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Stability assessment approach for soil slopes in seasonal cold regions

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ABSTRACT

In Hokkaido Japan, soil slope failures along some of the national highways are reported frequently in recent years. A stability assessment method which can consider the impact of seasonal changes like freeze-thaw action, snowmelt water infiltration etc. is of utmost importance and considered to be an immediate requirement for geotechnical practitioners, in order to properly predict the slope stability. In this study, a slope stability assessment approach based on two-dimensional numerical modelling is recommended which considers the water content changes of the soil induced by the seasonal climatic effects i.e. freeze-thaw action, snowmelt water infiltration etc. Two case studies of slope failures in Hokkaido have been studied using the recommended approach. In order to visualise the climatic parameters of most influence on slope stability, parametric studies have been performed through which many useful results in view of the soil slope stability in seasonal cold regions have been obtained. It is found that the freeze-thaw action has a considerable impact on the soil water content and slope stability. On the other hand, the snowmelt water infiltration has a very significant impact on soil slope stability. The recommended numerical modelling approach is found to be very useful in analysing the soil slope stability in seasonal cold regions.

Author keywords: Slope stability, seasonal cold regions, non-isothermal seepage, freeze-thaw action, snowmelt water (IGC: E06/E09/E13)

1. Introduction

Slope failures occur in seasonal cold regions like Hokkaido during the snow melting season and rainy season. Many incidents of soil slope failure along roadways of Hokkaido are continuously being reported (Nakatsugawa et al. 2015). These slope failures involved a collapse of the man-made embankment filling constructed to support the roadway as well as failures in natural slopes and cut slopes in the vicinity of roads. Most of the failures happened particularly during the snow melting season of Hokkaido between April and May. There is an urgent need of studying the causes of soil slope failures induced in seasonal cold regions in order to prevent such disasters.

An enormous number of studies have been done to identify the factors that affect the soil slope stability in cold regions (McRoberts and Morgenstern 1974a, 1974b; Mackay, 1981; Goodrich, 1982; Burgress, 1993; Niu, 2005). McRoberts and Morgenstern (1974b) investigated the failure mechanism of thaw-induced landslide slope failures observed in permafrost regions of Canada. Unlike permafrost regions, seasonal cold regions like Hokkaido experience abrupt weather changes throughout the year, which in turn fluctuates the ground temperature and water content of the soil. The excess water content originating from the snowmelt water and rainfall may induce a slope failure. Since most of the slopes are unsaturated, the key factor that determines the stability of this type of slope failures is the soil water content distribution. Ishikawa et al. (2015) comprehensively summarised the distinction between slope failures in seasonal cold regions and warm temperate regions and pointed out the important factors that need to be considered in the stability assessment of unsaturated soil slopes in seasonal cold regions. To date, an applicable stability assessment approach considering all the seasonal changes of water content, for soil slopes in cold regions has not been adopted in geotechnical practice.

In this study, a stability assessment approach has been recommended which considers the soil water content changes of the slope influenced by seasonal changes. Two case studies of slope failures were analysed using the proposed approach. The first case is a trial physical embankment slope constructed

using volcanic soil studied by Matsumura (2014) and Kawamura et al. (2016). The second case study is a slope failure that occurred in a man-made embankment along a national highway in Hokkaido, Japan (Hokkaido Regional Development Bureau, 2013). Back analyses of both the slope disasters using the proposed approach have been made to investigate the causes and influencing factors of failure. A coupled numerical analysis procedure to simulate the water content of the slope considering the effects of temperature change, latent heat phase change and precipitation including snowfall and rainfall have been adopted. Using the limit equilibrium method, a stability assessment of slope has been made which uses the soil water content distribution derived from the coupled simulation. Further, in order to visualise the parameters of most influence on slope stability, parametric studies have been performed by considering and neglecting the effect of ground freezing, considering and ignoring the effect of snowfall and considering different magnitudes of snowfall and rainfall for both the case examples.

2. Case studies of soil slope failures in Hokkaido

2.1 Case study of failure on embankment slope made up of volcanic soil

An embankment slope was constructed using a volcanic soil at an angle of 45° with 5 m elevation and 7.7 m length at the base. The site is located at Latitude 42°57′13″ North and Longitude 141°21′46″ East in Hokkaido, Japan. As shown in Fig. 1(a) thermometers, tensiometers, moisture content sensors, rainfall gauges and a snow gauge were installed in the slope to monitor the ground temperature, pore water pressure, soil water content, rainfall and snowfall respectively. To avoid disturbance to the surface soil of the slope, monitoring instruments were installed in three different cross sections namely Left (L), Centre (C) and Right (R) as viewed from the bottom of the slope. The monitoring instruments installed in cross sections are shown in Fig. 1 (b) in a two-dimensional view. In order to prevent the mutual water exchange between the physical embankment and ground surface, the embankment slope was covered by impermeable sheets made up of thin waterproof plastic materials at the bottom and side portions. A

geomembrane liner is not used in this case, as it was understood that the water flow through the slope might not reach the slope bottom due to the soil's low permeability, and the purpose of impermeable sheets are to avoid some possible flow of water from the foundation soil. The foundation was also made up of the same volcanic soil material. In this case, no consideration was given to the interface friction that may occur between the impermeable sheets and soil layers. After placing the sheets at the bottom and back side, the soil is compacted layer by layer with a constant depth of 0.25 m per layer up to 5 m height, using a hand-guided roller compactor. A prescribed volume of water (approximately 1 m^3/day) was supplied to the slope using water supply pipes at constant intervals through different cross sections. The amount of the water supplied is determined based on the storage of water supply tanks. The intervals at which water was supplied through Left (L) section are 27-07-2013 to 06-08-2013 (10 to 11 days) and 11-10-2013 to 17-10-2013 (6 to 7 days). The construction and setting up of monitoring instruments were finalised and monitoring starting on November 9, 2012 (09-11-2012). The monitoring continued until the day of slope failure on October 17, 2013 (17-10-2013). Total monitoring time was around 343 days. Further details about the slope monitoring program can be found from Matsumura, (2014) and Kawamura et al. (2016).

