

Slope Instability Phenomenon in the Permafrost Region Along the Qinghai–Tibetan Highway, China

Hong Wang, Xianli Xing, Tonglu Li, Zhou Qin and Junhai Yang

Abstract The area of permafrost occupies $150 \times 10^4 \text{ km}^2$ on the Tibetan Plateau. Slope stability in permafrost regions is one of serious geological problems in the construction of the Qinghai–Tibetan Highway, which has been claimed to be an environment harmony project. Based on the field investigation, it is found that the main types of slope failure in the permafrost regions include slope collapse, slope creeping, surface vegetation creeping, debris flow and thaw-slumping. Among which, the thaw-slumping is the most hazardous one to the environment, vegetation and the engineering. The slumping may attribute to engineering excavation, which can disturb the thermal balance of the slope soil and cause thawing. As the slope soil thawing, the strength decreases in a large extent or even lost mostly, then the slope may fail in a very low slope angle. The countermeasures for the thermal thawing slides should maintain the frozen state of the soil for keeping from thawing, using methods such as covering with coarse material which can prevent heat absorption and be good for heat release.

Keywords Slope stability · Permafrost · Tibetan Plateau · Failure mechanism · Embankment

1 Introduction

For construction of traffic lines and oil pipelines in high latitude permafrost region, research on permafrost was firstly performed in the developed countries such as Canada and United States. In 1974, McRoberts and Morgenstern (1974) divided

H. Wang · X. Xing · T. Li (✉)

Department of Geological Engineering, Chang'an University, Xi'an, Shaanxi 710054, China
e-mail: dcdgx08@chd.edu.cn

Z. Qin · J. Yang

First Highway Consultants Co. Ltd, Xi'an, Shaanxi 710054, China

slope instability of permafrost into three types: debris flow, landslide and collapse. For stability assessment of permafrost slope, three methods have been put forward at present, namely, the effective stress method proposed by Weeks (1969); the total stress method by Hutchinson (1974) and the method based on the effective stress and melting-consolidation theory by McRoberts and Morgenstern (1974). The three methods all established the calculation model on the shallow plane infinite slope and Mohr-Coulomb criterion, although their mechanisms are different. Another important feature of permafrost slope is creep deformation. With trees-rings, Wu (1984) estimated that one of the permafrost slopes in Alaska has been creeping in the past 80 years. Through laboratory tests, McRobert proved that creep of frozen soil can occur at very low stress levels (McRoberts 1978), and a long-term creep potential would cause a large-scale deformation and instability of the slope.

The area of permafrost occupies $150 \times 10^4 \text{ km}^2$ on the Qinghai-Tibet Plateau (QTP). In the early time, human influence in this region is relatively weak. However, since the 1950s, with the sequential constructing of Qinghai-Tibet highway (QTH), Yunnan-Tibet highway (YTH), the Qinghai-Tibet railway (QTR), and South-to-North Water Transfer Project, disturbance of geological environment in the permafrost region inevitably lead to instability of the slopes. Slope stability is one of the key problems faced in the engineering construction. However, the research on slope stability of permafrost is still insufficient in China. The research is carried out combining with the construction of QTR and QTH. The permafrost on QTP, belongs to the plateau permafrost, is different from that on the other parts of the world which belongs to high altitude permafrost. Based on temperature environment, Lin et al. (2011) divided instable permafrost slope on QTP into the two types of frozen slope and thawing slope. The change in temperature will lead to deformation and instability of slopes, even though permafrost is general a solid material with higher strength. The frozen slope often has two types of failure modes. One is rock or soil falls, mainly occurs in rocky slopes with fractures and the soil slopes with rich ice, which may suffer from frequently frozen-heaving cycles. Since it is the most common failure type in natural and excavation slopes, passive or active protective measures have to be taken. Another type is creeping. The existence of ice in permafrost makes its creep character much more complex than that of normal soil and the creeping occurs even at very low stress levels, the creeping can happen both in steep-slope and moderate-slope, so it is widespread on QTP. Creeping changes with seasons, and the results of field test in the Fenghuo mountain region of QTP show that creep mainly occurs in winter and summer, creep strain decreases with the increase of the depth, but increases with ground temperature at the same depth (Wang and French 1995). The inverse displacement along the slope in winter implies the action of frost-heave. Failure of the frozen slopes generally occurs naturally and has less influence on the engineering. The thawing slope has three types of failure models: thaw slumping, mudflow terrace and vegetation creeping. In permafrost region, thaw slumping is widespread. Due to rise of air temperature, surface frozen soil thaws, soil with high content of ice changes into mixture of hard rock block and flowing slurry, which

