Thermal Stability Evaluation Method Based on Pile’s Bearing Capacity in a Permafrost Region

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Abstract. Pile foundations are widely adopted in the Qinghai–Tibet Railway. The pile’s bearing capacity must be influenced by the trends of global warming, rising foundation soil temperatures, expanding active layers, ground ice melting, etc. Thus, the pile’s long-term bearing capacity must be understood, and methods for evaluating its stability in permafrost regions must be developed. The bearing capacity of a pile foundation in a permafrost region is caused by the freezing strength of the pile–soil interface and support force at the pile bottom. Based on the formation mechanism of the bearing capacity of pile foundations in permafrost regions, the stability ratio of the freezing area and the freezing strength between the pile and soil are proposed to evaluate the thermal stability of the pile’s bearing capacity in permafrost. Using the method, a bearing capacity study on a single pile in typical moist permafrost and two piles in a permafrost region where warm groundwater is underground is performed. Compared with the field-measured value, the evaluation method proves to be correct and effective. Using the two parameters dependent on the soil–pile interface temperature, the effect of the thermal stability of pile bearing capacity may be described intuitively. Thus, both parameters are conducive to the analysis of pile bearing capacity formation mechanism in permafrost.

1. Introduction

Pile foundations are widely adopted in Qinghai–Tibet; the bearing capacity of pile foundations is caused by the freezing strength of the pile–soil interface and the support force at the pile bottom[1]. Pile foundation is constructed using the boring and grouting method. The frozen soil around the pile is melted based on the concrete casting temperature and hydration heat; several months after completion of the pile construct, soil is refrozen. The pile’s bearing capacity increases via the pile–soil interface freezing strength[2-3].

The research indicates the following: under global warming, permafrost engineering observes a trend toward increasing foundation soil temperatures, active layer expansion, and ground ice melting. The possible thermal trends in the permafrost region over the Qinghai–Tibetan Plateau are studied using a numerical method. The numerical results show that at the 0.04 °C/a climate warming rate, the soil where the ground surface’s average temperature is 0.5 °C degenerates into seasonal frozen soil, and the connected permafrost where the ground surface’s average temperature is 0 °C to 0.5 °C degenerates into the detachment frozen soil[4]. Based on the field observation temperature data collected over 30 years on typical sections of the Qinghai–Tibet Railway and numerical simulation, global warming causes the permafrost degeneration process along the Qinghai–Tibet Railway, which has a substantial negative effect on the stability of railroad beds during the operation period[5]. The
effects induced by climate change and engineering activity on the stability of roadbed construction based on rising foundation soil temperature, active layer expansion, and ground ice melting are summarized. The roadbed’s long-term stability is determined by a combination of mechanisms involving climate change, active layer expansion, ground ice melting, etc. As a result, the permafrost construction’s thermal stability response processes to environmental change and human activities are the bottlenecks and difficulties of permafrost construction design. Thus, the study of permafrost is a new focus of research \cite{6}. For example, Russian scholars determined the changing climate temperature coefficient for the bearing capacity calculation of permafrost\cite{7}. American scholars indicated that the profound effects caused by uncertain climate, heavy rainfall, heavy snowfall, climate warming, unusually warm summers, etc., must be considered in the design process of pile foundations in permafrost regions\cite{8}.

The bearing capacity of a pile foundation in a permafrost region has been considered under the conditions of global warming, rising permafrost soil temperatures, active layer expansion, and ground ice melting. The bearing capacity of a bridge–pile foundation in the Qinghai–Tibet Qingshuimei Bridge has been discussed by Chinese scholars. Field-measured temperature data indicate that the maximum active layer thicknesses differ in soil layers located at various distances around the pile, and the maximum thawing depths increase annually \cite{9}. Canadian scholars studied the effects of changes in the maximum thaw depth of soil around the pile on its bearing capacity; the pile’s bearing capacity decreases with the increase in maximum thaw depth\cite{10}. Long-term thermal stabilization results of power pile foundations indicate that their long-term bearing capacity decrease. Frozen soil was cooled 1 to °C in 1996–1999 based on the measurements obtained by inserting thermal probes into the soil layers and covering the soil surface around the pile with foam board\cite{11}. An assessment method was established to determine the degradation of pile stability with the reduction in the amount of frozen soil in a cold region\cite{12}. Using the finite element method to consider the effects of solar radiation and warming climate on the long-term bearing capacity of piles in permafrost, the global warming climate was found to degrade the pile’s bearing capacity\cite{13}.