2.2 Case study of slope failure occurring along national highway route 230

On April 7, 2013, at 11:20 A.M., a slope failure happened along the national highway route No.230 near 42° 54' 52" North Latitude and 141° 07' 22" East Longitude in Hokkaido, Japan. The national highway connects Sapporo city with Setana, a town in the Hiyama subprefecture of Hokkaido. Hereafter, the slope failure will be referred as highway slope failure in this paper. The slope failure occurred in the embankment along the roadway. The size of the slope failure was around 40 m wide along the road and 19 m in vertical depth along the failure plane. Approximately 11000 m^3 of sediment, containing

95 embankment filling material and accumulated snow above the soil ground together flowed out downward
96 to the slope foot up to 40 to 50 m length, as shown in Fig. 2 (a) and (b).

97 The slope failure was induced by the combined action of heavy rainfall and snowmelt water
98 (Hokkaido Regional Development Bureau, 2013). The cumulative daily rainfall that occurred on the day
99 of slope failure was 92 mm and the cumulative snowmelt was 31 mm, as recorded in a nearby
100 meteorological telemetry, maintained by Ministry of Land, Infrastructure, Transport and Tourism,
101 Hokkaido Regional Development Bureau (MLIT) shown in Fig. 2 (c). The maximum hourly rainfall
102 recorded on 07-04-2013 was 12 mm. The rainfall was continuous from 07-04-2013 00:00 to 07-04-2013
103 11:00. The cumulative daily rainfall along with the cumulative snowmelt water caused the failure of the
104 slope. The cumulative daily rainfall and snowmelt water, which together amounted to 123 mm, is a
105 considerable intensity to make the slope fail.

106 **3. Recommended slope stability assessment approach**

107 In geotechnical design practice, for the long-term stability assessment of soil slopes i.e. embankment
108 slopes and cut slopes along highways, the factors such as freeze-thaw action and snowmelt water
109 infiltration are not considered. As found by previous research, for the design of soil slopes in seasonal
110 cold regions the above-mentioned factors are significant (Ishikawa et al. 2015 and Siva Subramanian et al.
111 2015). Unlike geotechnical problems like frost heave, the instabilities of soil slopes in Hokkaido are
112 driven by the abrupt changes in soil water content distribution (Nakatsugawa et al. 2015). For the analysis
113 of frozen soil behaviour, many complex thermo-hydro-mechanical models (THM) were developed
114 (Selvadurai et al. 1999; Li et al. 2000, 2002, 2008; Nishimura et al. 2009; Ishikawa et al. 2016).
115 Selvadurai et al. (1999) developed a novel computational approach to study the discontinuous frost heave
116 within a frozen soil region. Li et al. (2000) introduced a coupled heat-moisture-mechanical model for
117 frozen soil and demonstrated the applicability for a foundation problem. Later, Nishimura et al. (2009)

118 proposed a fully coupled THM model using finite element (FE) framework and developed a new
119 critical-state elasto-plastic soil constitutive model to consider problems involving water-saturated frozen
120 and unfrozen soils. Recently, Ishikawa et al. (2016) developed a coupled thermo-hydro-mechanical FE
121 analysis method to analyse freeze-thaw of unsaturated soils. The robustness of the THM models discussed
122 above is well proven for the prediction of soil behaviour influenced by frost-heave.

123 Generally, THM models consider the phase mechanics of ice, soil solid and pore water relationships in
124 a sophisticated manner. For slope instability problems in seasonal cold regions, the effects of
125 ground-atmosphere interactions i.e. freeze-thaw action and snowmelt water infiltration are more
126 significant because of the direct impacts of these factors on soil water content distribution. Computational
127 difficulties may arise if the ground-atmosphere interactions are modelled using a THM model. Chen et al.
128 (2013) pointed out the difficulties in performing a fully coupled THM analysis for frozen soil slope
129 stability problems. Simultaneous numerical solutions of coupled unsaturated seepage, thermal and stress
130 relationships including the ground-atmosphere interactions are very hard to achieve and computational
131 methods for this purpose are not common in geotechnical engineering practice. Performing a stress based
132 numerical simulation for a duration of one year including the time-dependent changes in soil water
133 content, changes in temperature and including the atmospheric effects are extremely cumbersome. In
134 addition, the dominant factor that influences the unsaturated soil slope stability in seasonal cold regions is
135 the change in soil water content due to freeze-thaw action and snowmelt water infiltration (Ishikawa et al.
136 2015; Kawamura and Miura, 2013). Since the soil water content distribution inside the slope is the major
137 factor determining the stability, it should be properly estimated for a precise stability assessment. For this
138 purpose, a coupled thermo-hydro (TH) analysis would be appropriate, followed by a pseudo coupled
139 mechanical analysis.

140 In view of these contexts, an approach to simulate the soil water content distribution subjected to
141 freeze-thaw action and snowmelt water infiltration is recommended in this study. The approach is based

on two-dimensional plane strain numerical modelling considering non-isothermal seepage simulation followed by a slope stability assessment as explained in the flow chart shown in Fig. 3. There are three parts i.e. initial analysis, soil water content simulation and slope stability analysis. The first part is the initial analysis to configure the initial equilibrium of the soil slope in terms of soil water content distribution and temperature. The second part is the method used to estimate the water content distribution of the soil slope subjected to freeze-thaw action and snowmelt water infiltration. Non-isothermal seepage flow has been used to simulate the soil water flow in frozen and unfrozen soil. The interactions between the atmosphere and ground surface are modelled using a sophisticated numerical method which considers the climatic effects i.e. precipitation (rainfall and snowfall), evaporation effects and ground temperature estimations, using various governing equations as briefly explained in the following sections. The outcome of the soil water content simulation is the water content distribution inside the soil slope on a day to day basis. In the slope stability analysis, a traditional limit equilibrium technique based on the Morgenstern and Price (1965) method is used to calculate the factor of safety (FOS). The unsaturated shear strength of soil is also considered, as discussed below. The proposed numerical simulations were performed in a code GeoStudio using Vadose/W (Krahn, 2012a) and Slope/W (Krahn, 2012b) modules. As of author's knowledge, this study is the first attempt of this kind to investigate the effects of extreme climate conditions on non-frost susceptible soil slope stability in seasonal cold regions. The application of the recommended numerical approach is expected to contribute to the pre-design studies of soil slopes.

