Numerical Analysis of the Influence of Block-Stone Embankment Filling Height on the Water, Temperature, and Deformation Distributions of Subgrade in Permafrost Regions

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Abstract: The hydrologic and thermal states of foundation soils have an important influence on subgrade stability in degrading permafrost regions. However, thawing settlement remains a problem in the permafrost regions of Northeast China, because there are few relevant research results in this area, and the foundation deformation mechanism caused by the hydrologic and thermal changes of foundation soils is not clear. Therefore, a subgrade structure with large block stones as embankment filler was proposed to improve the foundation thawing settlement, and it has been successfully implemented in the road project (Yiershi–Chaiqiao section of 308 Provincial Highway) in Inner Mongolia Autonomous Region. To study the action mechanism of this structure on the subgrade deformation, a numerical model of the hydro–thermal–mechanical interaction process in unsaturated frozen soil was established, and the calculation of the water content, temperature, and deformation conditions of the subgrade within 20 years of the highway were carried out. The results show that the embankment block-stone layer can reduce the total heat of the deep foundation, and reduce the migration of unfrozen water from the deep foundation to the active layer. It can also increase the hydrologic and thermal convective flux inside and outside the block-stone layer, thus raising the permafrost table and reducing the subgrade deformation. The block-stone layer thickness has an important influence on the subgrade stability. By comparing the hydrologic, thermal change, and deformation of the block-stone structures with different thicknesses (5.0 m, 4.0 m, 3.5 m, 3.0 m, and 2.0 m), we found that when the thickness of the block-stone layer is 4.0 m, the subgrade stability is the best. In this case, the maximum uneven settlement and the maximum transverse deformation difference of the top surface of the subgrade are +0.521 cm and +0.462 cm, respectively. The subgrade stability is less optimal when the block-stone layer thickness is greater or less than 4.0 m. Thus, there exists an optimal embankment filling height related to the hydrologic and thermal conditions of the foundation soils. This study helps to elucidate the effect of unfrozen water content change on subgrade deformation during permafrost degradation and provides an important reference for solving the problem of thawing settlement of subgrade in permafrost regions.

Keywords: warm permafrost; subgrade stability; filling height of block-stone embankment; numerical simulation of the hydro–thermal–mechanical interaction process in unsaturated frozen soil

1. Introduction

Affected by global warming, road deterioration occurs frequently in permafrost regions [1–3]. Uneven settlement and longitudinal cracks have become the main failure forms of subgrade. Permafrost thawing caused by rising ground temperature will reduce the strength of the foundation soils and increase the vertical and transverse deformations,
which is also the main reason for subgrade instability [4,5]. In recent years, the retreat rate of the permafrost coast in the Arctic has accelerated significantly, and the warming rate of permafrost in northern Europe is higher than expected [6,7], which indicates that the permafrost in the northern hemisphere is rapidly deteriorating. The temperature rise rate in the southern margin of the permafrost regions in Eurasia is higher than the global average [8–10]. The rise in ground temperature reduces the thickness of the permafrost layer and then increases the thawing settlement of foundations [11]. According to damage investigation results along the Qinghai–Tibet Railway, the Zabaikalye Railway, and the East Siberian Railway in Russia [12,13], the uneven settlement caused by permafrost thawing is the main form of subgrade failure, and the soil strength damage caused by hydrologic and thermal changes is the main cause of subgrade deformation [14–16]. The temperature change of the permafrost foundation mainly affects the subgrade deformation by changing the amount of water migration and uneven water distribution. However, research on the action mechanism of a change in water conditions, especially the change in unfrozen water content on subgrade deformation, is still in the experimental stage [16,17]. One of the main reasons for this delay is that it is difficult to accurately measure the change in unfrozen water content in the soil in an open system during freeze–thaw cycles. This limits the research progress of laboratory tests to a great extent, resulting in the problem of permafrost subgrade deformation not being effectively solved [9,18]. Therefore, with permafrost warming, it is of great theoretical and practical value to explore the action mechanism of hydrologic and thermal changes, especially unfrozen water migration, on the deformation of permafrost subgrade and to establish a road structure system with a universal applicability in cold regions.

Soil strength damage caused by permafrost thawing is the main cause of subgrade deformation. The design concept of cooling the embankment or protecting permafrost has become the main way to reduce thawing settlement [3,19–21] and has been applied in a large number of subgrade projects in the Qinghai–Tibet Plateau, Alaska, and Siberia. Among them, the most representative engineering measures include crushed-rock embankment structures, crushed-rock revetment embankment structures, ventilation pipe embankment structures, and composite embankment structures [3,20,22–24]. However, these measures are effective in low-temperature permafrost regions [1,21,25], but not in warm permafrost regions [8,9,24,26], because they are used to cool the embankment by increasing the convective heat exchange of air inside and outside the embankment or reducing the solar radiation absorbed by the embankment [5,27]. These measures only consider the thermal stability of the embankment, not the impact of the water distribution on the foundation deformation. Thus, their applicability is greatly limited.

Currently, it is difficult to accurately measure the dynamic change in unfrozen water content in freezing soil [28]. Researchers have tried to improve various water measurement tools, but the results are not satisfactory [29,30]. Nuclear magnetic resonance is considered to be the most accurate tool for measuring unfrozen water content, but it cannot be directly used to measure unfrozen water content during one-way freezing or freeze–thaw cycles in soils. Therefore, researchers have proposed various numerical models to simulate and study the distributions of water content, temperature, and deformation in frozen soil, and the simulation results have been validated by laboratory tests. The mainstream models include the convective heat transfer model, hydro–thermal coupling model, and hydro–thermal–mechanical coupling model [3,31–36] (which considers pore water pressure as the driving force for water migration). These models are widely used in laboratory tests and simulation studies in engineering practice. However, these studies only consider the thermal and mechanical stability of the subgrade and do not consider the action mechanism of the change in unfrozen water content and uneven water distribution on foundation deformation due to temperature gradients and matrix potential gradients. Therefore, they do not solve the fundamental question of how water distribution affects foundation deformation.
Aiming at the problem of subgrade settlement caused by permafrost thawing, the K32 + 900 section of the highway from Yiershi to Chaiqiao on Provincial Road 308 in the Inner Mongolia Autonomous Region is used as the research object, and a subgrade structure with large block stones as embankment filler is proposed to improve the settlement caused by permafrost thawing. To analyze the subgrade stability, a numerical model of the hydro–thermal–mechanical interaction process in unsaturated frozen soil was established to simulate the changes in water content, temperature, and deformation within 20 years after the completion of the subgrade. In addition, to determine the most reasonable filling height of the block-stone layer, five structures with different filling thicknesses of the block-stone layer are set up in this study. Their stability is discussed in terms of the three aspects: variation in the permafrost table, distribution of unfrozen water content, and deformation. Finally, the stability of the vertical deformation and transverse deformation of the subgrade is compared and analyzed according to the two evaluation indexes of uneven settlement and transverse deformation difference. Finally, the most reasonable embankment filling height applicable to this section is derived.

