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Full Length Research Paper

Bearing characteristics and influencing factors analysis of bored pile under refreezing condition in permafrost region

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This paper represents study results of the bearing characteristics and influencing factors of large diameter bored pile under refreezing condition in permafrost region. Principles for designing piles in permafrost on the basis of both ultimate capacity and limiting deformations are first reviewed. To solve problems such as pile cap and bridge pier construction, super structure construction, time limit construction, a field experiment was carried out on both the bearing capacity and deformation characteristic of large diameter bored pile in high-temperature fine-granular frozen soil of Fenghuoshan region of the Qinghai-Tibet plateau. Based on ground temperature and *in-situ* experiment data, pile tip resistance accounted for the entire load is just 2.5% when load on pile top reached 6000 kN after frozen soil refreezing was completed. In the meantime, it was also observed that the deformation features of refreezing large diameter bored pile in permafrost. The bored pile is a typical friction pile of the friction pile characterized by the settlement characteristic that pile tip resistance had less loading sharing ratio. The results also indicated that it was very difficult for the permafrost to refreeze to the original state as soon as it had been disturbed due to the bored pile construction. Also, pile shaft force and pile side resistance were connected to pile top loading, characteristic of frozen soil around the pile and ground temperature. The study results can be used as some guides and references in the process of pile foundation construction.

Key words: bored pile, permafrost, pile side resistance, pile bearing capacity

INTRODUCTION

Qinghai-Tibet railway with a length of 1118 km has been completed across the permafrost hinterland of the Qinghai-Tibet Plateau between Golmud and Lhasa in 2008. Some 630 km of the railway run across permafrost with a mean annual air temperature of -7 to -2°C. Permafrost, as a term used to describe permanently frozen ground, indicates a thermal condition where the temperature of the rock or soil remains below freezing

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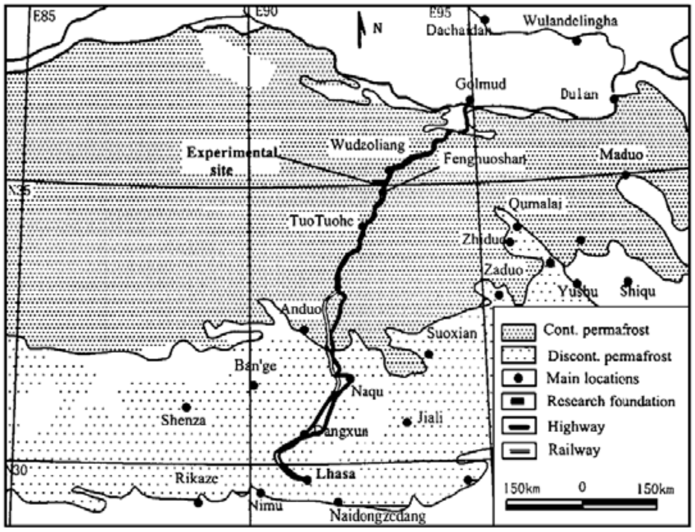


Figure 1. Test site of bored pile in the Qinghai-Tibet Plateau Permafrost.

throughout the year. Permafrost can exist for as few as three years to more than tens of thousands of years (Biggar and Kong, 2001; Suleiman et al., 2006; Zhang et al., 2008; JGJ118, 2011). About one-fifth of the earth's land mass contains permafrost. Almost one-half of the world's permanently frozen ground is in Alaska, Russia and Siberia, one-third is in Canada, and a large portion is in Qinghai-Tibet plateau and Northeast of China. Disturbing permafrost carelessly may cause thaw, resulting in uneven foundation settling and disastrous consequences for the building. And it is not always possible to safely build on permafrost. Permafrost can either be continuous or permanent. Changes in climate and construction disturbance may cause permafrost to thaw and disappear (Wang et al., 2013; Xiuli et al., 2012; Yao et al., 2012; Wang et al., 2008). Consequently, the ramifications for foundation design and particularly pile foundation design in permafrost are very significant, otherwise may bring about a great reduction in allowable load for foundations in permafrost (Zhu et al., 1982; Niu and Liu, 2004). Foundation design in permafrost must reckon with both thermal and rheological considerations.