has quite low shear strength or even lose the strength. Thaw slumping has a very shallow sliding plane which roughly paralleling to the slope surface, and could occur on very gentle slopes. In the permafrost region of QTP, thaw slumping is likely to happen on the slopes with the gradient of higher than 3° . The failure has a retrogressive extension to backward and sideward and the failed land surface is hard to recover. Mudflow terrace is a ladder-like landform caused by sliding of thawed soil. A unique case was found in a wide valley with gradient of about 10° in the Fenghuo mountain. There are ten steps with width of 5–10 m each and well vegetation cover. Mud flow terrace is thought to be formed as the thawed soil slides down on the slope surface, the vegetation underneath may check the movement and company drainage and consolidation. At this period, the soil sliding down behind overlays on the former sliding mass to form a step with gentle surface. The same process may occur in a valley to form number of steps. Vegetation creep is an especial landform. The vegetation on the slope surface slides down in variety size of pieces to form the scale-like landscape. It occurs as the slope gradient higher than 15° and has less impact on the engineering, usually results in vegetation degradation and soil erosion, as well as the ecological crisis.

From the above mentioned, it is clear that thaw slumping is the worst of all the slope failures to engineering, but it is also induced by engineering.

2 Engineering Induced Thaw Slumping and the Forming Conditions

With construction of roads and railways on QTP since last century, the engineering disturbances caused thawing of frozen soil, and consequently thaw slumping and other slope failure occurred. The Qinghai–Tibet Highway (QTH) starts from Xining city, Qinghai province in the North–East, terminates to Lhasa city, Tibet in the South–West, with the total length of 1,936 km. QTH is a second degree road and mostly runs aside QTR line. QTH was started to construct in 1950 and put onto working in 1954. It was improved for times in the history. The last portion from Gel mud to Lhasa with the length of 1,140 km mostly lies on the permafrost region of high altitude over 4,500 m.

The thaw slumping caused by construction of QTH mainly locate in the Hohxil hill region (K3014–3019), Beiluhe Basin (K3030–3060), and southern slope of the Fenghuoshan mountain (K3070–3080) (Fig. 1). The total length is about 45 km.

The thaw slumping near the milepost K3035 was caused by an accidental removal of soil during road reconstruction in the years between 1990 and 1992. Outcropped ground surface formed after earth borrowing. In summer, thawing of ground ice on the pit wall led to collapse of the upper part of the soil, which caused slumping of the soil for the first time. Afterward, it started to develop outward year by year and now formed a slumping with the length of 103 m East–Westward, the width of about 72 m North–Southward, the thickness of 1.5–2.0 m and the size of



Fig. 1 Distribution of the permafrost and the thaw slumping on QTP

about 10,000 m³ (Fig. 2). The slumping is located between Wudaoliang and the Tuotuo River, 40 km north of Fenghuo Mountain and at the altitude of 4,578 m. The slumping occurred on a gentle slope with the gradient of about 7°. We will take this case for further analysis on thaw slumping.

The area belongs to a dry-cold climate with thin air density and low air barometric. The four seasons here are not distinct and the annual freezing time lasts for seven to eight months. It freezes from September to next April, with the annual average temperature is -5.2 °C, extreme maximum temperature 23.2 °C and extreme minimum temperature -37.7 °C. It has the average annual rainfall of

Fig. 2 Thaw slumping near milepost K3035 of QTH



291 mm and the average evaporation of 1,317 mm. The annual mean ground temperature monitored is $-1.75\text{ }^{\circ}\text{C}$, so it belongs to permafrost area of low temperature.