2. Study Area

In this paper, the study area is located in the northwestern part of the Da Xinganling Mountains. The road area of the construction project from Yiershi to Chaiqiao on Provincial Road 308 in the Inner Mongolia Autonomous Region, China, with a geographical location of 47°15′59″ N, 119°48′46″ E to 47°28′22″ N, 120°36′59″ E, as shown in Figure 1. The climate of the study area is influenced by a combination of warm and humid marine airflow from the southeast and dry and cold airflow from the northwest, resulting in an alpine climate with a mean annual air temperature of approximately $-3.1 \degree C$ [37,38]. The mean annual rainfall is 410–470 mm, mostly concentrated from June to August. The snowfall in winter is 36.8 mm, accounting for 6.7% of the annual precipitation. The frost-free period is 70–120 days, and the annual average maximum freezing depth is 3.1 m. As this area is located at the southern edge of the Eurasian permafrost regions, the permafrost layer here varies in thickness from a few meters to tens of meters and is mainly developed in low-lying swampy wetlands and at the foot of mountain slopes [39].

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Figure 1. The permafrost distribution in Northeast China and the study area and project location in this paper, data from Jaroslav O. et al., 2019 [40].
The study section is an ordinary class 1 highway with asphalt concrete pavement, and the subgrade width is 22 m. In this paper, the K32 + 900 section of the S308 project is selected for numerical analysis.

3. Model Description

3.1. Geometric Model

Because the subgrade has linear characteristics along the length direction, the longitudinal influence is ignored. The geometric model of the subgrade structure is divided into four calculation units, as shown in Figure 2.

Figure 2. Geometric model and calculation unit division of the subgrade structure (Unit: m).

Note: the embankment refers to the fill layer above the natural ground surface (zone I), the foundation refers to the strata below the natural ground surface (zones II and III), and the subgrade includes the foundation and embankment.

We determine the size and calculation unit of the geometric model according to the following points: first, according to the subgrade cross-section construction drawing to determine the width of the top of the embankment, embankment height, slope rate, and other parameters; Second, we drilled five boreholes with a depth of 24 m at the foot of the slope, the shoulder, and the centerline of the road. Based on the borehole data, the parameters of the soil layers are determined. Zone I is a filled block-stone layer above the natural ground surface, with a height of 4.0 m. The width of the top of the embankment is 11 m, and the slope ratio is 1:1.5. Zone II is a clay layer below the natural ground surface, with a height of 4.0 m. The width of the top of the embankment is 11 m, and the slope ratio is 1:1.5. Zone III is a strongly weathered basalt layer, with a thickness of 8.8 m. Zone IV is a moderately weathered basalt layer. To reduce the influence of the boundary effect, the model is extended horizontally 30 m from the foot of the embankment slope to the right direction and vertically 30 m from the bottom of zone III vertically downward.

3.2. Mathematical Model

In unsaturated frozen soil, under the action of matric potential, temperature potential, and gravity potential, the water migration equation can be expressed as follows [41]:

\[
\frac{\partial \theta_w}{\partial t} + \frac{\rho_v}{\rho_w} \frac{\partial \theta_v}{\partial t} + \frac{\rho_i}{\rho_w} \frac{\partial \theta_i}{\partial t} = \nabla [K_{wh}(\nabla h + 1) + K_{vT} \nabla T + K_{vh} \nabla h + K_{vT} \nabla T] \tag{1}
\]
where $\theta_w$, (expressed as $\theta_w = a |T|^{b}$. $a$ and $b$ are the test parameters of unfrozen water content varying with temperature, and $T$ is temperature) $\theta_v$, and $\theta_i$ are the volumetric contents of unfrozen water, vapor, and ice, respectively. $\rho_w, \rho_v,$ and $\rho_i$ are their densities. $h$ is the height of the water head. $K_{wh}$ and $K_{wT}$ are the isothermal and non-isothermal hydraulic conductivities, respectively. $K_{vh}$ and $K_{vT}$ are the isothermal and non-isothermal vapor conductivities, respectively. The model considers two kinds of water phase transition processes, one is the ice–water phase transition, and the other is the water–vapor phase transition.

According to the van Genuchten model [42], the effective saturation $\Theta$ can be written as follows:

$$\Theta = \left[ 1 + \left( -\alpha h \right)^n \right]^{-m}$$

(2)

where $\alpha, n,$ and $m = 1 - 1/n$ are the model fitting parameters. Under isothermal conditions, the permeability coefficient of unsaturated soil can be expressed as follows [43]:

$$K_{wh} = 10^{-\Omega \Theta} K_s \left[ 1 - \left( 1 - \Theta^{-1/m} \right)^{m-1} \right]^2$$

(3)

where $\Omega$ is an empirical parameter with dimensions of 1, $l$ is a model adjustment parameter, taken as 0.5 [43], and $K_s$ is a saturated permeability coefficient.