However, bored pile, as an important foundation type in permafrost regions, can increase stability of bridges during warm weather more than other foundations and usually be used widely in the world. Ultimate capacity of piles is assessed by consideration of adfreeze strengths. A design based on settlement must ensure that pile displacements throughout the life of the structure are tolerable. So a lot of field load experiments have been carried out (Wang et al., 2005; Guo and Li, 2002; Zhang, 2002; Li et al., 2002) because prior experience with laboratory testing and field loading testing of permafrost is quite limited. These publications mainly considered the interaction mechanism between pile and permafrost, influence factors of the frozen strength, actual vertical bearing capacity measurement of pile and so on.

To meet the needs of the pipeline and housing construction in permafrost regions, the smaller diameter steel piles in North American countries had been studied primarily (Wu, 2003; Liu et al., 2002; Wang et al., 2003; Wu, 2003; Ma, 2003). Large diameter reinforced concrete piles had been seen seldom in permafrost regions of Russia although pile foundation is widely used in Russia. Model tests had been carried out to study the main factors of the frozen force in order to know the general rule that the freeze force influenced on temperature, moisture, soil particles on the freeze force (Crowther, 2013; Gang et al., 2013). Afterward, constructed buildings in Qinghai-Tibet plateau and Northeast of China requires specific knowledge about permafrost and specialized building techniques. The field loading testing the piles of house had been carried out in cold region of China northeast. It also meets constructing railways in Daxinanling. The pile testing ground had been set up specially to use for the static loading test and studying construction technology in permafrost region of Qinghai-Tibet plateau to meet the highway and railway constructions. However, so far the testing piles in permafrost have less than 0.65 m diameter and less than 8 m length. The testing piles have no means to meet the Qinghai-Tibet railway construction. Therefore, in order to know the formation law of bearing capacity of bored piles induced by the ground temperature change after bridge pile construction, the interaction between pile and soil around the pile must be carefully studied on conditions of field loading testing when the frozen soil refreezes. The transfer process of shaft force, pile lateral resistance and pile tip resistance must also be considered. The bearing experiment of bored pile in permafrost region of Qinghai-Tibet plateau will be used for the design and construction of railway.

IN SITU TEST

Ground temperature test hole was drilled at the southern Basin site of Beiluhe in Fenghuoshan Region between the Hoh Xil and Fenghuo Mountain on September 1, 2001 from Figure 1. It can be shown from Figure 2 that the site was found to be underlain by brownish red sandy soil, a fine-grained partly gritty soils, for example, angular gravel, detritus to a depth of about 0.7 m. At a depth of 0.7 m, a 3.3 m thick layer of brownish red clay, a massive ground-ice including soil, was present continuously across the site, and this layer was underlain again in turn by intense-weathered mudstone and sandstone, respectively at depths of 4 and 15.5 m, respectively below natural ground surface.

The measured temperatures on disturbed samples were obtained from test hole and some of the early hole for pile installation was highly variable from Figure 2. The sedimentary in the construction area are mainly lacustrine

