Stability of Thawing Slopes: Field and Theoretical Investigations

La Stabilité des Pentes Décongelantes: Récherches Pratiques et Théoriques

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SYNOPSIS Stability analyses of thawing slopes in cohesive soils are examined. On the basis of field investigations it was found that current methods which explain instabilities in thawing slopes are not correct in assuming the failing mass as a continuous and rigid translational block. Instead the structure of the failing mass was recorded to be composed of a deformable mixture of soil lumps and muddy water, the product of the thawing of a reticulate ice vein network subdividing the previously frozen soil into irregular blocks. A theoretical approach of stability analysis based on the particulate structure of the failing thawing slopes is presented.

INTRODUCTION

Shallow instability forms in thawing natural slopes in cold and temperate regions are explained to date using three approaches based on the infinite slope stability analysis which uses the Mohr-Coulomb criterion for failure and assumes, therefore, the sliding soil as a rigid-homogeneous-continuous mass. Two of them, the "ice-blocked drainage" approach developed by Chandler (1970) and the McRoberts-Morgenstern (1974) approach based on the thaw-consolidation theory, use effective strength parameters in their analysis as follows. At limit equilibrium, when movement is just possible, the shear stress on the plane of failure is equal to the residual shear strength of the thawed soil, or

$$\gamma\ d \sin\beta = c'_r + (\gamma\ d \cos\beta - u)\tan\phi'_r \qquad (1)$$

where γ, c'_r and ϕ'_r are the bulk unit weight, the effective residual cohesion intercept and the effective residual angle of internal friction of the thawed soil. β is the slope angle, d is the thickness of the failing mass, and u is the pore water pressure on the slip surface. The third approach is the one developed by Hutchinson (1974), based on the total stress analysis of stability. Hutchinson assumes that the thawed soil is completely cohesive and that during failure, undrained conditions take place. The resistance to movement is provided by the undrained shear strength, c_u, of the thawed soil. Therefore, at limit equilibrium, the following relationship applies,

$$\gamma\ d \sin\beta = c_u \qquad (2)$$

It is the aim of this paper to assess the adequacy of the approaches introduced above.

FIELD INVESTIGATIONS

Two natural slopes in frost susceptible soils on the western shore of Lake Michigan at the location of Kewaunee were studied. However, due to space limitations, the present study will focus on one of them, Kewaunee B slope. The engineering properties of the soils forming the slope as well as its profile changes covering a period of three years are shown in Table 1 and Figure 1. The Kewaunee B slope is stable with respect to deep failures (Vallejo, 1977), thus shallow failures are the only cause of instability.

Critical Depth of Thaw

Because fluctuations in air temperature above and below the freezing point occur in the Kewaunee area (Fig. 2) the surface of the Kewaunee B slope is subjected to alternate periods of freezing and thawing. During thawing, failure of the slope surface takes place when a

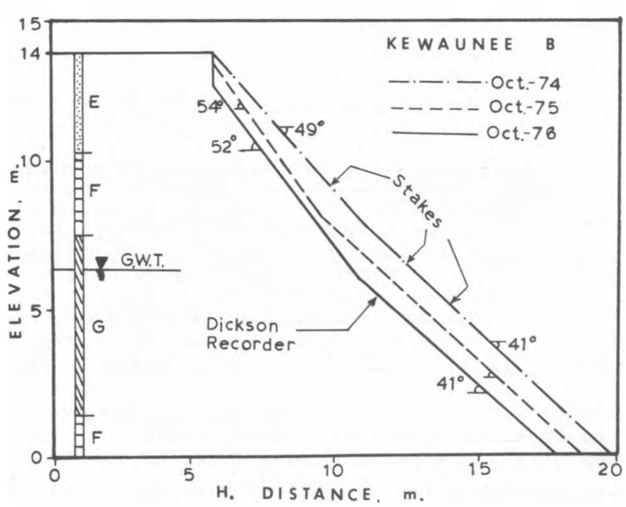

Fig. 1 Slope Profiles Just Before Freezing Conditions.

critical depth of thaw is reached. Field and theoretical investigations on the critical depth of thaw for the Kewaunee B slope were undertaken during the winter-spring seasons of 1974-1975 and 1976-1977.