The non-isothermal unfrozen water permeability coefficient $K_{wT}$, the vapor permeability coefficient $K_{vT}$, and the vapor permeability coefficient $K_{vh}$ can be described as follows [41]:

$$\begin{cases} 
K_{wT} = K_{wh} h G_{wT} (\frac{1}{\rho w}) \frac{dt}{dT} \\
K_{vh} = \frac{D}{\rho v} \rho v M v \frac{dT}{dT} \\
K_{vT} = \frac{D}{\rho v} \gamma H_r \frac{dq_v}{dT} 
\end{cases}$$

(4)

where $G_{wT}$ is the empirical parameter for evaluating the influence of temperature on the soil–water characteristic curve and $\gamma$ is the surface tension at 25 °C. $M$ and $g$ are the molar mass of water and gravitational acceleration, respectively. $R$ is the gas constant, $H_r$ is the relative humidity, $\eta$ is the water vapor diffusion enhancement factor, and $D$ is the water vapor diffusion.

The heat transfer equation of unsaturated soil can be written as follows [44]:

$$C \frac{dT}{dt} - L_v \rho_v \frac{d\theta_v}{dt} + L_i \rho_i \frac{d\theta_i}{dt} = \nabla (\lambda \nabla T) - C_w [\nabla (q_w T)] - C_v [\nabla (q_v T)] - L_v \rho_v \nabla q_v$$

(5)

where $C$ is the equivalent specific heat, $\lambda$ is the equivalent thermal conductivity, and $L_v$ is the latent heat of water evaporation. $C_w$ and $C_v$ are the specific heat capacity of liquid water and vapor, respectively. The expressions of other hydrologic and thermal parameters are shown in Table 1.

| Parameter | Expression |
|-----------|------------|
| Surface tension | $\gamma = 75.6 - 0.1425 T - 2.38 \times 10^{-4} T^2$ |
| Saturated vapor density | $\rho_v = \exp(31.37 - 6014.79/T - 7.92 \times 10^{-3} T) \times 10^{-3}/T$ |
| Water vapor diffusion enhancement factor | $\eta = 9.5 + 30 \theta_w \theta_s^{-1} - 8.5 \exp(1 + 2.6 (f_c) 0.5 \theta_w \theta_s^{-1})^4$ |
| Relative humidity | $H_r = \exp(0.15Mg/RT)$ |
| Water head | $h = L_i(T - T_i)/g T_i$ |
| Equivalent specific heat of the soil | $C = C_w \theta_w + C_v \theta_v + C_i \theta_i + C_v \theta_v$ |
| Latent heat of vapor evaporation | $L_v = 2.501 \times 10^9 - 2369.2 T$ |
| Equivalent thermal conductivity | $\lambda = \lambda_w^0 \lambda_w^0 \lambda_v^0 \lambda_v^0$ |
Based on viscoplastic theory, the stress–strain relationship of frozen soil can be expressed as follows [15,36]:

$$\{\Delta \sigma\} = [D_T](\{\Delta \varepsilon\} - \{\Delta \varepsilon_{vp}\} - \{\Delta \varepsilon_v\}) \quad (6)$$

$$[D_T] = \frac{E_T(1 - \nu_T)}{(1 + \nu_T)(1 - 2\nu_T)} = \begin{bmatrix} \frac{1}{1+\nu_T} & \frac{\nu_T}{1+\nu_T} & 0 \\ \frac{\nu_T}{1+\nu_T} & 1 & 0 \\ 0 & 0 & \frac{1-2\nu_T}{2(1-\nu_T)} \end{bmatrix} \quad (7)$$

where $[D_T]$ is the elasticity matrix related to the temperature and water content, $E_T$ is the elastic modulus and $\nu_T$ is Poisson’s ratio. $\{\Delta \varepsilon\}$, $\{\Delta \varepsilon_{vp}\}$, and $\{\Delta \varepsilon_v\}$ are the total strain increment vector, the viscoplastic strain increment vector, and the frost heave strain increment vector, respectively.

In the two-dimensional stress state, the viscoplastic strain rate can be described as follows [5]:

$$\varepsilon_{vp} = \gamma_T \langle \Phi(F) \rangle \frac{\partial Q}{\partial \{\sigma\}} \quad (8)$$

where $\gamma_T$ is the viscosity parameter, $Q$ is the plastic potential function, and the scalar function $\Phi(F)$ can be expressed as

$$\Phi(F) = \frac{F - F_0}{F_0} \quad (9)$$

and

$$\langle \Phi(F) \rangle = \begin{cases} \Phi(F) & \Phi(F) > 0 \\ 0 & \Phi(F) \leq 0 \end{cases} \quad (10)$$

where $F$ is the yield function and $F_0$ is the yield stress.

The volumetric strain caused by the ice–water phase change can be expressed as follows [14]:

$$\varepsilon^v = \delta[(1 + \zeta)(\theta_0 + \Delta \theta_L - \theta_w) - (n - \theta_w)] \quad (11)$$

where $\zeta$ is the in situ frost heave rate, $n$ is the porosity of the soil, $\theta_0$ is the initial volumetric water content, $\Delta \theta_L$ is the volumetric content of the migrated unfrozen water, and $\delta$ is defined as follows [5]:

$$\delta = \begin{cases} 1 & (1 + \zeta)(\theta_0 + \Delta \theta_L - \theta_w) - (n - \theta_w) > 0 \\ 0 & (1 + \zeta)(\theta_0 + \Delta \theta_L - \theta_w) - (n - \theta_w) \leq 0 \end{cases} \quad (12)$$

Then, $\{\varepsilon_v\}$ can be defined as follows [36]:

$$\{\varepsilon_v\} = \varepsilon^v \begin{bmatrix} \frac{1}{3} & 1 + \nu_T & 1 + \nu_T & 0 \end{bmatrix} \quad (13)$$

Equations (1)–(13) compose a hydro–thermal–mechanical coupling model for unsaturated frozen soil.