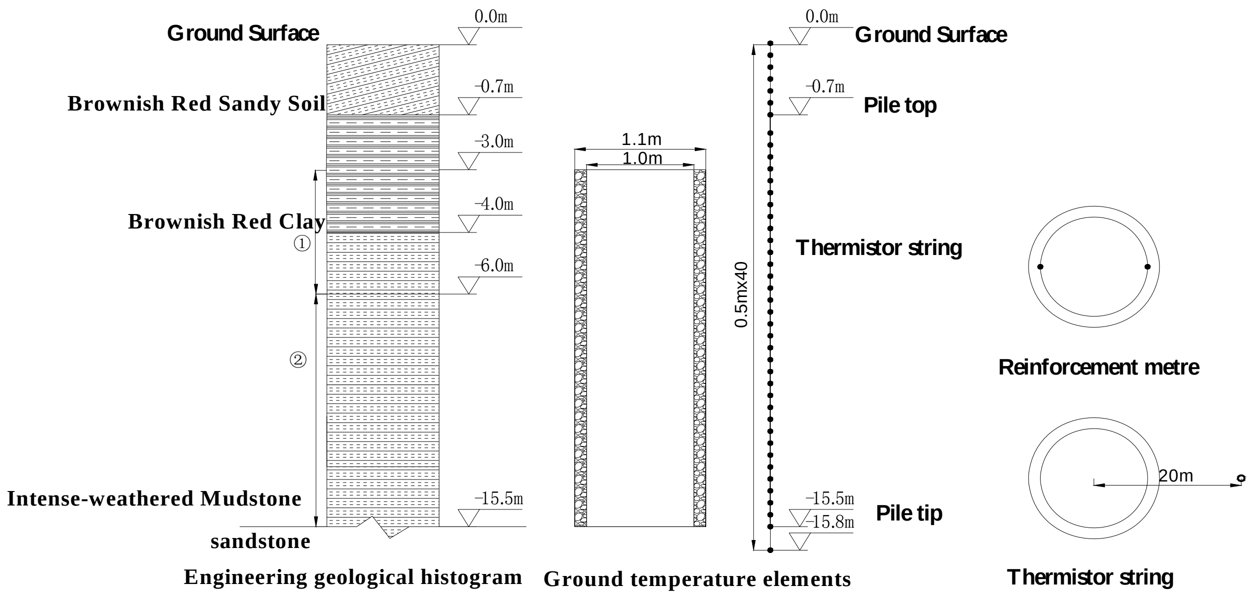


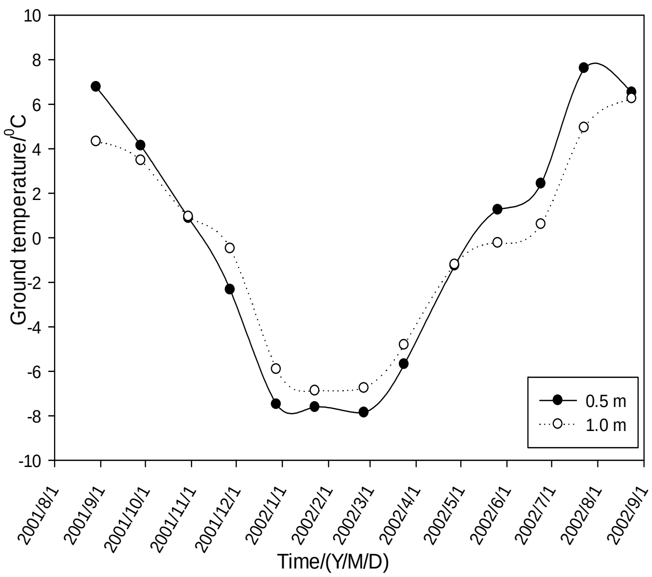
Figure 2. Geological columns, monitoring point arrangement and ground temperature elements.

sediments of tertiary and alluvial layer in holocene of quaternary. Massive ground-ice containing soils occur below the table with a typical thickness of 1 to 2 m. Ice-saturated and ice-rich soils are formed below the ice layers. According to the observed data from 2001 to 2002 in Figure 3, the mean annual air temperature in the region is -3.8°C and means annual ground temperature at a depth of 15.0 m ranges between -1.6 and -0.9°C, respectively. The annual ground temperature over most of the experimental site lies below -1.0°C. According to the permafrost site division theory based on ground temperature in China, the district is considered as basically stable with low temperature. And it belongs to the area of thick layer underground-ice in permafrost regions (JGJ118, 2011). It also is the arid climate zone and the freeze period as long as 7 to 8 months every year. At the site, the strata consist predominantly of lacustrine deposits of the upper Tertiary and diluvium of the Quaternary Holocene Series, such as silty clay. The permafrost table averages 2.0 to 2.5 m deep, but ranges up to 2.8 m deep.