Boreholes and shafts revealed that the formations here are the upper Neocene lacustrine sediment and Holocene alluvial deposits. The strata from the surface is composed of 0.00 ~ 1.10 m reddish-brown sand, wet, loose, and locally intercalated with reddish-brown silty clay ($w_p = 11.4\text{--}16.8\%$, $w_L = 18.9\text{--}26.9\%$); 1.10–1.30 m, gray loose silt and fine sand; 1.30–2.00 m, red silty clay ($w_p = 14.6\%$, $w_L = 25.5\%$), with 20 cm ice block inclusions at the base; 2.00–4.00 m, thick ice bed with smooth surface; 4.00 ~ 11.80 m, highly weathered reddish-brown mudstone freezing all year; 11.8–13.3 m, gray sandstone, hard, with tiny fracture ice locally; under 13.3 m, argillaceous sandstone. According to the ice content and structure characteristics, the frozen soil from top to base is divided into the following three layers: 0.0–2.0 m, seasonal thawing layer; 2.0–4.0 m, soil containing ice; under 4.0 m, massive cryostructures. The upper boundary of the permafrost is about 2.0 m in depth, that is the upper surface of the thick ice bed.

3 Monitoring of the Thawed Soil Thickness

Two thermometer boreholes, A and B, were drilled on the K3035 slumping. Hole A is located behind the head scarp of the slumping, in which 5 temperature probes were installed in the distance of 0.5 m each other; Hole B is located in the upper of the slumping mass, also 5 probes set as Hole A. Positions of monitoring boreholes in plane and profile were shown in Figs. 3 and 4. Monitoring data applied here are from 16 August 2002 to 2005, which lasts for three climatic years.

Hole A represents the natural slope outside the K3035 slumping. Ground temperature at depth of 0.5, 1.0, 1.5, 2.0 and 2.5 m in the monitoring period is shown in Fig. 5 and the frozen depth in Fig. 6 respectively. First, it can be seen that soil temperature changes annually and there is a freezing period and a thawing period each year. The thawing period at the depth of 0.5 m lasts from early April to the beginning of December, and the freezing period lasts from early December to early April next year. With increase of the depth, the thawing period is shortened and delays in some extent. Figure 6 shows that the maximum thawing depth monitored is 2.2 m, generally appears in the mid-September and approximately the same in the three thawing periods. So, the upper boundary of the permafrost is 2.2 m.

The ground temperature at the depth of 0.5, 1.0, 1.5, 2.0 and 2.5 m monitored in Hole B is shown in Fig. 7 and the frozen depth in Fig. 6. Similar to Hole A, the soil temperature changes annually and the thawing period at the depth of 0.5 m is almost the same as that of Hole A. The thawing period is shortened with the increase of depth and also delays clearly. It can be seen from Fig. 6 that maximum thawing depth is relatively shallow, which is 1.5 m, and also the same in the three