Following the completion of the pile construction project, given the other general building activities, the pile absorbs heat from the atmosphere via convection and from solar radiation. The heat balance system of frozen soil layers is broken by inserting piles, resulting in additional heat flow into the frozen soil layers. Moreover, under the warming climate, active layer expansion, and ground ice melting tendencies, the pile’s long-term bearing capacity and thermal stability degrade. No unified evaluation criteria are currently available to evaluate the thermal stability of pile bearing capacity. Based on the formation mechanism of the bearing capacity of pile foundations in permafrost, the stability ratio of freezing $\phi_p$ and the freezing strength between the pile and soil $\phi_f$ are proposed to evaluate the thermal stability of pile bearing capacity in permafrost regions. Compared with the field-measured values, the evaluation method is proven to be correct and effective.

2. Thermal Stability of Pile Bearing Capacity

2.1. Evaluation parameters

The bearing capacity of a pile foundation in a permafrost region comprises the freezing strength of the pile–soil interface and the support force at the pile bottom, with the freezing strength of the pile–soil interface being the main component. According to Formula 8.3.5 in reference, the bearing capacity of a bored pile in frozen soil is computed as follows:

$$[P] = \frac{1}{2} \sum \tau_i F_i m_i + mA[\sigma]$$

where $[P]$ is the pile bearing capacity (kN); $\tau_i$ is the freezing strength between the pile surface and frozen soil layers (kPa); $F_i$ is the freezing area between the soil surface and frozen soil layers (m$^2$); $m_i$ is the corrected coefficient of the freezing force, $m_i = 1.4$; $m$ is the corrected coefficient of the support force at the pile bottom, $m = 0.7$; $A$ is the area of the support surface of the pile bottom, known as belled pile $A = 3.14 \times 0.72 = 1.54$ m$^2$; $[\sigma]$ is the allowable stress of the foundation soil located at the pile bottom, $[\sigma] = \sigma_0 = 350$ kPa. The freezing strength between the pile and frozen soil is altered by
changes in the frozen soil temperature around the pile. With global warming, the ground temperature fluctuates with the changes in season variety. Thus, the pile’s bearing capacity is altered with the seasons, showing high and low bearing capacities in winter and summer, respectively. In consideration of the safety factor, the lowest bearing capacity in a year must be determined. Thus, the lowest pile bearing capacity in summer, when the soil maximum thaw depth appears, is used to determine the pile bearing capacity. The thermal stability of pile bearing capacity $\phi$ is defined as follows:

$$\phi = \frac{[P]}{[P_0]}$$ (2)

where $[P]$ is the lowest pile bearing capacity in each year, and $[P_0]$ is the lowest bearing capacity of the pile in the first year after the new pile–soil heat balance system is established. $[P_0]$ is called the initial pile bearing capacity:

$$[P_0] = 1 \frac{\sum r_i F_{0i} m_{0i} - m_i A[\sigma_0]}{2}$$ (3)

where $r_i$(kPa) is the freezing strength between the pile and soils in the summer of the first year after the new pile–soil heat balance system is established, $F_{0i}$ is the freezing area between the pile and frozen soil layers, $m_{0i}$ is the modified factor of adfreezing force, $m_0$ is the modified factor of the supporting force at the pile bottom, and $[\sigma_0]$(kPa) is the allowable load-bearing stress of the foundation soil located at the pile bottom. Using Formulas (2)–(3), the thermal stability of pile bearing capacity $\phi$ is expressed as follows:

$$\phi = \frac{[P]}{[P_0]} = 1 \frac{\sum r_i F_i m_i - m_i A[\sigma_i]}{2}$$ (4)

The adfreezing force between the pile and soil and the supporting force at the pile bottom constitutes the pile bearing capacity; the supporting force does not exceed 10% of the total bearing capacity. Ignoring the supporting force, the pile bearing capacity is only provided by the adfreezing force between the pile and soil. Thus, the thermal stability of pile bearing capacity $\phi$ is expressed as follows:

$$\phi = \frac{[P]}{[P_0]} = 1 \frac{\sum r_i F_i m_i}{2}$$ (5)