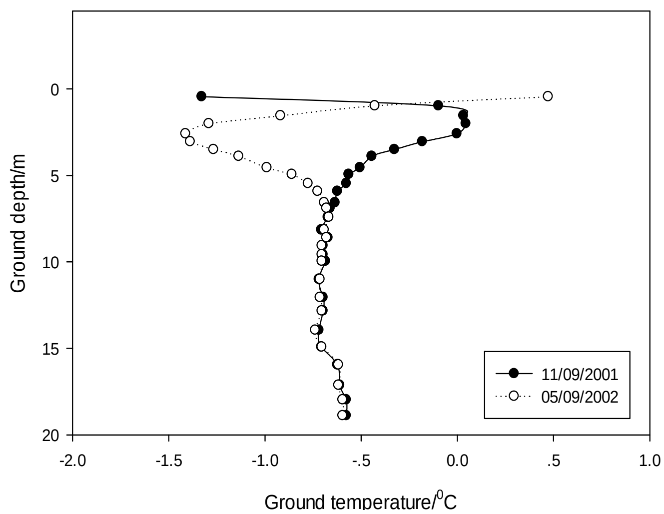
Permafrost is an excellent foundation as long as it remains frozen. But it is very sensitive to temperature changes. Changes to the ground surface, for example, removing the ground cover or constructing will change the ground temperature and cause the permafrost to thaw and possibly lose its rigidity. A bridge radically changes the way heat moves in and out of the soil; and constructing a bridge on a permafrost site will affect the permafrost. Bridges are normally heated in the construction and this will add heat to the soil. A bridge also shades the soil in the summer, preventing exposure to the sun. So, the soil is warm in the winter when it should be cold, and cold in the summer when it should be warm. And every attempt is made to keep the soil beneath the bridge frozen and the permafrost stable. The strategy for alleviating the engineering risks of bridge on permafrost sites is to build the structure on piles or an elevated foundation, taking special care to insulate the ground and prevent thawing.

After general site conditions are evaluated, a more detailed investigation is normally required at the specific construction site. Structure of pile can be selected when there is permafrost present. Perennial freezing on well-drained, coarse-grained river sand and gravel or bedrock can be ignored because it has few associated problems. But in the permafrost zone, particularly with fine-grained soils with high ice content, every effort must be made to preserve frozen conditions. For some types of structures, it may not be possible to prevent thawing without special design considerations. However, the piles were widely used because the piles should be well embedded in the permafrost and the structure rose above the ground to permit natural air circulation beneath the structure and to minimize heat flow from the structure to the frozen ground. Piles are also driven in place with a pile driver. Therefore, the permafrost must be thawed with a steam jet while the bored pile can be constructed by drilling not by a steam jet. On the contrary, the permafrost can seldom be thawed when the bored pile can be drilled. Some permafrost around the pile can be thawed only after the concrete are poured into the drill. But the thawed permafrost would restore the original frozen state after refreezing. And the refreezing of the thawed permafrost around the pile would also increase the depth, stability and amount of permafrost and for stabilization during cold weather.

In addition, building foundations should be designed



(a)



(b)

Figure 3. Ground temperature (a) at the depth of 0.5 and 1.0 m versus time and (b) at the depth of 0 to 20 m.

with a uniform weight distribution considering heaving action and thawing settlement; for example, lightly-loaded, improperly anchored piles may be pushed out of permafrost by the active layer and heaving action while the bored pile was considered as a uniform weight distribution preventing from be pushed out of permafrost by the active layer and heaving action. So the bored piles were widely used in the Qinghai-Tibet railway. In the permafrost region of Qinghai-Tibet, the diameter of railway test pile is designed as 1.0 m, and the embedded length is 12.5 m. It belongs to the large diameter bored pile according to the Technical Code for Building Pile Foundations of China (JGJ118, 2011).

MATERIALS AND METHODS

Pile placement

Based on a review of published information at the time of installation on permafrost properties and pile load capacities in such a material, it was considered that significant uncertainties were present concerning pile design in frozen ground. So it was decided to carry out a limited program of pile load testing to confirm or modify some preliminary estimates of pile load capacities. Owing to the short construction period available for foundation construction, and the requirement to complete the piling in the first construction season, piling operations were already underway before pile testing was started. Ground temperature of frozen soil surrounding the pile measurements were obtained during the pile testing program from 25 thermistor strings installed on September 28, 2011. Readings at subsequent dates are shown plotted in Figure 3a. It is observed that all thermistor strings were installed about 1 m away from the axis of test pile. The measuring hole installing thermistor strings had a depth of 15 m. The distance between two thermistor strings is 0.5 m. The readings indicate that the thawed active layer extended to about 8 m below ground surface.