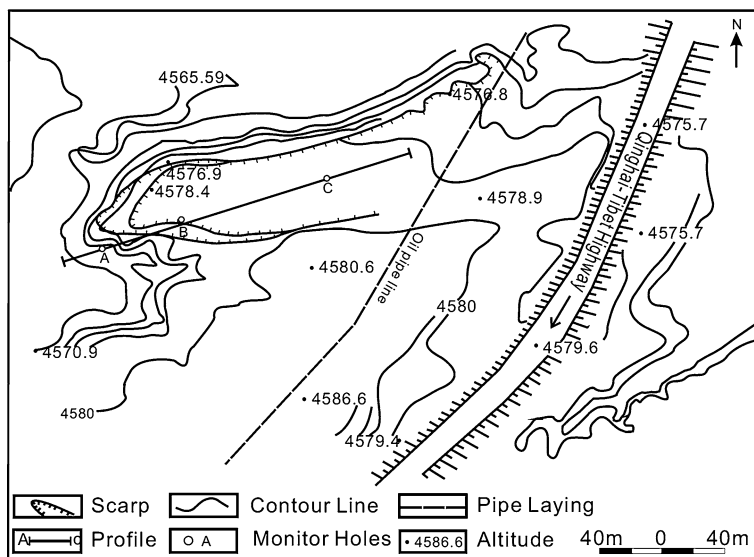


Fig. 3 Relief map and range of K3035 thaw slumping

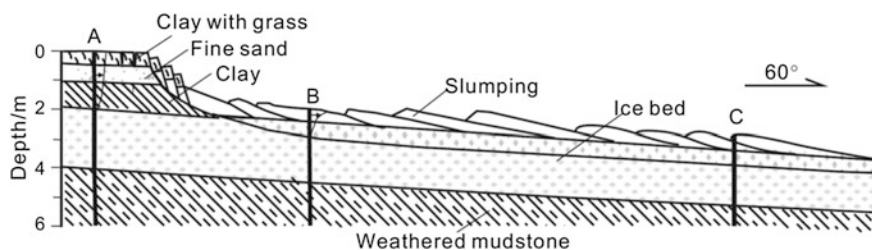


Fig. 4 Profile of the K3035 thaw slumping

is thawing periods. It shows that thaw slumping has changed the hydrothermal condition, and the annual maximum thawing depth is shallower than that in Hole A, so the upper boundary of the permafrost has redistributed in the slumping area.

In addition, Geokon-603 inclinometers were set in boreholes A, B and C. Of which, Hole C is located near the toe of the slumping mass as shown in Fig. 1. The monitoring started from 9 August 2002 to 16 April 2003, which had experienced a thawing period and a freezing one. Figure 8 shows the cumulative deformation of Holes A and C in the 250 days monitoring and that in Hole B in 15-day before 25 August, because the monitoring was stopped in Hole B at the date due to damage of the tube by extensive deformation. It can be seen that the maximum accumulative displacement at Hole A is 3.7 mm and that at Hole C is 10.9 mm. Even though Hole A is out of the boundary of the slumping, it still has slow displacement, but much smaller than that at Holes C, which indicates that the slumping

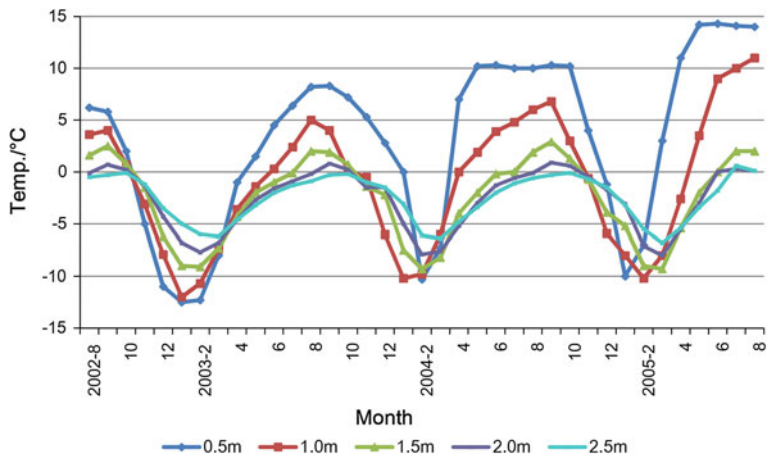


Fig. 5 Ground temperature changes at different depth versus time monitored in Hole A