The average freezing strength between the pile and frozen soil layers is $\bar{\tau} = \sum r_i F_i / \sum F_i$. The average freezing strength in the first year after the new pile–soil thermal balance system is established as $\bar{\tau}_0 = \sum r_{0i} F_{0i} / \sum F_{0i}$. Thus, the thermal stability of pile bearing capacity $\phi$ is expressed as follows:

$$\phi = \frac{\sum r_i F_i}{\sum r_{0i} F_{0i}} = \frac{\bar{\tau} F}{\bar{\tau}_0 F_0}$$ (6)

where $F$ is the minimum freezing area between the pile and soil in the first year after the new pile–soil thermal balance system is established; $\bar{\tau}$ and $\bar{\tau}_0$ are the minimum average freezing strengths between the pile and frozen soil layers in each year and first year after conducted, respectively. In normal conditions, the minimum freezing strength and the minimum freezing area all appear in summer. Thus, we consider that the minimum freezing strength and minimum freezing area appear at the same time in summer:

$$\phi_s = \frac{\bar{\tau}}{\bar{\tau}_0}$$ (7)

where $\phi_s$ is defined as the average freezing strength on the freezing area between the pile and soil layers, $\bar{\tau}$ is the minimum average freezing strength between the pile and soil layers each year; $\bar{\tau}_0$ is the minimum average freezing strength in the first year after the new pile–soil system thermal balance is established. Then, we set the following:
\[ \phi_A = \frac{F}{F_0} \]  

where \( \phi_A \) is the stability ratio of the freezing area between the pile and soil; \( F \) is the minimum freezing area between the pile and soil in each year (m²); \( F_0 \) is the minimum freezing area in the first year after the new pile–soil system thermal balance is established (m²).

\[ \phi = \phi_A \cdot \phi_r \]  

where \( \phi_A \) is the thermal stability ratio of the pile’s bearing capacity in permafrost. The high value of \( \phi_r \) indicates that the pile’s bearing is close to the initial design value. By contrast, the smaller value of \( \phi \) indicates that the pile’s bearing is lower than the initial design value.

3. NO. 1 Example

With a representative cast-in-site pile in typical moist permafrost as an example, using the stability ratios of the freezing area and freezing strength, the thermal stability of pile bearing capacity is described and analyzed. The pile diameter is 1 m, and its length is 23 m. The pile numerical model is simplified to a plane axisymmetric model (Figure 1).

![Figure 1 Two-dimensional pile numerical model (axisymmetric)](image)

3.1. Solar radiation and atmospheric convection

The pier exposed area absorbs the solar radiation heat, convects heat with air, and radiates with other structures. Ignoring the diffuse radiation, the bare pier’s heat conduction equation is expressed as follows:

\[ \lambda \left[ \frac{\partial T}{\partial n} \right] = h(T_c - T_a) + \alpha S \]  

where \( \lambda \) (W·m⁻¹·°C⁻¹) is the thermal conductivity of the pier concrete; \( h \) (W·m⁻²·°C⁻¹) is the heat exchange coefficient considering both convection and radiation; \( T_c \) (°C) is the concrete surface temperature; \( n \) (°C·m⁻¹) is the temperature gradient direction; \( T_a \) (°C) is the temperature of air in contact with concrete; \( \alpha \) is the solar radiation absorption coefficient of the concrete surface; \( S \) (W·m⁻²) is the solar radiation intensity.

The model is simplified as a plane model, and ignoring the light and shadow effects, the pier’s surfaces receive the same intensity of solar radiation. Ignoring solar radiation intensity change in a day, the solar radiation intensity function received by the pier surface is summarized through the 1998–2002 measured satellite data as follows:\[14]:

\[ S = 186.4 + 66.88 \sin \left( \frac{2\pi}{8760} \theta \right) \]

where \( S \) (W·m⁻²) is the solar radiation intensity change regulation in a year, and \( t \) is time (h).