To measure the shaft force of test pile, side resistance and tip resistance of pile, the 25 steel instruments were installed symmetrically inside a concrete-filled test pile. The distance between two steel bar meters is 0.5 m. Test pile had been constructed by the underwater perfusion concrete method (JGJ118, 2011). Separate posts were embedded into the permafrost to act as displacement gage supports. Two displacement gages accurate to 0.001 mm were mounted on the posts in such a way as to measure the settlement of the crossbeam on either side of the point of load application. The displacement gage readings were then averaged to obtain the settlement of the pile head. Survey levels of the pile heads were also taken from time to time to confirm the overall accuracy of the pile settlement monitoring as recorded by the displacement gages. Test pile began to be constructed on November 27, 2001. Drill of bridge piles was completed by the percussion drill. And steel cage was welded at the bridge site. The poured concrete was mixed near the concrete batching plant. Temperature of concrete varied from 1 to 3°C when poured into the mold. And test pile was completed on the end of December, 2001.

Loading method

Loading system consists of anchor pile, reaction beam and jack as shown in Figure 4. A steel crossbeam was fabricated, and was welded to four adjacent piles for each pile load test. Separate posts were embedded into the permafrost to act as dial gauge supports. Four dial gauges accurate to 0.025 mm were mounted on the posts in such a way as to measure the settlement of the crossbeam on either side of the point of load application. The dial gauge readings were then averaged to obtain the settlement of the pile head. Survey levels of the pile heads were also taken from time to time to confirm the overall accuracy of the pile settlement monitoring as recorded by the dial gauges. Considering the static loading test of pile in permafrost foundation and the experience of the foundation pile on the site on the Tibetan plateau carried out by Northwest Research Institute of Railways Ministry in the 1970s and 1980s and physical and mechanical properties of permafrost soil around the pile after soil mass refreezing, the fast loading method was used as loading of test pile. The main steps are as follows (JGJ118, 2011).

Load classification

Load was applied by a 600 kN hydraulic jack. The jack was calibrated before and after the test program to ensure accuracy of

Figure 4. Field pile loading test.

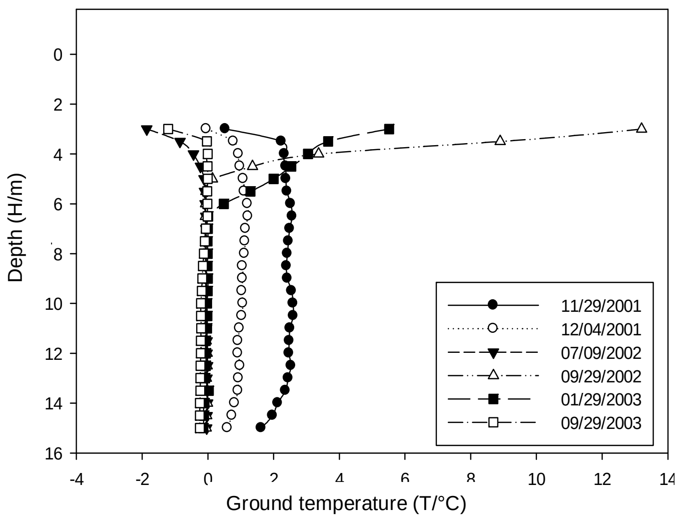


Figure 5. Curve of ground temperature around pile.

the load application system. Considerable fluctuations in the applied jack load occurred during the test pile, and steps were taken to reduce these fluctuations in subsequent load tests. According to the estimated ultimate bearing capacity of test pile, the load level is divided into 10 levels. Load was applied by a 600 kN hydraulic jack between 1 and 6 and between 9 and 10. Other load was applied by a 400 kN hydraulic jack. The settlement of the pile top is less than 0.1 mm every hour, or settlement rate of test pile is less than 0.1 mm/h under each level of loading. However, the settlement of testing pile has been considered to be stable under the loading level if the sedimentation rate of continuous observed average settlement in half-hour appears three times. The field loading test can be terminated when the settlement test pile accelerated or its total settlement of pile block is more than 40 mm. Considerable fluctuations in the applied jack load (up to 10% of the applied 600 kN load) occurred during the test pile, and steps were taken to reduce these fluctuations in subsequent load tests. The unloading value is as much as 2 times of loading value.