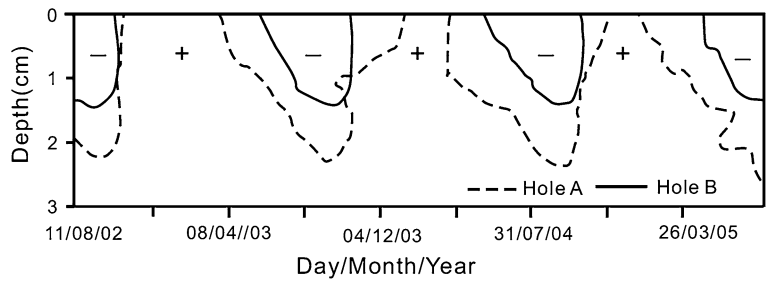


Fig. 6 The freezing depth monitored at Holes A and B

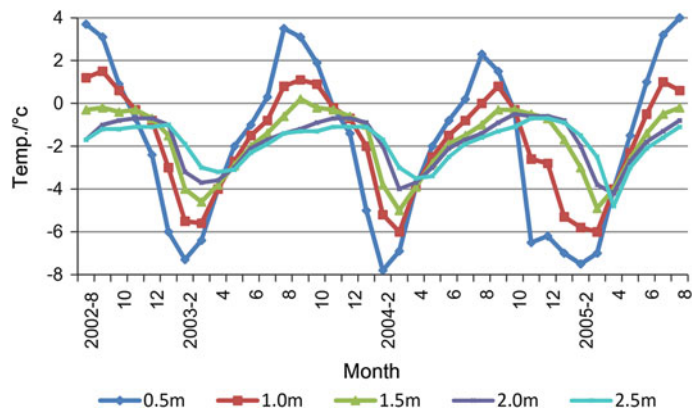
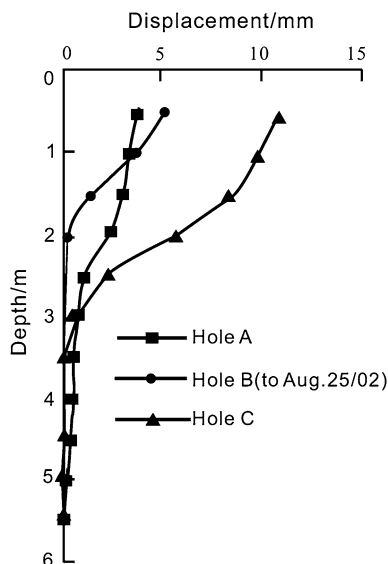


Fig. 7 Ground temperature changes at different depth versus time monitored in Hole B

Fig. 8 Displacements versus depth monitored in the Holes A, B and C (8 Aug 2002 to 16 Apr 2003)



tends to develop backward. The maximum accumulative displacement at Hole B reached to 5 mm in 15 days reflects that displacement at the back of the slumping mass is larger than that in the front. Reclined fold and imbricate structure on the slope surface can also prove the movement feature.

Examining the displacement vertically, it is found that the deformation character of thaw slumping is different from that of general landslide. General landslide moves as blocks and has an obvious sliding surface. While the thaw slumping deforms transitionally, it means that, displacement changes from maximum to diminished from the surface to the bottom, which should be associated with the thermal gradient under the surface. With decreasing of temperature, viscous force of the sliding mass in plastic or flow state is enhanced and the strength of solid frozen bed is much higher, the displacement would be restricted in degrees with depth. There is no deformation under 2 m out of the slumping reflecting by the curve of Hole A, and the thickness of thawing soil observed at that point is 2.2 m, they basically agrees with each other. Obviously the deformation extended deeper on the slumping mass as seeing the curves of Holes B and C, but mainly concentrated in the depth of 0–2 m and diminished under 4.5 m. The ground temperature in Hole B shows that the thickness of the thawing soil is only 1.5 m, but the deformation zone extends below 1.5 m. It indicates that there is deformation in the frozen soil lying underneath the thawing layer. The curves of ground temperature at the depth of 1.5 m and 2.0 m in holes A and B are shown in Fig. 9. It can be seen that although the thickness of thawing soil at hole B is decreased, the ground temperature is increased. The lowest temperature of them has the difference over 4 °C. It indicates that the frozen soil in high temperature creeps too, so the effect on sliding mass has extended to the soil under the thawing layer. On the