According to references \[15-16], the atmospheric temperature is described as a sinusoidal function:
where the annual atmospheric temperature is −3.4 °C; the temperature amplitude is 12.2 °C; the annual atmospheric temperature rises by 2.6 °C in 50 years, \( A = 2.28 \times 10^{-6} ^\circ C/h \); \( t \) is time (h).

With high wind velocity on the plateau, the solar radiation absorption coefficient of concrete structure’s surface is set as \( \alpha = 0.42 \) [17-19], and the coefficient of convective heat transfer on concrete surface is set as \( h = 4.74 \) (W·m\(^2\)·°C\(^{-1}\)) [20].

3.2 Initial and boundary conditions
The in-site soil temperature data in July 2000 are set as the initial temperature of numerical model. Given that the soil temperature around the pile is influenced by the pile and the soil far from the pile, the boundary layer (the layer located 0.5 m under the soil surface) of soil is free. The boundary layer’s temperature (the layer located 0.5 m under the soil surface) 1 m away from the pile is set as a sinusoidal function (Formula (14)). The boundary condition equations in the numerical model are as follows:

\[
\frac{\partial T}{\partial n} = 0 (r = 15 m) \\
\frac{\partial T}{\partial n} = \zeta (y = 30 m) \\
T = T_s(r \geq 4.5 m, y = 0.5 m)
\]  

where \( T \) is the soil temperature; \( \zeta \) is the geothermal gradient in the depth direction, and its value is 0.03°C/m; \( T_s \) is the layer temperature (the layer located 0.5 m under the soil surface):

\[ T_s = 0.9 + 12.2 \sin\left(\frac{2\pi}{8760} \cdot \theta\right) + At \]  

where \( A = 2.28 \times 10^{-6} ^\circ C/h \) based on the annual average temperature −3.4 °C in 2000, and it will rise by 2.6 °C in the next 50 years. \( t \) is time (h).

3.3 Frozen soil heat conduction
Heat conduction equilibrium equation of frozen soil:

\[
\frac{\rho C}{\partial T} = \frac{d}{dx}\left(\frac{\lambda}{dx}\right) + \frac{d}{dy}\left(\frac{\lambda}{dy}\right) + \rho L \frac{df}{\partial T}
\]

where \( \rho \) (kg·m\(^{-3}\)) is soil density; \( c \) (J·kg\(^{-1}\)·°C\(^{-1}\)) is the specific heat capacity of the soil, \( T \) is the soil temperature; \( f \) is the solid phase rate, dimensionless quantity, \( \lambda \) (W·m\(^{-1}\)·°C\(^{-1}\)) is the thermal conductivity of soil; \( L \) is the water latent heat (335 kJ·kg\(^{-1}\)). The increase (decrease) in \( f \) is proportional to the release (absorption) of latent heat through phase change. In the non-phase change zone, \( \partial f / \partial t = 0 \), Equation (15) becomes the common conduction equilibrium equation. The solid phase rate \( f \) is related to soil temperature:

\[
f = \left(\frac{T_L - T}{T_L - T_s}\right)
\]

where \( T_L \) and \( T_s \) are the thawing and freezing temperatures, respectively; \( T \) is the soil temperature in the phase change zone. The soil thermal conductivity \( \lambda \) (W·m\(^{-1}\)·°C\(^{-1}\)) and specific heat \( c \) (J·kg\(^{-1}\)·°C\(^{-1}\)) in the solid and liquid phases, respectively, are assigned to \( \lambda_s \) and \( \lambda_L \); \( C_s \) and \( C_L \). The soil thermal conductivity \( \lambda \) in the phase change zone is linearly interpolated between \( \lambda_s \) and \( \lambda_L \). Table 1 shows the thermal physical parameters of the pile and soil.