Three metal stakes, 1.59 cm. in diameter and 61 cm. in length were driven normal to the face of the Kewaunee B slope on March 17 of 1974 during freezing conditions (Figs. 1 and 2). The stakes were driven 55.9 cm. into the ground. When the stakes were recovered in May 1975, they were bent at a depth of 30.5 cm. from their top. This means that a depth of soil equal to 25.4 cm., representing the critical depth of thaw, d , normal to the slope face, moved down to the slope toe. When the stakes were recovered, they were in place with a depth

TABLE I. Properties of Intact Soil

Soil Designation	Soil Description. Symbol*	w_L	I_p	ϕ'_p deg.	c'_p kN/m^2	c_u kN/m^2	γ** kN/m^3	% clay	% silt	% sand
E	Fine Sand (SP)			30	0		17.3			100
F	Brown Silty Clay (CL)	30	9	31	20	90	20.7	60	39	1
G	Gray Clayey Silt (ML)	36	3	35	8	95	20.3	31	48	21

* Unified Soil Classification System ** Bulk Unit Weight

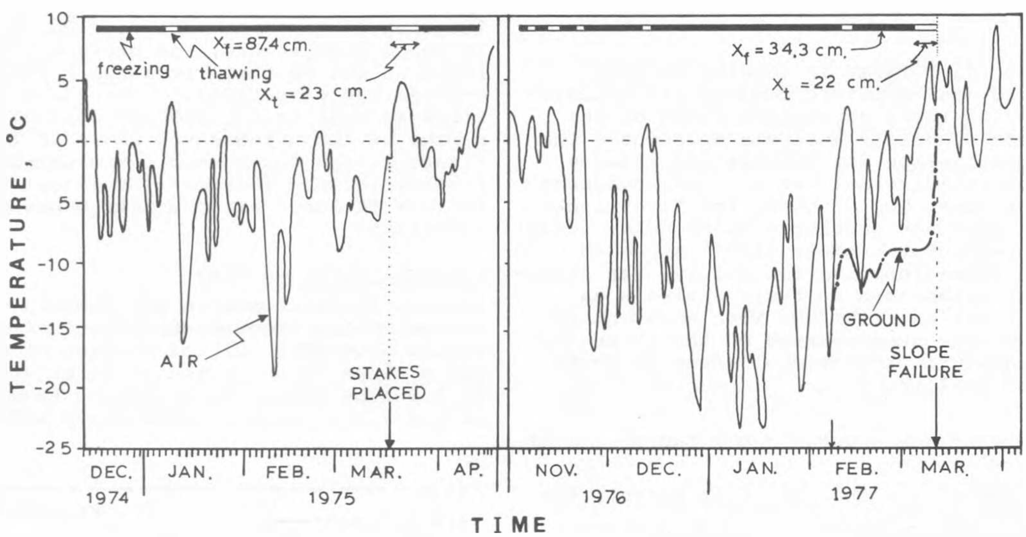

Figure 2. Air and Ground Temperatures

TABLE II. Properties of Thawed Gray Clayey Silt (G, Table 1 and Fig. 1)

w %	γ kN/m^3	γ_d kN/m^3	ϕ'_r deg. (1)	c'_r kN/m^2 (1)	c_u kN/m^2 (2)	k_u $\frac{Joules}{m - hr - °C}$ (3)	T_s °C (4)	L $\frac{Joules}{m^3}$ (5)	c_v $\frac{m^2}{hr}$ (6)
30	18.38	14.14	35	1.9	1.4	4.67×10^3	2.2	14.5×10^7	14×10^{-5}

(1) Obtained from direct shear tests. (2) Obtained at a depth of 10 cm. of thawed soil on the Kewaunee slope using a vane shear apparatus. (3) k_u = thermal conductivity of thawed soil, obtained from Kersten (1949) Tables. (4) T_s = step temperature above 0 °C causing the thawing failure of slope (Fig. 2). (5) L = Latent heat of fusion of soil = 34.169×10^4 (w%) (γ_d). (6) c_v = coefficient of consolidation, obtained from consolidation tests on the clayey lumps forming part of the frozen and thawed soil.