### 3.3. Parameters of the Soil Layers

According to the engineering data of the K32 + 900 section and the field test and laboratory test results, the physical parameters of the soil layers are shown in Table 2 [3,22,35,45–47], and the mechanical parameters are shown in Table 3 [5,15,36,48–53]. The block stones ranging from 20 to 40 cm in diameter are granite with good mechanical properties, and the mean particle size is 28.5 cm. Since the change in the material of the embankment over time is not considered in this study, it is suggested to strengthen the protection of the slope foot of subgrade during construction to reduce the influence of silt particles generated by rock weathering and external water flow on subgrade stability.
Table 2. Physical parameters of the soil layers.

| Soil Layer | $\lambda_s$ (W/m·K) | $C_s$ (J/kg·K) | $K_s$ (m/s) | $\rho$ (kg/m$^3$) | $\theta_0$ (%) | $n_0$ | $a$ | $b$ | $\alpha$ (m$^{-1}$) | $n$ |
|------------|---------------------|---------------|------------|------------------|---------------|------|-----|----|----------------|----|
| I          | 2.642               | 923.0         | $8.404 \times 10^{-4}$ | 2700             | 10.2          | 0.350 | 0.067 | -0.21 | 2.85            | 2.42 |
| II         | 1.363               | 1125.4        | $2.617 \times 10^{-6}$ | 1870             | 29.1          | 0.486 | 0.142 | -0.23 | 0.80            | 1.09 |
| III        | 1.917               | 1013.2        | $8.844 \times 10^{-6}$ | 2048             | 31.5          | 0.511 | 0.174 | -0.27 | 0.54            | 1.36 |
| IV         | 2.055               | 982.7         | $7.7648 \times 10^{-6}$ | 2156             | 19.8          | 0.502 | 0.091 | -0.13 | 0.56            | 1.41 |

Table 3. Mechanical parameters of the soil layers.

| Soil Layer | $a_1$ (MPa) | $b_1$ | $a_2$ | $b_2$ | $a_3$ ($^\circ$) | $b_3$ | $a_4$ (MPa) | $b_4$ | $c_1$ (MPa) | $d_1$ (MPa) | $c_2$ | $d_2$ ($^\circ$) | $e_1$ (MPa) | $c_3$ ($^\circ$) | $d_3$ (MPa) | $c_4$ | $d_3$ (MPa) | $c_5$ (MPa) | $d_3$ (MPa) | $e_3$ (MPa) |
|------------|-------------|------|------|------|------------------|------|------------|------|-------------|------------|------|------------------|-------------|------------------|------------|------|-------------|------------|------------|-------------|
| I          | 86          | 24   | 0.30 | -0.001 | 30               | 0.86 | 0.005      | -16  | -0.2        | 86         | 1.1  | -0.4            | 30          | 1.1              | -0.001    | -0.001 | 0           |
| II         | 3.21        | 1.49 | 0.40 | -0.007 | 8.8              | 0.52 | 0.038      | -12  | -0.3        | 3.21       | 1.2  | -0.6            | 30.7        | 1.2              | 8.8       | 1.1   | -0.041      | -0.013     | 0.038      |
| III        | 30.7        | 12.54| 0.37 | -0.006 | 22               | 0.34 | 0.031      | 0.058 | -11         | -0.6       | 30.7 | 1.2            | 9           | -7              | 22         | 1.2   | -0.036      | -0.039     | 0.031      |
| IV         | 64.2        | 24.71| 0.34 | -0.004 | 26               | 0.66 | 0.084      | 0.068 | -19         | -2.6       | 64.2 | 1.2            | -3          | 26              | 1.2        | -0.072 | -0.044      | 0.084      | 0.038      |
Other mechanical parameters of the soil layers are described as follows [54]:

\[ E_T = \begin{cases} a_1 + b_1 |T|^{0.6} & T \leq T_f \\ c_1 \theta_w^2 + d_1 \theta_w + e_1 & T > T_f \end{cases} \] (14)

\[ v_T = \begin{cases} a_2 + b_2 |T| & T \leq T_f \\ a_2 & T > T_f \end{cases} \] (15)

\[ \varphi_T = \begin{cases} a_3 + b_3 |T|^{c_3} & T \leq T_f \\ c_3 \theta_w^2 + d_3 \theta_w + e_2 & T > T_f \end{cases} \] (16)

\[ c_T = \begin{cases} a_4 + b_4 |T|^{c_4} & T \leq T_f \\ c_5 \theta_w^2 + d_5 \theta_w + e_3 & T > T_f \end{cases} \] (17)

3.4. Boundary Conditions

Based on monitoring data, the temperature boundaries for the embankment and the natural ground surface are defined as follows:

\[ T = T_0 + A \sin(2\pi t + \alpha_0) + \Delta T t \] (18)

where \(\alpha_0\) is the initial phase angle, determined by the completion time of the embankment. \(\Delta T\) is the climate warming rate, taken as 0.052 °C/a [10]. The mean annual ground surface temperature \(T_0\) and annual temperature amplitude \(A\) for each boundary are shown in Table 4. The side ABCDE is a symmetrical boundary, the side FGHI is an adiabatic boundary, and the geothermal flux at the bottom boundary EF is 0.037 W/m² [39]. The boundary IJK is water permeable, the AK and FGHI boundaries are impermeable [15,44,55], and the bottom boundary EF has a 19.80% liquid water supply. The horizontal displacement for the lateral boundary FGHI and the vertical displacement of the bottom boundary EF is restrained. The top surfaces of the embankment AK, the side slope JK, and the natural ground surface IJ are free boundaries without restraint.

| Boundary                        | \(T_0\) (°C) | \(A\) (°C) |
|---------------------------------|--------------|------------|
| Natural ground surface: IJ      | −0.35        | 16.21      |
| Side slope surface: JK          | 0.31         | 19.74      |
| The top surface of the embankment: AK | 1.10         | 20.40      |

