline at the halfway point between the transverse FSHP rows. For example, in the upper diagram of Figure 9, we queried the results along the top edge of the model domain (Y coordinate = 7.5 feet). For the lower diagram in Figure 8, we queried a line along the mid height of the model domain (Y coordinate = 3.25 feet). These queries resulted in 40-year stacks of simulated temperature records along a line transverse from the pipeline centerline at X coordinate = 0 feet.

We used the power function relationships and estimated in situ shear stresses presented in Figures 6 and 7, respectively, to calculate shear strain rates for the stacks of simulated temperature records. We then integrated the shear strain rate versus time results and multiplied them by the peat thickness to obtain incremental shear displacements at each timestep. Finally, we summed these incremental displacements to obtain cumulative 40-year displacement profiles and plotted them with the allowable displacements ($d_a$). Figure 10 presents a subset of these results for the lower portion of the slope (inclinometer 09S-392-1) for square patterns.

Figure 11 presents temperature results for the four-row, 12-foot-square pattern configuration along a line midway between the transverse FSHP rows. This plot shows the highest temperature timesteps for each year within the simulation. In other words, they do not show colder temperatures that would occur in the freezing season.

**CONCLUSIONS AND DISCUSSION**

Based on our thermal simulations and creep strain analyses, we developed a mitigation design configuration consisting of four rows of FSHPs in a square pattern at 12-foot spacing with a 3-foot-thick surface layer of woodchips insulation. This layout results in a grid of 100 FSHPs.

The analyses results suggest the proposed thermal mitigation system would cool the peat shear zone within 25 feet of the pipeline to about 24°F or cooler within 5 years of installation (see Figure 11). For the estimated in situ shear stresses within the peat shear zone, these temperatures should result in creep rates significantly lower than the allowable ranges. As suggested by the progression of annual high temperature profiles in Figure 11, the analyses suggest the thermal mitigation system will take 10 to 15 years to reach the lowest projected temperatures. Temperature after 15 years slowly increase due to the SNAP climate warming trends. Estimated displacements occur mostly within the first 5 years of mitigation construction.

**CLOSING COMMENTS**

Our thermal simulations and creep calculations are based on assumptions and confined by several limitations. We close with the following comments:

- Since we performed our analyses, three-dimensional (3D) modeling software, TEMP/W 3D, has become available. Use of a full 3D model may produce more accurate results and allow for a more economical design.

- The decision to ignore the Sample S17 test data at 23°F in the power function fit parameter regressions (see Figure 6) resulted in higher creep strain estimates at low temperatures. However, even with these higher estimates, the incremental strains at low temperatures are negligible and essentially do not contribute to the long-term cumulative displacements.
• A loglinear relationship assumption for the creep behavior may not be true at very low strain rates (below $10^{-6}$ hr$^{-1}$). As indicated in Figure 4, the test results appear reasonably linear in log space but the tests at lower temperatures appear to be slightly concave downward and the tests at higher temperatures appear to be slightly concave upward. This is a subjective observation and may not be borne out in reality. However, if true then this could shift the black and gray lines upward, resulting in higher cumulative displacements. This is one of the reasons why we decided to use a higher margin of safety with four rows of FSHPs instead of two.

• We did not include convective effects due to flowing groundwater.

• The design creep calculations assume the post-mitigation shear stresses along the shear zone remain the same as pre-mitigation. The mitigation measures would introduce a complex stress field with stress concentrations focused on the stiffer, super-cooled columns of frozen soil within the movement mass. We consider this a conservative assumption because the stresses in the relatively warmer zones of the stress field would likely be less and probably compensate for the inaccuracy of the simplified stress calculations.

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