3.1 Governing equations for non-isothermal seepage flow

The governing equation for two-dimensional seepage flow is as given by Richards, (1931) and Childs and Collins-George, (1950),

164 $$\frac{1}{\rho_w}\frac{\partial}{\partial x}\left(D_v\frac{\partial P_v}{\partial x}\right)+\frac{1}{\rho_w}\frac{\partial}{\partial y}\left(D_v\frac{\partial P_v}{\partial y}\right)+\frac{\partial}{\partial x}\left(k_x\frac{\partial\left(\frac{P}{\rho_w g}\right)+y}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y\frac{\partial\left(\frac{P}{\rho_w g}\right)+y}{\partial y}\right)+Q=m_w\frac{\partial P}{\partial t} \quad (1)$$

165 where, ρ_w = density of water (kg/m^3), P = pressure (kPa), P_v = vapor pressure of soil moisture (kPa), k_x =
166 hydraulic conductivity in the x direction (m/s), k_y = hydraulic conductivity in the y direction (m/s), Q =
167 seepage boundary flux applied over a unit length (m/s), D_v = diffusion coefficient of water vapor through
168 soil ((kg·m)/(kN·s)), y = elevation head (m), g = acceleration due to gravity (9.81 m/s^2), m_w = slope of the
169 soil water characteristic curve SWCC (1/kPa), γ_w = unit weight of water (kN/m^3) and t = time (unit
170 according to the numerical time step).

171 The governing equation for two-dimensional thermal flow is given by the concept based on Harlan and
172 Nixon (1978).

173 $$L_w\frac{\partial}{\partial x}\left(D_v\frac{\partial P_v}{\partial x}\right)+L_w\frac{\partial}{\partial y}\left(D_v\frac{\partial P_v}{\partial y}\right)+\frac{\partial}{\partial x}\left(k_{tx}\frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_{ty}\frac{\partial T}{\partial y}\right)+Q_t+\zeta V_x\frac{\partial T}{\partial x}+\zeta V_y\frac{\partial T}{\partial y}=\left(\zeta+L_w\frac{\partial\theta_u}{\partial T}\right)\frac{\partial T}{\partial t}$$

174 (2)

175 where, T = temperature (°C), ζ = volumetric heat capacity of soil (kJ/m^3-°C), k_{tx} = thermal conductivity in
176 the x-direction (kJ/(s·m·°C)), k_{ty} = thermal conductivity in the y-direction (kJ/(s·m·°C)), V_x = the Darcy
177 water velocity in x-direction (m/s), V_y = Darcy water velocity in y-direction (m/s), Q_t = thermal boundary
178 flux applied over a unit length (kJ/(sec·m^3)) and L_w = latent heat of water during phase change either
179 liquid to gas (vaporisation) or liquid to solid (fusion as ice) (kJ/kg) and θ_u = unfrozen volumetric water
180 content determined by the slope of soil freezing characteristic curve (SFCC) (m^3/m^3).

181 The seepage flow Eq. 1 and thermal flow Eq. 2 are linked by the relationships given by Edlefsen and
182 Anderson (1943) and Joshi (1993),

183 $$P_v = P_{vs}\left(e^{\frac{-P.w}{\rho.R.T}}\right) = P_{vs}h_{rair} \quad (3)$$

where, P_{vs} = saturated vapor pressure of pure free water (kPa), ρ = density of water vapor or ice (kg/m^3), w = molecular mass of water vapor (kg/kmol), R = universal gas constant (kJ/kmol·˚C), T = temperature (˚C) and h_{rair} = relative humidity of air (%).

3.2 Modelling atmosphere-ground interactions

To consider the freeze-thaw action, snowfall during winter and other climatic effects, a seepage flux boundary used in conventional seepage analysis may not be sufficient since the surface flux resulting from snowmelt and outgoing flux resulting from evaporation are needed to be properly considered in order to estimate the ground surface temperature, surface infiltration and soil water content etc. During the winter season, the precipitation occurs as snowfall in seasonal cold regions. The snow will accumulate above the soil ground until the air temperature is below 0 ˚C. Once the temperature rises high enough to melt the accumulated snow (> 0 ˚C), the snow will start to melt and will release water to the soil surface. The snow water equivalent is the water that is stored in the snowpack. The snow water equivalent is determined based on the following equation.

$$SWE_t = SWE_{t-1} + SF - SM \tag{4}$$

where, SWE_t = snow water equivalent at the present numerical time step (mm/day), SWE_{t-1} = snow water equivalent at the previous numerical time step (mm/day), SF = snowfall precipitation rate (mm/day), SM = snowmelt rate (mm/day).

The snowfall precipitation is calculated based on the amount of precipitation and air temperature according to the following relationships.

$$SF = Q_P \times P \tag{5}$$

where, Q_P = thermal factor, P = precipitation (mm/day). The thermal factor Q_p varies according to the average daily air temperature as,

$$Q_P = 0 \text{ (if } T_a > T_f \text{) and } \quad Q_P = 1 \text{ (if } T_a < T_f \text{)} \tag{6}$$

where, T_a = average daily air temperature (˚C) and T_f = freezing point temperature (0˚C). The snowmelt rate (*SM*) is determined based on an energy balance approach used by Bras (1990), Liang et al. (1994), Flerchinger and Saxton (1989). For the estimation of snow precipitation, precipitation data (including rainfall and snowfall) and air temperature data are required.

To calculate the snow depth and density of snow the following relationships are used.

$$D_{sn} = \frac{\rho_w}{\rho_{sn}} SWE_{t-1} \; ; \rho_{sn} = \frac{SWE_{t-1}}{D_{sn}} + \beta(0.55 - \rho_{sn(t-1)}) \text{ and } \beta = \begin{cases} 0.002 & \text{if } T_a < 1.0 \\ 0.002 \times T_a & \text{if } T_a \geq 1.0 \end{cases} \quad (7)$$

where, D_{sn} = snow depth (mm), ρ_{sn} = density of snow, $\rho_{sn(t-1)}$ = density of snow on the previous time step (kg/m^3), SWE_{t-1} = snow water equivalent on the previous time step (mm), β = snow consolidation parameter and T_a = average daily air temperature (˚C).

The ground surface temperature when there is snow cover in the ground is estimated using the following relationship given by Bras (1990),

$$T_g = T_{sn} - \frac{D_{sn}}{\lambda_{sn}} q_{Tg} \quad (8)$$

where, T_g = ground surface temperature (˚C), T_{sn} = temperature of snow surface considered equal to the air temperature (˚C), D_{sn} = snow depth (mm), λ_{sn} = thermal conductivity of snow (kJ/(s·m·°C)) and q_{Tg} = energy flux at the ground surface (kJ/(sec·m^2)).