3.5. Initial Conditions and Model Verification

Assuming that the hydrologic and thermal conditions of the foundation before embankment construction are stable, the hydrologic and thermal fields of the foundation soils are calculated without considering climate warming. When the ground temperature below the depth of the annual change tends to be stable, the hydrologic and thermal conditions of the foundation can be used as the initial conditions for the numerical simulation. The calculation results in Figure 3 show that the measured and calculated values of the borehole ground temperature match well. It should be noted that the S308 road has not yet been completed (until April 2022), but the subgrade project of the K32 + 900 section studied in this paper was completed on 15 October 2021. Since the thermal disturbance generated by subgrade engineering changes the hydrologic and thermal states of the original foundation, the hydrologic and thermal conditions of the foundation on 15 October 2021 are used as the initial conditions for the numerical calculation, and the water content, temperature, and deformation will be calculated for 20 years after the embankment is completed in a warming climate.
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Assuming that the hydrologic and thermal conditions of the foundation before embankment construction are stable, the hydrologic and thermal fields of the foundation soils are calculated without considering climate warming. When the ground temperature below the depth of the annual change tends to be stable, the hydrologic and thermal conditions of the foundation can be used as the initial conditions for the numerical simulation. The calculation results in Figure 3 show that the measured and calculated values of the borehole ground temperature match well. It should be noted that the S308 road has not yet been completed (until April 2022), but the subgrade project of the K32 + 900 section studied in this paper was completed on 15 October 2021. Since the thermal disturbance generated by subgrade engineering changes the hydrologic and thermal states of the original foundation, the hydrologic and thermal conditions of the foundation on 15 October 2021 are used as the initial conditions for the numerical calculation, and the water content, temperature, and deformation will be calculated for 20 years after the embankment is completed in a warming climate.

Figure 3. Comparison of the measured and simulated ground temperatures of the borehole at the foot of the slope from 15 January 2021, to 15 October 2021.

4. Results

4.1. Laboratory Test Verification

A unidirectional freezing test of unsaturated clay [47] is used to verify the numerical model. The height of the soil column is 12 cm, and the initial water content is 0.267. The top and bottom temperatures are $-2^\circ C$ and $1^\circ C$, respectively. During the experiment, adiabatic treatment is carried out around the specimen, and there is a water supply at the bottom. Three parallel soil samples are used for testing, and the total volumetric water content is measured by slicing after freezing for 5 h, 10 h, and 40 h. The results show that (Figure 4) the calculated and simulated values match well, which verifies the accuracy of the model.
4. Results

4.1. Laboratory Test Verification

A unidirectional freezing test of unsaturated clay [47] is used to verify the numerical content is measured by slicing after freezing for 5 h, 10 h, and 40 h. The results show that there is a deviation between the measured values and the simulated values. This is because there is an ice lens at this height, which makes it difficult to slice the soil samples. Therefore, there may be a deviation in the measured values.

Figure 4. Comparison of the measured and simulated values of unfrozen water content, temperature, and frost heave in one-side freezing tests: (a) the total volumetric water content at different moments, (b) the temperature at different moments, and (c) the variations in the frost heave.

It should be noted that the test temperature is slightly higher than the simulation temperature for t = 5 h (Figure 4b). This is because when the cold bath temperature is adjusted from 1 to −2 °C, the temperature at the top of the soil sample cannot reach −2 °C instantly, resulting in the cooling rate of the soil being slightly lower than the theoretical value at the beginning of the experiment. With the increase in freezing time and freezing depth, the migration rate of the freezing front gradually decreases, resulting in the decrease in this deviation. Therefore, the simulated and measured temperatures match well for t = 10 h and 40 h. In Figure 4a, when the measured volumetric water content is high (>0.4), there is a deviation between the measured values and the simulated values. This is because there is an ice lens at this height, which makes it difficult to slice the soil samples. Therefore, there may be a deviation in the measured values.

4.2. Ground Temperature Variation

The maximum seasonal thawing depth in the study area is reached in October each year when the permafrost table under the subgrade is the deepest. Figure 5(a1–a4) show the temperature distributions on 15 October of different years after the completion of the subgrade. Due to the influence of climate warming and engineering thermal disturbance, the permafrost tables decrease from −2.175 to −6.302 m from 15 October 2021 to 15 October 2025, and the average permafrost degradation rate is 1.032 m/a. From 15 October 2025 to 15 October 2041, the permafrost tables will rise from −6.302 to 0.109 m, and the permafrost that had previously thawed refreezes again. This phenomenon is related to the water migration process in different seasons and the heat transfer mechanism of the block-stone layer.

In warm seasons, a block-stone layer with good thermal insulation weakens the heat transfer efficiency of solar radiation in the embankment, thus reducing the total heat entering into the deep foundation [3,24]. In addition, compared with that of the ordinary fill layer, the temperature gradient inside and outside the block-stone layer is greater, and the pore connectivity is better. It is more conducive to the water and heat in the block-stone layer discharged from the embankment by convection, thus further reducing the subgrade temperature. In cold seasons, the poor water-holding capacity of the block-stone layer leads to the inability to form a continuous ice lens within it. Therefore, water (especially gaseous water) inside the thawing interlayer will be discharged from the embankment by convection [24,46], which cools the foundation.
In warm seasons, a block-stone layer with good thermal insulation weakens the heat transfer efficiency of solar radiation in the embankment, thus reducing the total heat entering into the deep foundation [3, 24]. In addition, compared with that of the ordinary fill layer, the temperature gradient inside and outside the block-stone layer is greater, and the pore connectivity is better. It is more conducive to the water and heat in the block-stone layer discharged from the embankment by convection, thus further reducing the subgrade temperature. In cold seasons, the poor water-holding capacity of the block-stone layer leads to the inability to form a continuous ice lens within it. Therefore, water (especially gaseous water) inside the thawing interlayer will be discharged from the embankment by convection [24, 46], which cools the foundation.

The maximum seasonal freezing depth in the study area occurs in March each year. Figure 5(b1–b4) show the temperature distributions on 15 March in different years. From 15 March 2022 to 15 March 2026, the cross-sectional area of the thawing interlayer decreases from 80.128 to 64.166 m². After 15 March 2031, the thawing interlayer disappears and the active layer is completely located in the block-stone layer. Because the block-stone layer can resist the deformation caused by freeze–thaw, the structure can effectively reduce the thawing settlement of the subgrade.
4.3. Change in the Vertical Distribution of Unfrozen Water Content under the Road Centerline

After the thawing of permafrost, the increase in unfrozen water content in the subgrade leads to settlement, which is a major cause of subgrade failure [5]. Under the action of the temperature gradient and matrix potential gradient, the uneven settlement caused by unfrozen water migration is another major cause of subgrade failure [16].