| Material         | Depth /m | Moisture content/% | Dry density /kg·m\(^{-3}\) | \( \lambda_s \)/W·m\(^{-1}\)·°C\(^{-1}\) | \( \lambda_f \)/W·m\(^{-1}\)·°C\(^{-1}\) | \( C_u \)/J·kg\(^{-1}\)·°C\(^{-1}\) | \( C_l \)/J·kg\(^{-1}\)·°C\(^{-1}\) | \( C_u \)/J·kg\(^{-1}\)·°C\(^{-1}\) |
|------------------|----------|--------------------|----------------------------|----------------------------------------|----------------------------------------|----------------------------------|----------------------------------|----------------------------------|
| Silty clay       | 0~1.5    | 40                 | 1200                       | 1.128                                  | 1.581                                  | 2830                            | 2230                             | 62254                            |
| Peat silty clay  | 1.5~3.5  | 60                 | 900                        | 0.569                                  | 0.848                                  | 2480                            | 1980                             | 78125                            |
Gravel silty clay | 3.5~8.0 | 24.5 | 1500 | 1.418 | 1.569 | 2480 | 1871 | 38258
Weathered bedrock | 8.0~30 | 4 | 2500 | 2.700 | 2.700 | 2290 | 2290 | 39865
Reinforced concrete | — | — | 2500 | 1.740 | 1.740 | 2300 | 2300 | —

3.4 Temperature of the pile–soil interface and pile bearing capacity
As the calculation results show, given the warming climate and solar radiation heat brought into frozen soil layers through the pile, the temperature of the soil–pile interface 4–20 m below the surface will increase in 2010–2050. The temperature of the pile–soil interface 0–4 m below the surface fluctuates with the external environment. The temperature of the pile–soil interface 4–20 m below the surface will be higher than −0.5 °C in 2050.

Fig. 2 Temperature of the pile–soil interface in March

Fig. 3 Temperature of the pile–soil interface in September
Table 2  2009 Measurement of soil temperature

| Depth/m | March soil temperature °C | September soil temperature °C |
|---------|---------------------------|-------------------------------|
| 0       | -6.5                      | 0.53                          |
| 5       | -1.625                    | 1.25                          |
| 10      | -0.92                     | -1.12                         |
| 15      | -0.82                     | -0.85                         |
| 20      | -0.78                     | -0.71                         |

Table 3 Tangential freezing strength between the pile and soil

| Soil type         | Freezing strength between the pile and the soil/kPa |
|-------------------|-----------------------------------------------------|
|                   | Monthly average soil temperature °C                |
|                   | >0        | -0.5      | -1.0      | -1.5      | -2.0      | -2.5      | -3.0      |
| Crushed stone soil| 30        | 70        | 110       | 150       | 190       | 230       | 270       |
| Sand soil         | 30        | 90        | 130       | 170       | 210       | 250       | 290       |
| Cohesive soil     | 30        | 60        | 90        | 120       | 150       | 180       | 220       |

Table 2 shows the measured soil temperature in March and September 2009. The calculated results in Figs. 2 and 3 are in good agreement with the measured data in Table 2. The tangential freezing strengths between the pile and different soil layers are obtained through linear interpolation in Table 1 based on the pile-soil interface temperature (Figures 1 and 2), and the pile bearing capacity is calculated based on Formula (1), as given in Table 4. The results show the following: the temperature of the soil–pile interface rises via warming climate and solar radiation, and the pile bearing capacity degenerates over time.

Table 4 Calculated bearing capacity of a single pile

| Pile's bearing capacity/kN | Year |         |         |         |         |
|---------------------------|------|---------|---------|---------|---------|
|                           | 2010 | 2020    | 2030    | 2040    | 2050    |
| March                     | 5110.9 | 4852.4  | 4476.1  | 4010.2  | 3628.6  |
| September                 | 2770.1 | 2691.3  | 2514.1  | 2263.7  | 2004.9  |

3.5 Thermal stability evaluation of the pile’s bearing capacity

The thermal stabilities of pile bearing capacity (Table 5) are calculated in accordance with Formula (4), and the bearing capacity in the summer of 2010 is considered as the initial value. The thermal stabilities of the pile’s bearing capacity (Table 6) are calculated using Formula (9). By comparing Tables (5) and (6), the thermal stability value obtained with Formula (9), which ignores the pile’s bottom support force, is smaller than the calculated value computed using Formula (4), with the error between the values obtained by Formulas (4) and (9) being less than 10%. Thus, the thermal stability evaluation method using the stability ratio \( \varphi_A \) of the freezing area and the freezing strength \( \varphi_\tau \) between the pile and soil is close to Design Specific\(^{[1]}\) and conservative. Furthermore, the pile’s bearing capacity reduction mechanism may be described by the two coefficients \( \varphi_A \) and \( \varphi_\tau \), and the contributions of the freezing area and strength to the pile’s bearing capacity are reflected distinctly.