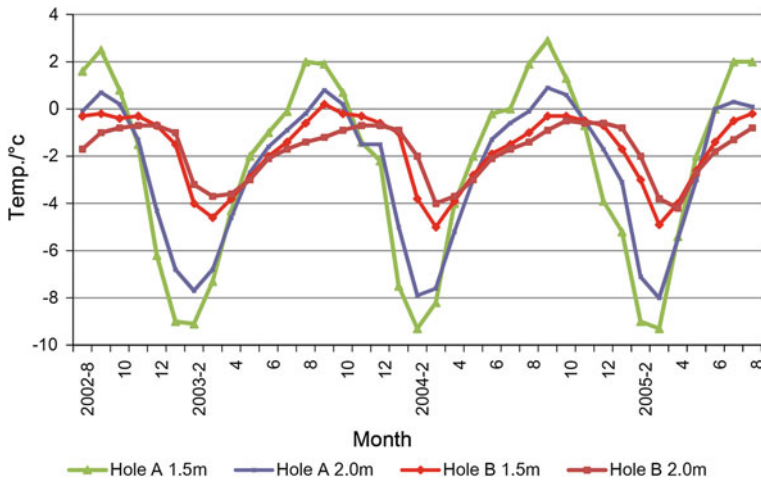


Fig. 9 Ground temperature versus month at depth of 1.5 and 2.0 m in Holes A and B

whole, deformation of the soil mainly occurred in the upper 2 m, above the surface of the thick ground ice, therefore, the thickness of thaw slumping is relatively less, but the influence area is large. It keeps on developing.

4 The Method for Thaw Slumping Stability Assessment

From the above analysis, thaw slumping has the features of thin thickness and broad area. The K3035 thaw slumping covers 7,200 m² of ground surface, but has the thickness of only 1.5–2.0 m, so the infinite slope model is suitable for its stability assessment as shown in Fig. 10. The model has the follow assumptions:

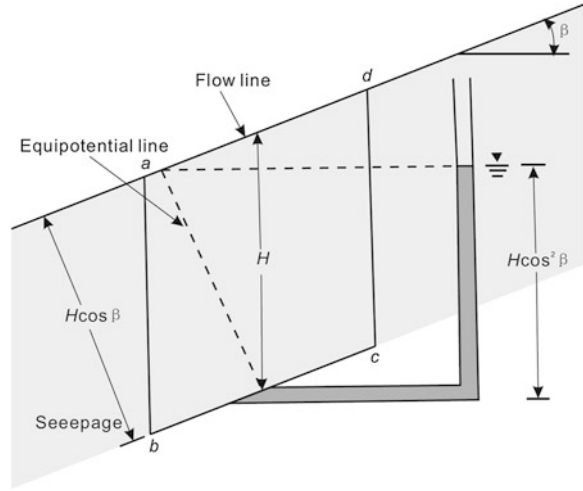
1. The sliding surface is a plane which parallels to the slope surface, so the thickness of the sliding mass is identical everywhere;
2. The thickness of the sliding mass is far more less than the length of the sliding mass;
3. The sides of the sliding mass extends infinitely, so the side effect is not considered;
4. The seepage flow direction is parallel downward to the sliding plane.

The shearing stress of the soil may be given by

$$\tau_f = c' + \sigma' \tan \phi' \quad (1)$$

To determine the factor of safety against failure along the plane *AB*, consider the slope element *abcd*. The forces that act on the vertical faces *ab* and *cd* are equal and opposite. The total weight of the slope element of unit length is

Fig. 10 Analysis model of infinite slope



$$W = \gamma_{sat} LH$$

The components of force acting on the base *bc* of the element *abcd* in the directions normal and parallel to plane *ab* are

$$N_r = \gamma_{sat} LH \cos \beta$$

$$T_r = \gamma_{sat} LH \sin \beta$$

The total normal stress and shear stress at the base of the element are:

$$\sigma = \frac{N_r}{L / \cos \beta} = \gamma_{sat} H \cos^2 \beta \quad (2)$$

$$\tau = \frac{T_r}{L / \cos \beta} = \gamma_{sat} LH \cos \beta \sin \beta \quad (3)$$

Generally, the factor of safety is defined as

$$F_s = \frac{\tau_f}{\tau} \quad (4)$$

where F_s is factor of safety with respect to strength; τ_f is average shear strength of the soil; τ is average shear stress developed along the potential failure surface. From Eqs. (1) and (4), we get

$$\tau = \frac{\tau_f}{F_s} = \frac{c'}{F_s} + (\sigma - u) \frac{tg \phi'}{F_s} \quad (5)$$