Figure 6 shows that the unfrozen water content of the filling layer (zone I) is smaller than that of other soil layers. The block-stone layer can not only reduce the unfrozen water content transferred from the deep foundation to the active layer, but can also increase the water convection flux inside and outside the block-stone layer, thus reducing the subgrade temperature. There are two reasons for this phenomenon. One is that the poor water holding capacity of the block-stone layer reduces its matrix potential [56], thus reducing the driving force of water migration within it. The decrease in the water migration driving force reduces the water content migrating from the deep foundation to zone I, which eventually leads to the reduction in the unfrozen water content in zone I. Second, the pore connectivity of the block-stone layer is good [44], and the internal and external temperature gradients are large. So, the water in zone I is more effectively discharged from the embankment by convection, thus further reducing the unfrozen water content in zone I. In addition, the unfrozen water content in zone I continues to decrease over time because the water in the thawing interlayer gradually freezes, which reduces the water content transferred from the deep foundation to the active layer. Notably, the reduction in water content in zone I can effectively reduce the subgrade settlement and, thus, improve its mechanical stability [4].

![Figure 6](image_url)

**Figure 6.** Vertical distributions of volumetric unfrozen water content under the road centerlines in warm and cold seasons in different years: (a) vertical distributions of volumetric unfrozen water content in warm seasons and (b) vertical distributions of volumetric unfrozen water content in cold seasons.

From 15 October 2021 to 15 October 2025, the permafrost table continues to fall, increasing unfrozen water content near the freeze–thaw interface (at a depth of −6.302 m). This is because the temperature gradient and matrix potential gradient gradually decrease with increasing depth, which makes it difficult for most of the unfrozen water in the deep foundation to migrate to the ground surface. From 15 October 2025 to 15 October 2041, the rise in the permafrost table results in a decrease in the unfrozen water content near the freeze–thaw interface. Notably, at a depth of −3 m, the water continues to migrate to the active layer and discharges out of the subgrade, resulting in the unfrozen water content decreasing over time.
4.4. Deformation Distributions

In permafrost regions, the wider the subgrade is, the greater the settlement [57]. The heat absorption effect of asphalt pavement not only increases the degradation rate of permafrost, but also aggravates the uneven settlement.

The permafrost table in the study area is the lowest in October every year when the subgrade settlement is the largest. Figure 7(a1–a4) show the vertical deformation distributions of the subgrade on 15 October in different years. From 15 October 2021 to 15 October 2025, the continuous decrease in the permafrost table leads to increasing settlement. The maximum settlement reaches $-0.835$ cm (the positive and negative signs in this paper only represent the deformation direction), which is located at 7.156 m on the right side of the road centerline. From 15 October 2025 to 15 October 2040, the freezing of unfrozen water within the thawing interlayer reduces the subgrade settlement. By 15 October 2040, the vertical deformation of the top surface of the subgrade is positive, and the maximum vertical deformation is $+0.951$ cm, which is located at the road centerline. In this process, frost heave is greater than settlement. There are two reasons for this phenomenon: first, the thawing settlement lags behind permafrost thawing [4], and the short thawing time of the permafrost causes most of the meltwater under the embankment to drain too late. This results in less settlement from 15 October 2021 to 15 October 2025. Second, during the meltwater refreeze process, a large amount of water migrates toward the freezing front and forms an ice lens [4,27], resulting in a larger frozen heave.

Under the gravity of the embankment filling layer, uplift deformation will occur at the slope toe. The increase in foundation settlement will also squeeze the soil near the slope toe, resulting in a further increase in uplift deformation. Figure 7(a1–a4) show that the maximum uplift deformation is $+1.029$ cm from 15 October 2021 to 15 October 2031, with an average rate of increase of $0.103$ cm/a. The maximum uplift deformation is $+1.038$ cm from 15 October 2031 to 15 October 2040, with an average rate of increase of $0.001$ cm/a. The location of the larger uplift deformation is mainly located near the slope toe, and this local deformation has less influence on the subgrade stability. Therefore, this paper does not conduct an in-depth analysis.

The uneven settlement will produce transverse deformation, and too much transverse deformation will produce longitudinal cracks, thus destroying the subgrade stability. Figure 7(b1–b4) show the transverse deformation distributions of the subgrade on 15 October in different years. The transverse deformation increases from 0 to $+1.058$ cm from 15 October 2021 to 15 October 2025, with an average rate of increase of $0.265$ cm/a. However, the maximum deformation drops to $+0.926$ cm on 15 October 2040, with an average rate of increase of $-0.008$ cm/a. The variation patterns of transverse deformation and settlement are the same, both of which are related to water refreezing within the thawing interlayer. It is noted that the distributions of both vertical deformation and transverse deformation near 7.156 m on the right side of the road centerline show a difference, mainly in the form of uneven settlement and transverse deformation in the opposite direction. It can be inferred that longitudinal cracks may occur at this location.
5. Discussion

The block-stone layer not only raises the permafrost table, but also reduces the water migration from the thawing interlayer, which is the main reason why the structure can reduce settlement. Related research shows that [57] when the embankment width is small, the uneven settlement at the road centerline and shoulder decreases with increasing embankment height. When the embankment width is large, the opposite trend is observed. The larger the embankment width is, the easier it produces longitudinal cracks. The embankment width of this section reaches 22 m, which is a wide embankment. So, the most reasonable filling thickness of the block-stone layer needs to be discussed.

Five different subgrade structures with block-stone layer thicknesses of 5 m, 4 m, 3.5 m, 3 m, and 2 m are set up to discuss the stability of these structures from several aspects: the variation in the permafrost table, the distribution of unfrozen water content, the uneven settlement, and the transverse deformation difference.