The maximum rate of evaporation from a pure water surface/saturated soil pore under given climatic conditions is defined as the potential rate of evaporation (PE). If the surface soil becomes unsaturated, the amount of water inside the soil pore will become limited and the rate of evaporation begins to decline. This phenomenon is defined as the actual rate of evaporation (AE). Actual evaporation is the actual rate of evaporative flux from a partially saturated soil pore. The actual evaporation from an unsaturated soil is modelled adopting the approach given by Wilson et al. (1994),

$$AE = \frac{\Gamma Q_n + \eta E_a}{\Gamma + \eta / h_s} \tag{9}$$

where, AE = actual vertical evaporative flux (mm/day), Γ = slope of the saturation vapor pressure versus temperature curve at the mean temperature of the air (kPa/˚C), Q_n = net radiant energy flux available at the water surface (mm/day), η = psychrometric constant, E_a = flux associated with vapor pressure mixing (mm/day) and h_s = relative humidity at the soil surface (%). For the estimation of actual evaporation from unsaturated soil, relative humidity of air and soil, wind speed and net radiation are required. To model the soil surface evaporation reasonably, Wilson et al. (1994) equation is appropriate.

The infiltration, runoff and actual evaporation are considered based on the following relationship.

$$I = P - AE - R \tag{10}$$

where, I = infiltration (mm/day), P = precipitation (mm/day), AE = actual evaporation (mm/day) and R = runoff (mm/day). The runoff R is defined as the amount of water that cannot infiltrate into the soil ground. The runoff is determined by the amount of precipitation, actual evaporation and infiltration. If the precipitation is less than the anticipated actual evaporation, then the applied surface flux boundary condition will be equal to the precipitation value minus the actual evaporation and a negative flux will be applied to the node. If the precipitation minus actual evaporation is a positive value, then a positive infiltrative surface flux will be applied as a boundary condition. The amount of total surface water flux near the ground surface is determined by the length of the sloping ground (length of the surface nodes along the slope surface). The slope angle will change the length of the slope surface which in turn affects the total surface flux. The total flux near the ground surface can be expressed simply using the following relationship,

$$Q_{surface} = P \times \sum L_n \tag{11}$$

where, $Q_{surface}$ = amount of total surface flux (m^3/day), P = precipitation (mm/day) and $\sum L_n$ = summation of length between nodes along the entire slope surface (m). The length of the slope surface is indirectly proportional to the slope angle as given by,

$$\sum L_n \propto \frac{1}{S_{\alpha ngle}} \tag{12}$$

where, $\sum L_n$ = summation of length between nodes along the entire slope surface (m) and S_{angle} = slope angle (°).

The amount of surface flux that is applied at the boundary strongly depends on the slope angle and the length of sloping ground. The assumption, by neglecting the slope angle effect in runoff calculations, is not valid in terms of surface water flow simulations and runoff through a mountain valley etc. Surface water flow and runoff caused by flooding cannot be simulated using these assumptions. The adopted numerical simulations do not track the route of surface water and flow speed etc. The infiltration boundary condition for the model is defined by Gitirana (2005). If the surface soil is saturated, the pore spaces will get filled with water and no more water can get into the soil element. Conceptually, this effect is modelled considering the rate of applied surface flux *(q)* and saturated hydraulic conductivity of the soil *(k_s)*. If the soil surface is a flat ground, there is a possibility of surface ponding. If conditions for infiltration met later, the ponded water from the flat surface will infiltrate into the soil ground. To incorporate these conditions, the method given by Gitirana (2005) is useful. By this way, the effect of slope angle on surface infiltration is considered in the analysis.

3.3 Shear strength of soil under unsaturated conditions

The shear strength of an unsaturated is expressed based on Bishop's effective stress principle by Vanapalli et al. (1996) as given by,

$$\tau = c' + \left(\sigma_n - u_a\right)\tan\phi' + \left(u_a - u_w\right)\left[\chi\tan\phi'\right] \tag{13}$$

where, τ = shear strength of soil (kPa), σ_n = net total stress (kPa), u_a = pore air pressure (kPa), u_w = pore

water pressure (kPa), c' = effective cohesion (kPa), ϕ' = effective angle of internal friction (°) and χ = parameter related to the degree of saturation. According to Vanapalli et al. (1996) the magnitude of parameter χ can be expressed in terms of volumetric water content as,

$$\chi = \frac{\theta_w - \theta_r}{\theta_s - \theta_r} \tag{14}$$

where, θ_w = volumetric water content (m^3/m^3), θ_s = saturated volumetric water content (m^3/m^3) and θ_r = residual volumetric water content (m^3/m^3).

3.4 Factor of safety estimation for slope stability analysis

Slope stability has been analysed using limit equilibrium technique based on the method given by Morgenstern and Price (1965). The Morgenstern and Price method is a widely used slope stability method in general geotechnical engineering practice. The factor of safety equations with respect to moment equilibrium (F_m) and force equilibrium (F_f) considering the unsaturated shear strength of soil are given in Eq. 15 and Eq. 16 respectively. The unsaturated soil shear strength in the factor of safety is considered based on the nonlinear relationship given by Vanapalli et al. (1996) as explained in Eq. 13 and Eq. 14.

$$F_m = \frac{\sum\left[c' lR + \left\{N - u_w l\chi - u_a l\left(1-\chi\right)\right\} R\tan\phi'\right]}{\sum Wx - \sum Nf \pm \sum Dd \pm \sum Aa} \tag{15}$$

$$F_f = \frac{\sum\left[c' l\cos\alpha + \left\{N - u_w l\chi - u_a l\left(1-\chi\right)\right\}\tan\phi'\cos\alpha\right]}{\sum N\sin\alpha - \sum D\cos\omega \pm \sum A} \tag{16}$$

where, W = the total weight of a slice of width b and height h (kN/m^2), N = the total normal force on the base of the slice (kN), D = an external point load (kN). R = the radius of a circular slip surface (m), x = the horizontal distance from the centerline of each slice to the center of rotation or to the center of moments (m), d = the perpendicular distance from a point load to the center of rotation or to the center of moments (m), f = the perpendicular offset of the normal force from the center of rotation or from the center of moments (m), a = the perpendicular distance from the resultant external water force to the center

of rotation or to the center of moments (m), A = the resultant external water forces (kN), ω = the angle of the point load from the horizontal (°), α = the angle between the tangent to the center of the base of each slice and the horizontal (°) and l = the base length of each slice (m).

4. Study of embankment slope failure using the recommended approach

The case example of failure on embankment slope constructed using Shikotsu Komaoka volcanic soil is analysed using the recommended approach.