Frequency readings

During normal working periods, readings were generally taken once separately every 5, 15, 40 min during each stage of loading, and during night time periods, every 4 to 8 h. Afterwards, loading is measured once every one hour. Loading is measured once every 30 min when loading is close to the bearing capacity of the pile. Ground temperature of permafrost around the pile was observed once every 24 h. This reading schedule was continued for the duration of each test, which continued for 3 to 7 days.

RESULTS AND DISCUSSION

Refreezing analysis process of testing pile side surface

Readings at six subsequent dates are shown plotted in Figure 5. The readings indicate that the thawed active layer extended to about 0 to 3 m below ground surface. The ground temperature around test pile within 0 to 3 m under natural ground surface is not considered when the thawed frozen soil induced by construction refreezes. The reason is that the ground around test pile within 0 to 3 m under natural ground surface had been often excavated and exposed. The natural ground surface within 0 to 3 m also had no influences on the bearing capacity of test pile. The ground temperature around test pile within 0 to 3 m under natural ground surface has little significance for the bearing capacity of test pile. Therefore, the ground temperature around test pile only within 3 to 15.5 m under natural ground surface was considered.

As shown in Figure 2, the test pile is divided into two parts to analyze ground temperature below test pile top because the ultimate bearing capacity was related closely with the active depth of permafrost. The first part was marked as ① where it was -3 from -6 m below natural ground surface. The second part was marked as ② where it was beyond -6 m below natural ground surface. Figure 3 showed that ground temperature below the ground surface had been basically less than 0°C after November 1, 2001 at the depth of 0.5 and 1.0 m, respectively. But ground temperature below the natural ground surface had been under 0°C except within 0 to 3 m under natural ground surface.

According to the arrangement of reinforcement meter and the measured frequency, the following shaft force of pile formula was calculated by the pile concrete strain data.

$$Q_i = A_p \sigma(\varepsilon_i) \quad (1)$$

Where: Q_i is pile shaft force under the loading level I; A_p is the cross-sectional area of pile; ε_i is the pile concrete strain under the loading level i.

Pile side freezing force or pile side resistance of each layer soil is calculated by the following equation:

$$f_i = (Q_i - Q_{i-1}) / A_p \quad (2)$$

Pile tip resistance is calculated by the following equation:

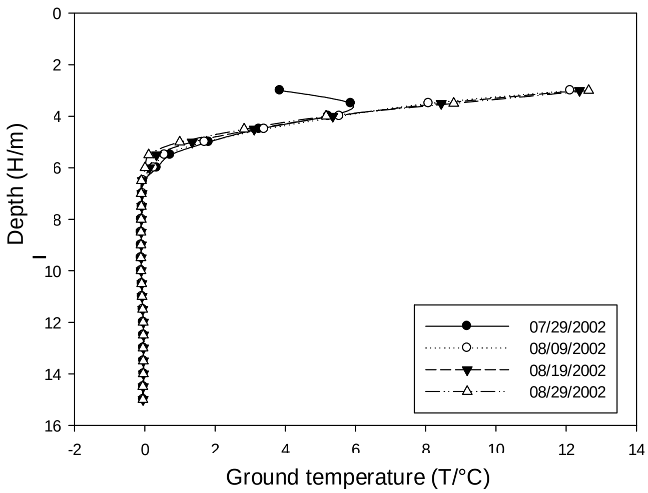


Figure 6. Ground temperature curve of pile's wall during loading.

$$Q_b = Q_{up} / A_p \quad (3)$$

Where: Q_{up} is pile tip resistance.