5.1. Influence of the Change in Permafrost Table on the Thermal Stability of the Subgrade

The settlement caused by permafrost thawing is the main cause of subgrade failure, and the subgrade stability can be improved by increasing the permafrost table [3,25,26,35,58].
There are two main reasons why the block-stone layer can raise the permafrost table. First, the block-stone layer has a large number of interconnected pores, and the water and heat inside it can be quickly discharged from the embankment by convection, thus reducing the embankment temperature. Second, a larger porosity and smaller unfrozen water content of the block-stone layer reduce its thermal conductivity, which reduces the total heat entering the embankment through the top surface of the embankment. The combination of these two factors ultimately raises the permafrost table. The increase in the thickness of the block-stone layer can increase the permafrost table, but the increase in the height of the side slope will also increase the heat absorption of the embankment, thus decreasing the rise of the permafrost table [21]. Therefore, when the permafrost table under the road centerline is higher than the natural ground surface, the contribution of the increase in the thickness of the block-stone layer to the thermal stability of the subgrade will be reduced.

From 2021 to 2025, the permafrost tables under the road centerlines of the five structures fall rapidly (Figure 8), but the change rates do not differ significantly. From 2025 to 2041, the permafrost tables under the road centerlines of the three structures with block-stone layer thicknesses of 5 m, 4 m, and 3.5 m are raised to 0.226 m, 0.109 m, and 0.106 m, respectively. At this time, the active layer is located within the block-stone layer, and there is no thawing interlayer in the foundation. The thermal stability of these three structures is good because the deformation of the block-stone layer is minimally affected by seasonal freezing and thawing. However, the permafrost tables under the road centerlines of the two structures with block-stone layer thicknesses of 3 m and 2 m drop to −6.064 m and −8.461 m, respectively. Permafrost degradation increases the settlement and, thus, destabilizes the subgrade. Therefore, the thickness of the block-stone layer should not be less than 3.5 m from the perspective of thermal stability.

5.2. Influence of Unfrozen Water Content Variation on Subgrade Deformation

Comparing the vertical distributions of volumetric unfrozen water content under the road centerlines of the five structures (Figure 9), it can be seen that the smaller the embankment height is, the greater the unfrozen water content in the block-stone layer. This is because the increase in embankment height increases the water convection flux of the slope, thus reducing the unfrozen water content in the block-stone layer. The change in unfrozen water content in the block-stone layer has less effect on embankment deformation because it can resist the uneven deformation caused by the water phase change.
at this location (some of the unfrozen water will migrate around the structures, thus producing local uneven deformation, which is the reason for the large uneven settlement and transverse deformation of these two structures), thus increasing the subgrade settlement. At a depth of $-6$ m, the complete thawing of the permafrost results in large unfrozen water content at this location for both structures. As time increases, this part of unfrozen water continuously migrates to the active layer, eventually increasing the settlement at the top of the subgrade.

Figure 9. Vertical distributions of volumetric unfrozen water content under the road centerlines of different structures: (a1–a4) vertical distributions of volumetric unfrozen water content in warm seasons and (b1–b4) vertical distributions of volumetric unfrozen water content in cold seasons.
The unfrozen water content is small at a depth of −3 m under the centerlines of the two structures with block-stone layer thicknesses of 3 m and 2 m. This indicates that a large amount of unfrozen water migrates upward and discharges from the embankment at this location (some of the unfrozen water will migrate around the structures, thus producing local uneven deformation, which is the reason for the large uneven settlement and transverse deformation of these two structures), thus increasing the subgrade settlement. At a depth of −6 m, the complete thawing of the permafrost results in large unfrozen water content at this location for both structures. As time increases, this part of unfrozen water continuously migrates to the active layer, eventually increasing the settlement at the top of the subgrade.

5.3. Deformation Characteristics of the Road Centerline and Shoulder and Stability Evaluation of the Subgrade

The deformation characteristics of the embankment top can reflect the overall stability of the subgrade. In warm permafrost regions, the main deformation affecting the subgrade stability is the settlement.

Comparing the vertical deformation curves at the road centerlines and the shoulders of the three structures with block-stone layer thicknesses of 5 m, 4 m, and 3.5 m (Figure 10), it can be seen that from 2021 to 2025, the cumulative settlements at the road centerlines are −0.989 cm, −0.572 cm, and −0.772 cm, and the deformations at the shoulders are −0.263 cm, −0.383 cm, and −0.459 cm, respectively. From 2025 to 2041, the maximum vertical deformations at the road centerlines are +0.178 cm, +0.951 cm, and +0.534 cm (including frost heave), and the deformations at the shoulders are +0.583 cm, +0.909 cm, and +0.852 cm, respectively. Comparing the vertical deformation curves at the road centerlines and the shoulders of the two structures with block-stone layer thicknesses of 3 m and 2 m from 2021 to 2041 (Figure 10), it can be seen that the cumulative settlements at the road centerlines are −8.095 cm and −11.140 cm and that the cumulative settlements at the shoulders are −4.900 cm and −7.591 cm, respectively. This shows that if the thickness of the block-stone layer is at least 3.5 m, the cumulative settlement of the subgrade top is smaller. If the thickness is less than 3.5 m, the cumulative settlement is larger. This is because when the thickness is at least 3.5 m, the structure can raise the permafrost table, thus reducing the thawing settlement. When the thickness is less than 3.5 m, the heat in the active layer cannot be completely discharged from the subgrade in cold seasons, and the residual heat will spread to the deep foundation and expand the range of the thawing interlayer. Eventually, it will increase the subgrade settlement.

**Figure 10.** Vertical deformation curves at the road centerlines and the shoulders of different structures: (a) at the road centerline and (b) at the shoulder.

The vertical deformation reflects the local deformation characteristics of the subgrade, while uneven settlement is a comprehensive index used to evaluate its stability. Comparing the difference (uneven settlement) in vertical deformations between the shoulders and the road centerlines of the five structures with block-stone layer thicknesses of 5 m, 4 m, 3.5 m, 3 m, and 2 m from 2021 to 2041 (Figure 11), the maximum uneven settlements are +0.929 cm, +0.521 cm, +0.987 cm, +3.512 cm, and +3.776 cm, respectively (positive sign means the settlement at the shoulder is smaller than that at the road centerline). When the thickness of
the block-stone layer is at least 3.5 m, the uneven settlement of the corresponding structure is small, and the stability is good. When the thickness is less than 3.5 m, the uneven settlement of the corresponding structure is large, and the stability is poor.