Table 5 Thermal stability of pile bearing capacity in different years

| Thermal stability of the pile’s bearing capacity | 2010 | 2020 | 2030 | 2040 | 2050 |
|------------------------------------------------|------|------|------|------|------|
| Stability ratio of the freezing area \( \varphi_A \) | 1    | 0.999| 0.995| 0.990| 0.986|
| Stability ratio of the average freezing strength \( \varphi_\tau \) | 1    | 0.952| 0.855| 0.720| 0.559|
| Thermal stability of the pile’s bearing capacity \( \varphi \) | 1    | 0.951| 0.851| 0.713| 0.551|

Table 6 Calculated minimum pile bearing capacity in different years/kN

| Calculated pile’s bearing capacity/kN | Year |
|---------------------------------------|------|
The numerical results (Table 5) show the following: the main reason for the degeneration of pile bearing capacity is the freezing strength reduction between the pile and soil, and the freezing area shows no evident change. The numerical results (Table 6) show the following: the main reason for the degeneration of pile bearing capacity is the freezing strength reduction between the pile and soil, and the freezing area shows no evident change.

4. NO. 2 Example

With a pile foundation in a frozen soil region with warm groundwater as an example, the thermal stabilities of pile bearing capacity are evaluated using the stability ratio $\phi_A$ of the freezing area and the freezing strength $\phi_f$ between the pile and soil. The thermal stability values of the bearing capacity are proportional to the in-situ pile settlements, and the evaluation method is feasible and widely available.

A bridge located in a permafrost region suffered from insufficient bearing capacity and excessive settlement problems in 2009. The bridge is seated on the right bank of the river valley, which is in the middle of a wide basin located 5030–5040 m above sea level, and the vegetation coverage rate is approximately 5%. The annual average temperature is $-4°C$ to $-5.2°C$ in this zone, and October to the following April are negative-temperature months. The annual rainfall, including rain, snow, and hail, is 205–300 mm, mostly concentrated in a period of 6–8 months, and the annual evaporation is approximately 1500 mm.

The upper part of the main stratum is Quaternary Holocene pluvial breccia soil and fine sand, and the lower part is Jurassic marl. In the original design geological prospecting for the bridge, several abutments are located in the low-temperature stable sub-region ($-2.0°C \leq TCP \leq -1.0°C$), and the others are located in the high-temperature unstable sub-region. No groundwater exists in the soil layers. In 2010, a geological survey was conducted to control disease. The survey results showed that the geological structure, the upper limit of permafrost, and the classification of frozen soil are consistent with the geological data of the original design, and the permafrost base changed greatly under abutments. The maximum value of the permafrost base is 39.5 m, and minimum value is 19.9 m; the thickest permafrost base is approximately 30 m. Moreover, confined water exists under frozen soil layers around the abutments. With two piers as examples, 0.6°C confined water is prospected 19.9 m below ground level around pier No.1, which has a length of 20 m and diameter of 1 m; 0.7°C confined water is prospected 28.3 m below ground level around pier No.2, which has a length of 20 m and diameter of 1 m.

| Duration time/a | Initial | 1  | 2  | 3  | 4  | 5  |
|-----------------|---------|----|----|----|----|----|
| Stability ratio of the freezing area $\phi_A$ | 1       | 0.944 | 0.917 | 0.902 | 0.897 | 0.872 |
| Stability ratio of the average freezing strength $\phi_f$ | 1       | 0.855 | 0.761 | 0.728 | 0.694 | 0.660 |
| Thermal stability of the pile’s bearing capacity $\phi$ | 1       | 0.807 | 0.697 | 0.652 | 0.622 | 0.576 |

| Duration time/a | Initial | 1  | 2  | 3  | 4  | 5  |
|-----------------|---------|----|----|----|----|----|
| Thermal stability of the pile’s bearing capacity | 1       | 0.957 | 0.872 | 0.752 | 0.610 |

The numerical results (Table 5) show the following: the main reason for the degeneration of pile bearing capacity is the freezing strength reduction between the pile and soil, and the freezing area shows no evident change. The numerical results (Table 6) show the following: the main reason for the degeneration of pile bearing capacity is the freezing strength reduction between the pile and soil, and the freezing area shows no evident change.