equal to 30.5 cm. inside the ground. Therefore, when the failure took place, a subsurface frozen layer must have been present which served as an anchor to the stakes and prevented their removal. Using the Modified Stefan Equation (Kersten, 1949) together with the air temperatures (Fig. 2) and the properties of the soils when frozen and thawed (Table 2), Vallejo (1977) calculated the depth of frost penetration (X_f) during the freezing period when the stakes were placed as well as the depth of thawing (X_t) just after the stakes were placed (Fig. 2). The depth of frost penetration was equal to 87.4 cm. and exceeded the depth of thawing (equal to 23 cm., which is very close to the critical depth of thaw obtained using the stakes) by 64.4 cm. Therefore, a frozen subsurface layer was present if failure took place after placing the stakes.

An investigation to measure the ground temperature on the face of the Kewaunee B slope was undertaken on the winter of 1976-1977. A Dickson temperature recorder described in detail by Vallejo (1977) was installed in the top 21 cm. of the slope face on Feb. 7, 1977 (Fig. 1 and 2). On March 29 the recorder was found at the toe of the slope together with the soil product of a shallow slope failure. The temperature recorded by this apparatus before and during failure (equal to 2.2 °C) is shown in Fig. 2. Also shown are the depth of frost penetration and thawing just before failure took place.

For the stability calculations a critical depth of thaw, d , equal to 0.254 m. and measured in a direction normal to the slope face will be used.

STABILITY CALCULATIONS

Particulate Structure

From field studies in the winter of 1978-1979 the structure at shallow depths of the frozen cohesive soils forming the Kewaunee B slope were found to consist of a reticulate ice vein network subdividing the frozen soil into irregular blocks. The ice veins were generally alligned in parallel and vertical direction to the ground surface. The vertical veins formed when ice filled shrinkage cracks in the soil.

According to Linell and Kaplar (1959), the necessary conditions for the formation and growth of ice veins which give a particulate structure to a cohesive soil are: a) low overburden pressures (shallow depths), b) a soil with 3 to 10% of grains smaller than 0.02 mm., c) a free supply of water to the ice veins which usually comes from the blocks of soil between the fissures or the ground water table, d) slow rate of freezing, and e) capillary saturation of the soil at the beginning and during the freezing process. An examination of Figures 1 and 2 and Table 1 show that the cohesive soils forming the Kewaunee B slope meet most of the above requirements which explains the formation of a particulate structure at shallow depths.

Upon reaching the critical depth of thaw, the structure of the Kewaunee B slope will then consist of a mixture of lumps of clayey soil and water. This water can change to mud after failure has taken place and the water has mixed with part of the soil forming the lumps as reported by McRoberts and Morgenstern (1974). Therefore, Equations (1) and (2) which assume the soil to behave as rigid-homogeneous-continuous mass and do not consider the particulate structure of the thawed mass can not be rigorously used to analyze the stability of the Kewaunee B slope.

Also, the fissures and joints in the thawed soil, resulting from the ice veins, will prevent the development of excess pore water pressures. The Chandler (1970) and McRoberts-Morgenstern (1974) approaches involve excess pore water pressures in their stability analysis. They have used excess pore water pressures to explain failures in low-angled clay slopes. When thawing takes place, the excess pore water pressures on the slip plane can be obtained from the following equation (McRoberts and Morgenstern, 1974)

$$u_1 = (\gamma - \gamma_w) \, d \cos \beta \left(\frac{1}{1 + \frac{L \, c_v}{k_u \, T_s}} \right) \tag{3}$$

where γ_w is the unit weight of water (9.8 kN/m3) and the rest of terms defined and given in Table 2 for the case of the G layer (Fig. 1). Using the values of Table 2 and $\beta = 41°$ (Fig. 1), a value for the excess pore water pressure, u_1 , equal to 0.553 kN/m2 is obtained. However, this excess pore water pressure can not exist because the fissure and joint systems prevent its development.