![Graph showing vertical deformation difference](image1)

**Figure 11.** Curves of the variation in vertical deformation difference (uneven settlement) between the shoulders and the road centerlines of different structures.

In the permafrost regions, the influence of transverse deformation on the subgrade stability is second only to settlement, but it has not yet received sufficient attention. Comparing the transverse deformations at the road centerlines and the shoulders of the five structures with block-stone layer thicknesses of 5 m, 4 m, 3.5 m, 3 m, and 2 m from 2021 to 2041 (Figure 12), it can be seen that the maximum transverse deformations at the road centerlines are $-0.352$ cm, $-0.056$ cm, $-0.046$ cm, $+0.374$ cm, and $+0.376$ cm and that the deformations at the shoulders are $+0.767$ cm, $+0.465$ cm, $+0.621$ cm, $+0.893$ cm, and $+1.435$ cm, respectively. Among them, the maximum transverse deformations at the road centerlines and the shoulders of the two structures with block-stone layer thicknesses of 3.5 m and 4 m are smaller, and their stability is better. The other structures undergo large deformations, resulting in poor stability. Notably, the increase in the self-weight load of the embankment increases the subgrade settlement when the thickness of the block-stone layer reaches 5 m. The increased settlement produces greater tensile stress, which leads to a larger transverse deformation.

![Graph showing transverse deformation curves](image2)

**Figure 12.** Transverse deformation curves at the road centerlines and shoulders of different structures: (a) at the road centerline and (b) at the shoulder.

Of note, the directions of the transverse deformations at the road centerline and the shoulder are related to the uneven water distributions and water phase changes there. Any changes in the temperature gradient and uneven settlement may cause uneven water distributions (Figure 9), thus changing the direction of the transverse deformation at this point. A relevant study has shown that [34] longitudinal cracks extend downward with a certain dip angle and that the dip angle changes with increasing depth. The difference in the water migration direction and the degree of phase change occurring at different
depths leads to different deformation directions. Especially in places where the change in
the water distribution is drastic, the influence of the water distribution on the deformation
is more obvious.

The difference in transverse deformation between the shoulder and the road centerline
can reflect the change features in longitudinal cracks in the subgrade. When the subgrade
enters the plastic deformation stage, the transverse deformation difference is positively
correlated with the width of the longitudinal crack [34].

Comparing the differences in transverse deformation between the shoulders and the
road centerlines of the five structures with block-stone layer thicknesses of 5 m, 4 m, 3.5 m,
3 m, and 2 m from 2021 to 2041 (Figure 13), it can be seen that the maximum transverse
deformation differences are +1.117 cm, +0.462 cm, +0.643 cm, +0.765 cm, and +1.164 cm,
respectively. The structure with a block-stone layer thickness of 4 m has the smallest
transverse deformation difference. This shows that a block-stone layer that is too thick or
too thin is not conducive to subgrade stability.

Table 5 shows that the maximum uneven settlement and the maximum transverse
deformation of the structure with a block-stone layer thickness of 4 m are the smallest
among the structures tested, +0.521 cm and +0.462 cm, respectively. This shows that
the overall stability of this structure is the best. The other structures undergo larger
deformations, and their stabilities are inferior to that of this structure.
6. Conclusions

Given the permafrost subgrade settlement caused by climate warming and engineering thermal disturbance, based on the engineering of the Yiershi to Chaiqiao section of 308 Provincial Highway, a subgrade structure with large block stones as embankment filler is used to improve the subgrade deformation. To analyze the stability of the structure and determine the most reasonable embankment height, the numerical calculation of the five structures with different block-stone layer thicknesses is carried out. The stabilities of these structures are analyzed in terms of three main aspects: the change in the permafrost table, the unfrozen water content distributions, and the deformation distributions. Finally, the subgrade stabilities are discussed according to two evaluation indexes: the uneven settlement and the transverse deformation difference. The following main conclusions are drawn:

(a) The increase in the block-stone layer thickness can raise the permafrost table and thus improve the thermal stability of the subgrade. The permafrost tables under the road centerlines of the five structures with block-stone layer thicknesses of 5 m, 4 m, 3.5 m, 3 m, and 2 m are 0.226 m, 0.109 m, 0.106 m, −6.064 m, and −8.461 m, respectively. When the thickness of the block-stone layer is at least 3.5 m, the permafrost table under the road centerline of the corresponding structure is higher than the natural surface. However, a slowdown in the rising rate of permafrost tables leads to a reduction in its contribution to the thermal stability of the subgrade with the increase in block-stone layer thickness. When the thickness of the block-stone layer is less than 3.5 m, the permafrost table of the corresponding structure is lower than the natural surface, resulting in a large range of the thawing interlayer in the foundation, which greatly reduces the thermal stability of the subgrade.

(b) The increase in the block-stone layer thickness can reduce the content of unfrozen water migrating from the deep foundation to the active layer, thus reducing the foundation settlement caused by permafrost thawing, which is beneficial to improving the mechanical stability of the subgrade.

(c) When the thickness of the block-stone layer is 4 m, the mechanical stability of the corresponding structure is the best among those tested. At this time, the maximum uneven settlement and the maximum transverse deformation difference between the shoulder and the road centerline are +0.521 cm and +0.462 cm, respectively. When the thickness of the block-stone layer is greater than or less than 4 m, the mechanical stability will be less optimal.

(d) Only from the perspective of thermal stability, the thicker the block-stone layer is, the more beneficial it is to the subgrade stability. From the perspective of the interaction of water, temperature, and deformation, the subgrade stability is the best when the thickness of the block-stone layer is 4 m. This shows that studying the subgrade stability from only the thermal perspective is disadvantageous. It is also the main reason for the poor general applicability of many engineering measures at present.

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