4.1 Numerical model, soil properties and analytical conditions

The two-dimensional numerical finite element mesh with the slope geometry adopted for the embankment slope is given in Fig. 4. Shikotsu Komaoka volcanic soil has been used as the slope material in the embankment slope. The soil parameters used for the numerical simulation of embankment slope are summarised in Table 1. The parameters i.e. dry density (ρ_d), hydraulic conductivity of saturated soil (k_s), effective cohesion (c') and effective angle of internal friction (ϕ') have been obtained from laboratory element tests (Matsumura et al. 2015). The parameters for which no laboratory measurements are available, i.e. thermal conductivity (λ), volumetric heat capacity (ζ) and volumetric water content of soil at 0°C (θ_{wf}), have been estimated using equations given by Kersten (1949), Jame (1977) and Black and Tice (1989), respectively. The soil-water characteristic curve (SWCC) was obtained from laboratory element tests by Matsumura et al. (2014). The coefficient of permeability under unsaturated conditions is estimated using the SWCC (van Genuchten, 1980). The initial distribution of the volumetric water content and temperature of the slope has been configured based on the soil water content and temperature data recorded during day 1 (09-11-2012). To derive an equilibrium of volumetric water content distribution corresponding to day 1 (09-11-2012), the average volumetric water content recorded at locations SML0, SML1, SML2 and SML3 (as shown in Fig. 1 and Fig. 4) are specified exactly at the corresponding locations in the numerical model. For the temperature distribution on the initial day

316 (09-11-2012), the measured temperature has been specified at the depths of 0 m to 1 m. For the climatic
317 boundary conditions used in the transient non-isothermal seepage model, climate data i.e. maximum and
318 minimum air temperature, average daily rainfall, maximum and minimum relative humidity, average daily
319 wind speed and average daily net radiation are required. During the monitoring of the volcanic soil
320 embankment slope, the temperature and rainfall were monitored. Additional required climatic data were
321 obtained from the AMeDAS (Automated Meteorological Data Acquisition System) data provided by the
322 Japanese Meteorological Agency (JMA) and as given in Fig. 5.

323 **4.2 Results and discussions**

324 From the numerical simulations, the magnitude of the various influencing factors i.e. precipitation,
325 accumulated snow depth, snowmelt water, ground temperature, net surface infiltration etc. was analysed
326 and finally the factor of safety of the slope was estimated. The numerical results are compared with the
327 measured data.

328 A comparison of volumetric water content has been made from the numerical results and monitoring data
329 as given in Fig. 6(a) showing the comparison between numerical estimation and measurement of the
330 average volumetric water content of the soil water content sensors at SML0, SML1, SML2 and SML3
331 installed at depths 0.2 m to 1.5 m. The ground temperature estimated from the numerical simulation is
332 compared with the measured data as given in Fig. 6(b). The numerically estimated snow depth and
333 measured snow depth is compared and given in Fig. 6(c). A close similarity between the numerical
334 estimation and measured data has been found. Slope failure will occur if the soil is saturated near the
335 slope surface and the failure may trigger along the slip surface. In such circumstances, the reason for
336 failure could be judged by estimating the average volumetric water content. The average volumetric water
337 content is compared here in order to visualise the saturation of the slope. It may be seen that the average
338 volumetric water content at locations SML1, SML2 and SML3 reaches to a range of 0.52 m^3/m^3 to 0.56

339 m^3/m^3. The saturated volumetric water content of the soil is 0.63 m^3/m^3 as given in Table 1. From this
340 observation, it could be said that during the day of the slope failure the degree of saturation (S_r) was up to
341 85 % on the slip surface. The numerical simulation results match well with the trend of the measured data.
342 The maximum difference found between the numerically estimated volumetric water content and
343 measured volumetric water content is 0.02. The stability of the soil slope starting from the day of slope
344 construction until the failure is expressed as a factor of safety and has been plotted in Fig. 6(d). The factor
345 of safety during the day of slope failure is 0.954. The slip surface estimated using the limit equilibrium
346 method is compared with the field slope failure data, as shown in Fig. 7 (a) and (b) respectively.
347 In field conditions, the failure surface was about 3 m in height and 0.6 m to 0.8 m in depth. From the
348 numerical simulation, the failure surface is estimated as 4.5 m in height and 0.6 m to 0.9 m in depth. The
349 numerical simulation demonstrated close similarity in estimating the influencing parameters i.e. net
350 surface infiltration, snow depth, snowmelt water and ground temperature etc. and could predict the soil
351 water content distribution of the slope appropriately. The numerically estimated data closely matches with
352 the measured data almost in all circumstances. Through these observations, it could be claimed that the
353 adopted approach is reliable to predict the soil slope stability in seasonal cold regions.

354 **5. Study of the highway slope failure using the recommended approach**

355 The slope failure that occurred along the national highway route 230 (highway slope failure), is analysed
356 using the recommended approach. The two-dimensional numerical model, boundary conditions and
357 material properties were considered to be similar to the embankment slope failure model and are
358 discussed in the following section.

359 **5.1 Numerical model, soil properties and analytical conditions**

360 The two-dimensional numerical model for the highway slope failure is designed using the geological
361 cross-sectional data. The two-dimensional numerical model with the soil/rock stratigraphy and surveyed

362 ground water table are given in Fig. 8. The slope stratigraphy has three soil/rock types, namely
363 embankment filling, talus slope materials and the bedrock (Andesite). The surveyed ground water table is
364 at an average depth of 8 m from the ground surface. The soil material properties used for the numerical
365 simulation of the highway slope model are given in Table 2. The parameters i.e. dry density (ρ_d),
366 hydraulic conductivity of saturated soil (k_s), and undrained shear strength (q_u) have been obtained from
367 laboratory measurements (Hokkaido Regional Development Bureau, 2013). The parameters for which no
368 laboratory measurements are available, i.e. thermal conductivity (λ), volumetric heat capacity (ζ),
369 volumetric water content of soil at 0°C (θ_{wf}), effective cohesion (c') and effective angle of internal friction
370 (ϕ') have been estimated using methods given by Kersten (1949), Jame (1977), Black and Tice (1989) and
371 Hoek and Brown (1977), respectively. The SWCC of the embankment filling material and Talus
372 sediments were estimated from the grain size distribution curve based on the method given by Fredlund et
373 al. (2002). For the embankment filling and talus material, the unsaturated shear strength data is not
374 available so that the available undrained shear strength is considered for the embankment filling and
375 saturated shear strength is estimated for the talus slope material. The bedrock is modelled as a low
376 permeability material.
377 For the long-term slope stability analysis, drained shear strength parameters are needed. The effective
378 cohesion (c') and effective angle of internal friction (ϕ') for the embankment filling material were derived
379 using rigorous back calculation methods given by Duncan and Stark (1992), Okui et al. (1997) and Zhang
380 et al. (2013). Several values of cohesion and angle of internal friction are given initially. By setting the
381 FOS = 1 and using the reference ground water table derived from the non-isothermal seepage simulation
382 and using the pre-known dimensions of the slope failure, several iterations of calculations have been
383 made from which different cohesion and angle of internal friction values are obtained. In consideration to
384 the height of the failure surface, the effective cohesion and effective angle of internal friction for the
385 embankment soil were derived as shown in Fig. 9 (a) and (b). Back-calculation of shear strength