In addition, the fissure and joint systems in the thawed soil will prevent undrained conditions ($\phi = 0$) from taking place during failure. Therefore, the Hutchinson (1974) approach represented by Eq. (2) does not accurately represent the field conditions during failure. To substantiate this, the limit equilibrium condition represented by Eq. (2) is checked using the parameter values from Table 2 and a value for $\beta = 41°$. A value of 3.06 kN/m2 is obtained for the shear stress on the failure plane. The value of the undrained shear strength, c_u , is 1.4 kN/m2. Therefore, the limit equilibrium condition is not met for the Kewaunee slope using Equation (2).

Particulate Approach of Stability Analysis

A method of stability analysis developed by Vallejo (1979, 1980) which takes into consideration the particulate structure developed by thawing slopes like Kewaunee B, will be used to analyze their stability. The shear stress, τ , exerted on the plane of failure by a mixture of lumps of clay and fluid (Fig. 3) can be obtained from

$$\tau = [\gamma_{mix}] \, d \sin \beta$$
$$= [\gamma_f + (\gamma_s - \gamma_f) \, C] \, d \sin \beta \tag{4}$$

where γ_f is the unit weight of the fluid (water at the moment of thawing, mud after failure

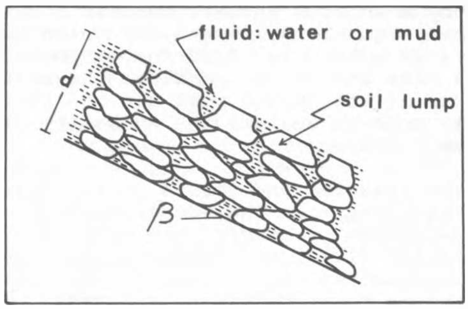

Figure 3. Particulate Structure

takes place), γ_s is the bulk unit weight of the lumps of clay, C is the ratio of the volume of the lumps of clay to the volume of the whole mass, γ_{mix} is the bulk unit weight of the thawed soil (mixture of clay lumps plus fluid) and the rest of the terms as defined before.

The shear strength, s , provided by the mixture (Fig. 3) on the plane of failure is

$$s = [(\gamma_s - \gamma_f) d \cos \beta \tan \phi'_r + c'_r] C \qquad (5)$$

When the critical depth of thaw normal to the slope face (d= 0.254 m.) is just reached, $\gamma_f = \gamma_w = 9.8\ kN/m^3$. Also, the value of $\gamma_{mix} = \gamma = 18.38\ kN/m^3$ (Table 2) and the value of $\gamma_s = 20.3\ kN/m^3$ (Table 1). Using these values and Eq. (4) a value of C = 0.82 is obtained. Using a value for β= 41° (Fig. 1) and the values for c'_r and ϕ'_r from Table 2, at limit equilibrium conditions Eq. (4) and (5) should give the same results. If the values of the corresponding parameters are reeplaced in Eq. (4) and (5) a value of 3.06 kN/m^2 is obtained for the shear stress on the plane of failure, and a value of 2.71 kN/m^2 is obtained for the shear strength. The value of the shear strength is therefore close to the value of the shear stress on the plane of failure using the particulate approach. However this was not the case when the Hutchinson (1974) approach was used.

After failure takes place, c'_r becomes equal to zero in Equation (5). Combining Equations (4) and (5) with $c'_r = 0$, the least slope angle of mobilization of the mixture represented by Fig. 3 is obtained as follows (Vallejo, 1979)

$$\tan \beta = \tan \phi'_r \left[\frac{(\gamma_s - \gamma_f) C}{\gamma_f + (\gamma_s - \gamma_f) C} \right] \qquad (6)$$

Using Equation (6) Vallejo (1979, 1980) successfully predicted the low angle of mobilization of a mudflow (mixture of hard clay fragments and mud) on the London Clay slopes as well as for mudslides (mixture of rock fragments and mud) at the location of Wellingborough and Isham, England described by Chandler (1970) independently of excess pore water pressures on the plane of failure.

CONCLUSIONS

On the basis of the study presented, the following observations can be made.

Under the right conditions, cohesive soils when frozen develop a particulate structure, that is blocks of soil sorrounded by a reticulate ice vein network. If the mixture forms part of a slope, a combination of soil lumps amd muddy water will slide when a critical depth of thaw is reached. Therefore, a stability analysis of the thawed soil mass must consider its particulate structure. The current study presents such an analysis with good results.

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