Figure 5 presented the curved temperature profile below this elevation which is typical of a late-winter temperature profile in a continuous permafrost zone. It can be seen that the ground temperature of all measured points around the test pile wall had a sharp increase due to absorption of concrete hydration heat and became higher than 0°C after the pile hole had been just poured into concrete which varied from 1 to 3°C on November 27, 2001. The measured maximum temperature even reached 13°C on the pile top because of the hydration heat of concrete releasing. Subsequently, the overall measured ground temperature around the test pile wall began to decrease and dropped to 2°C or so 3 days after the pile had been completed. The ground temperature only at a depth of 3.5 m under the pile top still dropped to -0.025°C 25 days after the pile had been completed and this provides an estimate of the average annual ground temperature at the site. The ground temperature of all measured points around the test pile wall was below 0°C 103 days after the pile had been completed on September 29, 2003. Bored-pile directly could be bonded with hole-wall in permafrost as long as the ground temperature around test pile was below 0°C. So the shear strength of the interface between pile and soil was enhanced. The bearing capacity of the pile can be improved. The bearing capacity of the pile had almost reached the maximum value at present and would be a stable state. The temperature below this elevation would not normally be expected to vary by more than 1°C or so throughout the year. The design poured depth of bored pile is from 3 m to 15.5 m under natural ground surface. The measured ground temperature 3 m below the natural ground surface would be analyzed carefully.

Besides, it is very essential for the bearing capacity of pile to analyze ground temperature change around pile when the thawed frozen soil induced by construction refreezes because the shear strength is a function of temperature. It demands more worthy attention especially during test pile loading. The measured ground temperature on the pile top was up to 13.2°C during loading of the test pile, as can be seen from the Figure 6. The ground temperature in the area of ① is always above 0°C, but the ground temperature in the area of ② is always below 0°C during loading of the test pile. And this is mainly caused by the warming climate and excavation of pile foundation pit of pile caps in bridge abutment. The measured ground temperature on the pile top had dropped to 3.85°C when loading of the test pile was completed. But the all measured ground temperature around the test pile wall were higher than before the test pile was loaded in despite being lower than 0°C. This was mainly caused by the external loading. In addition, permafrost is particularly insensitive to the outside temperature when the ground temperature is near 0°C.

The above studies show that the ground temperature around the test pile was higher than before test pile was not excavated because the ground was disturbed by the construction and the weather outside. After this period, it was difficult to recover the original ground temperature. The ground temperature in the area of ① is always above 0°C, but the ground temperature in the area of ② is always below 0°C during loading of the test pile. Therefore, there was no freezing force in the pile side surface in the area of ① but there was freezing force in the pile side surface in the area of ②.

Bearing characteristics of testing pile

Figure 7 presented the results for settlement on pile top against load on pile top for test pile. Steep drop P-S curve of test pile indicates that test pile is nearly close to pure friction pile because the soil on the end of the test pile is sandstone with lower strength and high compression, as shown in Figure 2. It is also seen from Figure 7 that the loading-settlement process of test pile can be divided into three stages based on the elastic-plastic deformation of permafrost around pile. They are respectively elastic (OA), elastic-plastic (AB) and plastic (BC) in Figure 7.

(Elastic stage OA (0 to 2400 kN)

As can be seen from Figure 6, the ground temperature where it was 3 to 6 m below test pile top is always above 0°C, but the ground temperature within 6 to 15.5 m below test pile top is always below 0°C at the beginning of test pile loading. And there was no freezing force on the pile side surface 3 to 6 m below test pile top but there was freezing force on the pile side surface within 6 to 15.5 m

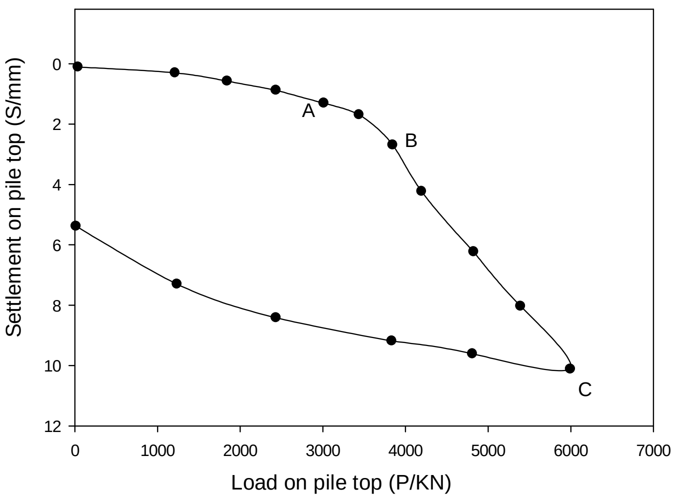


Figure 7. P-S curve of pile.