parameters may have some limitations concerning with the precision of the estimated shear strength (Tang et al. 1999; Duncan and Wright, 2005; Deschamps and Yankey, 2006). Since the major objective of this study is to examine the influencing parameters of slope failure, the problems with the precision of the estimated shear strength may be negligible and these values can be used as a basis for a parametric study. The shear strength of the Talus materials was intentionally kept larger so that the steep portions of the mountain slope located far away from the embankment does not influence the factor of safety calculations. Prior to the transient non-isothermal coupled seepage analysis, an initial equilibrium condition in terms of pore water pressure and the ground temperature is necessary. The ground water table has been measured from the geological survey performed after the slope failure. The ground water table line from the geological survey is kept as a reference and an average of 10-year climate data recorded between years 2002 to 2012 obtained from a meteorological telemetry at Mui Ne, Hokkaido, Japan maintained by the Ministry of Land, Infrastructure, Transport and Tourism, Hokkaido Regional Development Bureau (MLIT) has been used and to derive the initial equilibrium. The climate data used for the numerical simulation is also obtained from the meteorological telemetry at Mui Ne which is closest to the disaster site. The climate data is given in Fig. 10. The air temperature on the day 1 (01-04-2012) is just close to 0˚C and it increases during the thawing period. The maximum rainfall of 92 mm/day occurred during the day of slope failure 07-04-2013.

5.2 Results and discussions

To study the changes in pore water pressure and volumetric water content of the embankment, histories were given at particular locations 1 to 6 in the numerical model, as shown in Fig. 8. The variation in volumetric water content and pore water pressure are given in Fig. 11 (a) and Fig. 11 (b) respectively. From Fig. 11, it could be seen that there is a higher fluctuation of soil water at locations 1 and 2. During the day of slope failure, the volumetric water content and pore water pressure at all locations of the

409 embankment reaches 0.47 m^3/m^3 and 0 kPa respectively, as shown in Fig. 11 (a) and (b). The saturated
410 volumetric water content of the slope is 0.47.
411 The ground temperature and accumulated snow depth measured at locations 1 to 6 are given in Fig. 11(c)
412 and (d). The ground temperature rises above 0˚ after March 12, 2013. The maximum accumulated snow
413 depth is 3708 mm at location 1 (x=-7 and y=490). A comparison of snowmelt with the measured data
414 could not be obtained in this case because the snow depth varies with location. The accumulated snow
415 started to melt from March 12, 2013. During this period, the stability may get reduced due to the
416 snowmelt water. The factor of safety estimated from the simulation is plotted in Fig. 12(a).
417 During initial days from 01-04-2012, the stability of the slope reduced rapidly due to continuous
418 precipitation until early May 2012. During the month of June 2012, there is not much precipitation
419 observed and hence an increasing trend in safety factor is estimated. Further, the factor of safety reduces
420 whenever there is continuous precipitation. During the month of October to November 2012, the stability
421 reduces rapidly due to continuous precipitation. A similar trend continues until December 2012. The
422 ground temperature reduces below 0˚ C during January 2013 and the soil surface remains frozen until
423 March 12, 2013. During this period, an increase in the factor of safety is observed and later the factor of
424 safety value keeps nearly constant. The reduction in the factor of safety commences once the ground
425 temperature increases above 0˚ C, exactly on March 12, 2013. The reduction continues markedly, and on
426 the day, April 07, 2013, the factor of safety reaches a value 0.992 which denotes the slope failure. The
427 surveyed failure surface and the numerically estimated failure surface with the factor of safety on April 07,
428 2013 is given in Fig. 12(b). The height of the numerically estimated slip surface is 19 to 20 m and 30 to
429 35 m in length. From the geological survey, the length of the failure is around 40 m with a height of 20 m.
430 The minimum safety factor 0.992 has been found for the critical slip surface passes through the
431 embankment portion.

6. Parametric studies

The effects of the factors i.e. freeze-thaw action, snowmelt water infiltration and the weight of the accumulated snow need to be known for the precise assessment of the long-term stability of soil slopes i.e. embankment and cut slopes along the highways. Based on this purpose, in this study, a series of parametric studies using the recommended numerical simulation method has been performed using the embankment slope model and highway slope model with the analytical conditions as given in Table 3.

6.1 Effect of increased magnitudes of rainfall on slope stability

The amount of rainfall has been increased twice and thrice in order to see the effect on slope stability using the embankment slope model. The twice and thrice considered rainfall magnitude significantly reduces the stability as given in Fig. 13 (a).

6.2 Effect of increased magnitudes of snowfall on soil slope stability

To clearly visualise the effect of snowmelt on slope failure, the snowfall precipitation has been configured in different magnitudes, like twice and thrice for the embankment slope model. The stability of the slope abruptly reduces during March-April-May months due to snowmelt and surface infiltration. It could be seen that as the depth of accumulated snow increases, the duration took to melt the snow also increases and finally it results in a reduction in safety factor as shown in Fig. 13 (b). The failure-inducing factor is not only the rainfall or not the snowmelt water alone. The rainfall along with snowmelt water induces such a reduction in slope stability. Through the parametric studies, it is found that the accumulated snow depth increase may result in an excess amount of snowmelt water which may eventually reduce the soil slope stability.