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A bridge located in a permafrost region suffered from insufficient bearing capacity and excessive settlement problems in 2009. The bridge is seated on the right bank of the river valley, which is in the middle of a wide basin located 5030–5040 m above sea level, and the vegetation coverage rate is approximately 5%. The annual average temperature is $-4°C$ to $-5.2°C$ in this zone, and October to the following April are negative-temperature months. The annual rainfall, including rain, snow, and hail, is 205–300 mm, mostly concentrated in a period of 6–8 months, and the annual evaporation is approximately 1500 mm.

The upper part of the main stratum is Quaternary Holocene pluvial breccia soil and fine sand, and the lower part is Jurassic marl. In the original design geological prospecting for the bridge, several abutments are located in the low-temperature stable sub-region ($-2.0°C \leq TCP \leq -1.0°C$), and the others are located in the high-temperature unstable sub-region. No groundwater exists in the soil layers. In 2010, a geological survey was conducted to control disease. The survey results showed that the geological structure, the upper limit of permafrost, and the classification of frozen soil are consistent with the geological data of the original design, and the permafrost base changed greatly under abutments. The maximum value of the permafrost base is 39.5 m, and minimum value is 19.9 m; the thickest permafrost base is approximately 30 m. Moreover, confined water exists under frozen soil layers around the abutments. With two piers as examples, 0.6°C confined water is prospected 19.9 m below ground level around pier No.1, which has a length of 20 m and diameter of 1 m; 0.7°C confined water is prospected 28.3 m below ground level around pier No.2, which has a length of 20 m and diameter of 1 m.

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With a pile foundation in a frozen soil region with warm groundwater as an example, the thermal stabilities of pile bearing capacity are evaluated using the stability ratio $\phi_A$ of the freezing area and the freezing strength $\phi_f$ between the pile and soil. The thermal stability values of the bearing capacity are proportional to the in-situ pile settlements, and the evaluation method is feasible and widely available.

A bridge located in a permafrost region suffered from insufficient bearing capacity and excessive settlement problems in 2009. The bridge is seated on the right bank of the river valley, which is in the middle of a wide basin located 5030–5040 m above sea level, and the vegetation coverage rate is approximately 5%. The annual average temperature is $-4°C$ to $-5.2°C$ in this zone, and October to the following April are negative-temperature months. The annual rainfall, including rain, snow, and hail, is 205–300 mm, mostly concentrated in a period of 6–8 months, and the annual evaporation is approximately 1500 mm.

The upper part of the main stratum is Quaternary Holocene pluvial breccia soil and fine sand, and the lower part is Jurassic marl. In the original design geological prospecting for the bridge, several abutments are located in the low-temperature stable sub-region ($-2.0°C \leq TCP \leq -1.0°C$), and the others are located in the high-temperature unstable sub-region. No groundwater exists in the soil layers. In 2010, a geological survey was conducted to control disease. The survey results showed that the geological structure, the upper limit of permafrost, and the classification of frozen soil are consistent with the geological data of the original design, and the permafrost base changed greatly under abutments. The maximum value of the permafrost base is 39.5 m, and minimum value is 19.9 m; the thickest permafrost base is approximately 30 m. Moreover, confined water exists under frozen soil layers around the abutments. With two piers as examples, 0.6°C confined water is prospected 19.9 m below ground level around pier No.1, which has a length of 20 m and diameter of 1 m; 0.7°C confined water is prospected 28.3 m below ground level around pier No.2, which has a length of 20 m and diameter of 1 m.

| Duration time/a | Initial | 1  | 2  | 3  | 4  | 5  |
|-----------------|---------|----|----|----|----|----|
| Stability ratio of the freezing area $\phi_A$ | 1       | 0.944 | 0.917 | 0.902 | 0.897 | 0.872 |
| Stability ratio of the average freezing strength $\phi_f$ | 1       | 0.855 | 0.761 | 0.728 | 0.694 | 0.660 |
| Thermal stability of the pile’s bearing capacity $\phi$ | 1       | 0.807 | 0.697 | 0.652 | 0.622 | 0.576 |

| Duration time/a | Initial | 1  | 2  | 3  | 4  | 5  |
|-----------------|---------|----|----|----|----|----|
| Thermal stability of the pile’s bearing capacity | 1       | 0.957 | 0.872 | 0.752 | 0.610 |
Stability ratio of the freezing area $\varphi_A$

|            | 1  | 1  | 1  | 1  | 1  | 1  |
|------------|----|----|----|----|----|----|