From Fig. 9, the pore water pressure can be deduced as

$$u = \gamma_w H \cos^2 \beta \quad (6)$$

Substituting σ in Eq. (2), τ in Eq. (3) and u in Eq. (6) into Eq. (5), we have

$$\gamma_{sat} L H \cos \beta \sin \beta = \frac{c'}{F_s} + (\gamma_{sat} H \cos^2 \beta - \gamma_w H \cos^2 \beta) \frac{tg \phi'}{F_s}$$

The factor of safety with respect to strength can be deduced from the above equation as

$$F_s = \frac{c'}{\gamma_{sat} H \cos^2 \beta tg \beta} + \frac{\gamma' tg \phi'}{\gamma_{sat} tg \beta} \quad (7)$$

where γ' is effective unit weight.

As motioned above, during the warm seasons, the depth of the thawed soil on the ground can reach to 1.5–2.0 m and the soil is completely saturated. In addition, the surface of the frozen ice under the soil could be the sliding surface.

By direct shearing CD tests for six groups of the remolded silty clay sampling from the sliding plane K3035 slumping, its friction angle ϕ' is about $14^\circ \sim 15^\circ$ under the water content of liquid limit, while the cohesion c' is nearly zero, the saturated unit weight γ_{sat} of the soil is measured as 1.80 g/cm^3 . As the slope gradient β is 7° , the safety factor F_s can be easily calculated from Eq. (7). It is between 0.90 and 0.97. Apparently, the sliding mass of the thaw-slumping is not stable in warm time. However, the stability is variable with seasonal thawing and frozen cycle, so it developed year by year.

5 The Forming Mechanism and the Mitigation Measure for Thaw-Slumping

Thaw-slumping is often initially triggered under disturbance of permafrost by engineering excavation, and then enlarges in the retrogressive process. A new cutting would damage the vegetation and disturb the thick ice bed underneath. The uncovered ground and the scarp may adsorb more solar radiation, then resulting in ice-rich permafrost thawing and part of the ice bed melting. Because the thick ice bed is impermeable, the thawed ice-rich permafrost could be extra-saturated and become liquid state, so it may flow down on the smooth ice surface. That is why the thaw-slumping may occur on a very gentle slope. The thaw slumping is commonly caused by slope toe disturbance. Once it is formed, slumping process would develop year by year, both laterally and backward.

From the experiences construction of QTH in decades, the conventional mitigation measures, such as retaining wall, stabilizing piles and frameworks are not effective for thaw-slumping control. The basic concept for mitigation of this special problem is to protect the permafrost, i.e., to reduce absorption and increase release of the solar radiation. Therefore, for those slopes already with thaw-

slumping have occurred, coarse grain material like sand or rock blocks are applied to cover the boundary of the slumping for keeping it in frozen state and block it from developing outward. The coarse material can obstruct the solar radiation to the permafrost, as well as release more heat out by the cold air flow through the porous. The rock blocks have been successfully used in the railway embankment to protect the permafrost in the foundation. For the slopes with slumping traces, it is important to fill and compact the fissures on the ground as early as possible, to make the ground recover to the original form and keep the surface sooth, as well as plant vegetation. For the slopes without slumping but within 500 m from the highway, it should be strictly prohibited to take soil material and vegetation should be planted to protect the slopes. Vegetation can absorb some solar energy and the roots can also stabilize the ground surface.

6 Conclusions

Construction of the QTH has caused some disturbance and damage to the permafrost environment. Thaw-slumping is one of the typical geological hazards induced by the engineering, which is characterized by the thin thickness, broad area, gentle sliding plane, saturated sliding mass, flow movement. Investigation for the K3035 thaw-slumping suggests that a little disturbance of the permafrost slope may induce a retrogressive slumping with long time activation, which may develop year by year. It is known that the QTP has a very sensitive and fragile environment because of the high elevations, insufficient oxygen, cold air and sparse vegetation. It is very difficult to recover the destructed environment. In engineering construction, protection of the environment is a severe challenge in the permafrost area.

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