6.3 Effect of freeze-thaw action on soil slope stability

Two different numerical simulations, one considering the freeze-thaw process and the second without considering the freeze-thaw process were performed using embankment slope model and highway slope model. When there is no freeze-thaw action considered, the effects of soil water freezing, the effect of the latent heat phase change and the temperature flow are neglected. The calculation of all other variables i.e. precipitation (rainfall and snowfall), air temperature, relative humidity etc. are kept same for both the analysis and the shear strength of frozen soil is not considered for this parametric study. For the simulation considering freeze-thaw action, the soil water content distribution is estimated using non-isothermal seepage flow and for the simulation without freeze-thaw action, the soil water content distribution is estimated using isothermal seepage flow. These analyses were performed for both the embankment slope model and highway slope model. The factor of safety for both the simulations was analysed and compared as given in Fig. 14(a) and (b). A stability difference could be seen between analysis considering freeze-thaw action and analysis without freeze-thaw action during the period when the ground surface temperature is below zero for both the cases. In the case example of embankment slope, the freeze-thaw action has very small impact on soil water content and factor of safety. Whereas, for the highway slope, a major difference in the factor of safety between analysis considering freeze-thaw action and analysis without considering freeze-thaw action is observed. On the day of slope failure, the factor of safety estimated by the analysis considering freeze-thaw action is 0.992 and the factor of safety estimated by the analysis without considering freeze-thaw action is 1.003. From this observation, it could be said that the freeze-thaw action has a considerable effect on soil water content fluctuation which in turn affects the slope stability and for the proper assessment of the stability of soil slopes in seasonal cold regions, the freeze-thaw action must be considered in estimating the soil water content distribution.

6.4 Effect of snowfall/snowmelt water infiltration on soil slope stability

The snowfall accumulated above the slope during the winter season will start to melt once the air temperature increases above the phase change temperature (0˚C). During this snow melting period, the snowmelt water will infiltrate into the soil or runoff above the slope based on the ground surface temperature. From Fig. 14(c) and (d), it can be seen that the factor of safety does not get reduced during the months of March, April and May for the analysis without considering snowmelt water. The reason is due to the lack of snowmelt water. Whereas when snowfall is considered, the factor of safety abruptly reduces during the snow melting season. The contribution of snowmelt water in the net surface infiltration is high during the thawing season which abruptly reduces the slope stability. One more important observation made from the embankment slope model is that when there is no snow on the ground the safety factor starts deviating from the early February 2013 itself. The reason behind this phenomenon is, if there is no snow accumulated on the soil ground, the soil temperature may be much lower than it would be under the accumulated snow. There would not be a heat flux variation in this case. Due to this fact, the ground will freeze up to a certain depth more than it will freeze under the accumulated snow. If the ground is frozen over greater depth, it will become impermeable and there will not be any surface infiltration. The factor of safety will be high during the winter period if there is no snow accumulated on the soil ground. Due to the absence of snowmelt water, during snowmelt period, there will not be any reduction in stability. Similar behaviour has been observed from the highway slope model as well. It is very interesting to note that for the highway slope if the snowmelt water infiltration is ignored, the factor of safety and stability of the slope is considerably higher throughout the year from 01-04-2012 to 07-04-2013. On the day of slope failure, the factor of safety of slope when the snowmelt water is ignored is 1.224 which emphasises considerable stability. In the case of simulation, in which the snowmelt water is included in the calculation of soil water content, the factor of safety on the day of slope failure is 0.992 which clearly denotes the slope failure. Based on this observation, it could be said that for the case of

498 highway slope failure, the 92 mm cumulative rainfall alone would not have induced the slope failure. The
499 water originating from the snowmelt that infiltrated into the soil ground together with the cumulative
500 rainfall should have induced the disaster.

501 One more interesting observation found for the highway slope model is that the difference in the
502 factor of safety during the month of December 2012 between analysis considering snowfall and analysis
503 neglecting snowfall, as shown in Fig. 14(d). The air temperature fluctuates below and above 0˚C during
504 the starting of the freezing season, November and December 2012. In this situation, there is a possibility
505 of rainfall and snowfall together. If there is accumulated snow above the soil ground originating from the
506 previous day snowfall, on the next day if the air temperature is above 0˚C and there will be precipitation
507 in terms of rainfall. The factor of safety reduces to 1.137 on December 25, 2012, when the effect of
508 snowmelt water is considered in the analysis. Whereas, for the analysis without the effect of snowmelt
509 water, the factor of safety is 1.507 on December 25, 2012. Based on this observation it could be said that
510 the snowfall that occurs during the initial stages of freezing (November and December months) will also
511 influence the slope stability considerably.

512 From these parametric studies, it is very clear that snowmelt water infiltration seriously affects the soil
513 slope stability and it should be considered in long-term slope stability analysis of embankment structures
514 and cut slopes along the highways in seasonal cold regions.

515 **6.5 Effect of weight of snow on soil slope stability**

516 The snow accumulated on top of the slope surface has its own weight. In such cases, it may induce a
517 reduction in slope stability. To analyse this effect, the weight of the snow is considered as a surcharge
518 load. The snow load is estimated using the snow density (ρ_{sn}) and accumulated snow depth. As the
519 accumulated snow depth increases, the snow load starts to build up. The maximum snow load calculated
520 for 1689 mm snow depth is 7 kN/m^2. To see whether this snow load will affect the slope stability or not,

the load is considered as a surcharge pressure in the slope stability analysis. Since it is not possible to consider the increasing snow load day by day in a safety factor calculation, a 25 days averaging has been adopted, as given in Fig. 15(a). The safety factor seems to be affected much by the accumulated snow weight, as given in Fig. 15(b).

In Fig. 15, the factor of safety obtained with and without snow load both considers the effect of snowmelt water. It seems the weight of snow combined with the snowmelt water may affect the slope stability to a considerable amount during the freezing and snow melting seasons, respectively. The maximum difference in safety factor is observed during the month of March 2013 when the snow load reaches its maximum. Once the snow starts melting, the load of the snow will decrease and diminish when all the accumulated snow is converted into water.