Stability ratio of the average freezing strength $\varphi_T$

|            | 1  | 0.968 | 0.920 | 0.889 | 0.858 | 0.827 |
|------------|----|--------|--------|--------|--------|--------|

Thermal stability of the pile's bearing capacity $\varphi$

|            | 1  | 0.968 | 0.920 | 0.889 | 0.858 | 0.827 |
|------------|----|--------|--------|--------|--------|--------|

Based on the measured geological parameters, the thermal analysis model of pile foundation is established considering the temperature and location of groundwater, and the ground temperature field of the pile–soil system is obtained. According to the numerical results, the thermal stability $\varphi$ of the bearing capacity of the pile in permafrost region is evaluated by the freezing area stability rate $\varphi_A$ and the freezing strength between the pile and soil $\varphi_T$. Then, the results are compared with the in-situ measured settlement values.

In summary, under the action of 0.6 °C confined water at the pile bottom, the NO.1 pile’s stability ratios of the freezing area $\varphi_A$ and average freezing strength $\varphi_T$ are reduced. Under the action of 0.7 °C confined water 8.3 m below the pile bottom, the NO.2 pile’s stability ratio of the freezing area $\varphi_A$ did not decrease, whereas the stability ratio of the average freezing strength $\varphi_T$ decreased. Comparing the two piles’ thermal stabilities, the warm confined water directly acting on the pile causes reduction in the stability ratios of the freezing area and the strength. The warm confined water below pile bottom reduces the stability ratio of the average freezing strength $\varphi_T$, whereas it has no influence on the stability ratio of the freezing area $\varphi_A$. Under 5 years of action of warm confined water, the thermal stabilities of NO. 1 and NO. 2 pile bearing capacities decreased to 0.576 and 0.827, respectively.

The pile settlement curve the in-situ test (Figure 4) showed that the NO. 1 pile settlement is greater than that of the NO. 2 pile. The thermal stability of NO. 1 pile bearing capacity $\varphi$ is smaller than that of the NO. 2 pile. The relationship between the thermal stability of the bearing capacity and the in-situ settlements is coincident. The pile with greater thermal stability has smaller settlement, and the pile with smaller thermal stability has larger settlement. The thermal stability of the bearing capacity of pile $\varphi$ in a permafrost region is evaluated by the stability ratio of the freezing area $\varphi_A$ and the stability ratio of the average freezing strength $\varphi_T$, in agreement with the actual situation.

![Figure 4 Pile's settlement curve in-situ test](image-url)

**5 Conclusions**

1. The frozen soil layer’s heat balance is broken via pile insertion because the pile’s surface exposed to air absorbs solar radiation, transfers heat into the frozen soil layers, and causes the temperature rise at 1 m soil around the pile and 5 m under the ground surface, resulting in the reduction of pile bearing capacity.

2. Under the combined action of warmer climate and solar radiation, the temperature of the pile–soil interface in the permafrost horizon increases, and the refreezing strength decreases. Given that the
stability ratio of the average freezing strength $\varphi_t$ is reduced, and the stability ratio of the freezing area $\varphi_A$ remains unchanged, the reduction of the stability ratio of the average freezing strength $\varphi_t$ decreases the pile bearing capacity.

(3) Based on the formation mechanism of the bearing capacity of a pile foundation in permafrost, the stability ratio $\varphi_A$ of the freezing area and the freezing strength $\varphi_t$ between the pile and soil are proposed to evaluate the thermal stability $\varphi$ of pile bearing capacity in a permafrost region. The two major temperature-dependent factors of the pile–soil interface freezing strength and area, which cause the decrease in bearing capacity, are visually expressed, and the contribution of both factors to the thermal stability of the pile foundation bearing capacity in a permafrost region is highlighted. This method can be used to analyze the mechanism of the decrease in the bearing capacity of a pile foundation in a permafrost region.

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