6.6 Effect of slope angle on soil slope stability

Kawamura and Miura (2013) performed experiments with small scale model soil slopes made up of volcanic soil material under different slope angles considering a slope angle range from 45˚ to 65˚. For the design of embankment slopes and cut slopes along the express highways in Japan, the NEXCO (Central Nippon Expressway Company Limited) has set some guidelines (Yasuda and Fujioka, 2012). For the design of embankment slopes in express highways, the standard slope would be 1:1.8 that is 29˚ and for highways other than the national expressways in Japan, a standard slope angle ratio of 1:1.5 to 1:1.8˚ is adopted depending on the soil/rock material type underlying the embankment. Further, for natural cut slopes along the highways, the slope may range up to a slope angle of 60˚ if the base of the soil is rock. In view of all these considerations, a study to evaluate the effect of slope angle is performed by making numerical slope models with different slope angles. The slope angles are considered to be 30˚, 35˚, 40˚, 45˚, 50˚, 55˚ and 60˚, though the physical embankment model considered in this study was built at an angle of 45˚. In order to study the effect of slope angle, 30˚, 35˚, 40˚, 45˚, 50˚, 55˚ and 60˚ numerical

slope models are made and simulated with the recorded climatic measurements. It may be seen from Fig. 15(c) that the safety factor of the slope varies with different slope angles. During the slope failure day, October 17, 2013, the safety factor for slope angles 45˚ to 60˚ are lesser than slope angles 30˚, 35˚ and 40˚. For shallow slopes, 30˚, 35˚ and 40˚, the factor of safety is more than 1. Interestingly, the reduction in safety factor during the snow melting season is more for the shallow slopes 30˚, 35˚, 40˚ than 45˚, 50˚, 55˚ and 60˚ slopes. The variation in the factor of safety between different slope angles is governed by the infiltration and runoff which will vary significantly according to the slope angle.

6.7 Effect of shear strength of frozen soil

Chen et al. (2013) analysed the effect of shear strength of frozen soil on soil slope stability conceptually. When the soil is frozen, it will gain additional shear strength depending on the ice content. In the case study of volcanic soil embankment, during the freezing months, a maximum freezing depth of 0.2 m is observed from the embankment monitoring (Matsumura, 2014). The region where the soil water is frozen would have gained additional shear strength which could result in an increase in stability. In any case, in this study so far this phenomenon is not considered due to the following reasons,

1. The additional shear strength will only increase the stability which will eventually increase the factor of safety.
2. Once the frozen water content of the soil melts, the additional shear strength will diminish.
3. Since the freezing depth in the present study is only 0.2 m (20 cm) it would not have strongly influenced the overall stability of 5 m tall slope.

In order to check the generality of the above-mentioned statements, a slope stability analysis considering additional shear strength in terms of effective cohesion and effective angle of internal friction is performed. Due to non-availability of frozen shear strength data, basic empirical equations as referred from the literature is adopted to calculate the frozen soil shear strength parameters. The frozen soil shear

strength is estimated as given by Arenson and Springman (2005) and Chen et al. (2013), assuming that the effective angle of internal friction ϕ' is independent of temperature and strain rate, and the effective cohesion c' is independent of strain rate.

$$\phi_f = \phi' - \phi'(\theta_i)^{2.6} \tag{17}$$

ϕ_f = angle of internal friction of frozen soil (˚), ϕ' = effective angle of internal friction (˚), θ_i = volumetric ice content (m^3/m^3). For the estimation of frozen shear strength, two variables viz. unfrozen water content and unfrozen ice content of the soil are needed. The unfrozen water content of the soil is estimated using the relationships given by Black and Tice, (1989).

A simple empirical estimation for cohesion of frozen soils is given by Chen et al. (2013),

$$C = \begin{cases} c_f(T,\theta_{uw}) = (\beta_1.T + \chi_1)(\beta_2.\theta_{uw} + \chi_2), T_g < 0°C \\ c', T_g \geq 0°C \end{cases} \tag{18}$$

C = cohesion (kPa), c' = effective cohesion at unfrozen state (kPa), C_f = cohesion at frozen state (kPa), T = Temperature (˚C), θ_{uw} = unfrozen volumetric water content (m^3/m^3), β_1 = 0.074, β_2 = −5.14, χ_1 = −0.015, χ_2 = −1.23 fitting parameters for the cohesion function for coarse grained soils.

Since the motivation of considering the estimation equation in this study is to only see the effect of frozen shear strength on slope stability, the maximum cohesion value and minimum angle of internal friction are specified to the frozen soil zone up to 0.2 m depth and a slope stability analysis has been performed.

The increment in safety factor due to frozen shear strength seems to be very minimal and it has very little impact on the safety factor for the analysis performed, as shown in Fig. 15(d). There could be two possible reasons behind this behaviour. The increased frozen shear strength is too low (only 0.67 kPa) and the freezing depth is very small (0.2 m) comparing with the overall slope geometry. However, if the frozen soil occupies an adequate amount of the slope slip surface, the frozen shear strength should be properly considered for the estimation of slope stability.

7. Conclusions

A slope stability assessment approach applicable to analyse the long-term stability of soil slopes i.e. embankment slopes, and natural cut slopes along highways in seasonal cold regions is recommended in this study. The validity of the proposed approach has been studied by applying the method for two soil slope failure case examples that occurred in a seasonal cold region in Hokkaido, Japan. Based on the detailed studies performed, it has been found that the adopted slope stability assessment approach can be used for the design of slope structures along highways through performing pre-assessment of stability against anticipated climatic influences. Through the various parametric studies performed, the following conclusions can be drawn.

- The freeze-thaw action has a considerable effect on soil water content distribution which in turn influences the stability of soil slope. It is important to consider the freeze-thaw action in estimating the soil water content distribution for the proper pre-judgement of slope stability.
- The soil water content distribution inside the soil slope is seriously affected by the snowmelt water infiltration. Ignoring the amount of water derived from the snowmelt may not predict the soil water content distribution of the slope precisely.
- The weight of snow accumulated above the soil ground may reduce the slope stability considerably. The consideration of snow weight in the estimation of the factor of safety is essential if the snow during accumulation is not removed manually.
- The frozen shear strength of the soil should be properly considered to analyse the stability during freezing seasons if it is expected that the soil ground will freeze for a considerable depth.
- The freeze-thaw action and snowmelt water infiltration have a substantial impact on soil slope stability regardless of the slope angle considered.

In geotechnical engineering practice, the factors such as freeze-thaw action, snowmelt water infiltration are generally not considered for the design and analysis of slope structures, especially for cold regions like Hokkaido. It is evident that these factors seriously affect the soil slope stability. The stability assessment approach adopted in this study is found to be very useful in analysing the slope stability against anticipated climatic factors i.e. freeze-thaw action and snowmelt water infiltration. Based on the studies performed, it is strongly recommended that an assessment of the long-term stability of soil slopes considering the climatic effects will be helpful for the prevention and mitigation of such disasters.

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