below test pile top. The pile length below 0°C is far more than over 0°C. Load ground transfer along the pile was undertaken by pile side resistance because freezing force on the pile side surface was far more than pile side resistance. With the pile loading increasing little by little, the pile side resistance was also increasing. At present, pile tip resistance also has a small increase. The ground temperature in the area of ① is always above 0°C during loading of the test pile. So there is no freezing force on the pile side surface in the area of ①. The ultimate value of pile side resistance is less than the ground temperature in the area of ① which is always below 0°C. A of P-S curve appears earlier than the ground temperature in the area of ① which is always below 0°C. The elastic deformation duration of soil around test pile is relatively short. However, both freezing force on the pile side surface and pile side resistance had always been less than the shear strength of ground within the static loading of 0 to 2400 kN. The permafrost did not appear as shear failure although the shear deformation zone of frozen soil around test piles expanded to the surrounding continuously.

Elastic-plastic AB (2400 to 3800 kN)

Ground around the test pile begin to appear as plastic deformation because pile side resistance in the area of ① began to be more than the shear strength of ground when the static loading was beyond 2400 kN. But both freezing force on the pile side surface and pile side resistance in the area of ② had been less than the shear strength of permafrost. So the permafrost in the area of ② did not appear as shear failure although the static loading had already reached 2400 kN. The shear deformation zone of frozen soil around test piles continued expanding to the surrounding continuously with the static loading. Pile side resistance begins to extend down from test pile top along the pile. Meanwhile, the plastic-deformation of the soil around test pile begins to expand downward permafrost gradually. Pile side resistance along the pile reaches the limit value when the plastic zone extends from the top of test pile to the end of the pile until the static loading reached 3800 kN. Pile side resistance was unable to withstand the increased loading again from test pile top. Both freezing force on the pile side surface and pile side resistance had been more than the shear strength of ground when the static loading reached 3800 kN. The pile tip resistance of test pile begins to bear the increased load again if the static loading continued increasing. Due to the large diameter and the short length of bored piles, and the low strength of sandstone at the pile tip, the curvature of the curve AB is relatively large and duration time of elastic-plastic deformation of the frozen soil around the pile is very short.

Plastic stage BC (3800 to 6000 kN)

The plastic deformation of permafrost around pile would persist for a long time due to the irreversible shear effect of particles and particle aggregating in permafrost causing visco-plastic flowing deformation. The sedimentation rate of test pile is relatively slow as shown from Figure 8. The settlement on the pile top have a sharp increase; the settlement rate is accelerated too, and the curves of S-lg t appear polyline and have a sharp increase in the slope of the curve when the load capacity reaches 6000 kN on the top of the pile. It can be speculated that 6000 kN is ultimate bearing load capacity of the test pile. Under this load, the total settlement of pile is 10.15 mm. The residual settlement of test pile is 5.53 mm after unloading. Rebounding rate of test pile was 58.48%. The load-settlement characters of test pile show a typical characteristic of a friction pile. The total settlement of the test pile in the elastic and elastic-plastic stages is 8.74 mm known from the load-settlement process of test pile. And the settlement in the plastic stage is 7.41 mm. So the deformation of permafrost around test pile is mostly plastic deformation from pile top to pile tip. The load-settlement characteristic of test pile also shows characteristics “L” of a typical friction pile. In addition, thawing and refreezing of permafrost around test pile are caused by cement hardening and its temperature after the pouring of concrete in permafrost regions.

In addition to the mortar infiltration into soil around test pile, test pile bonding directly with pile wall can improve the bearing capacity of pile greatly. However, it was affected by the excavation of foundation pit before loading on the test pile. There is only frozen force on the pile side surface in the area of ②. The bearing capacity of test pile has not been fully realized.

Therefore, the pile at this site appeared to be incapable

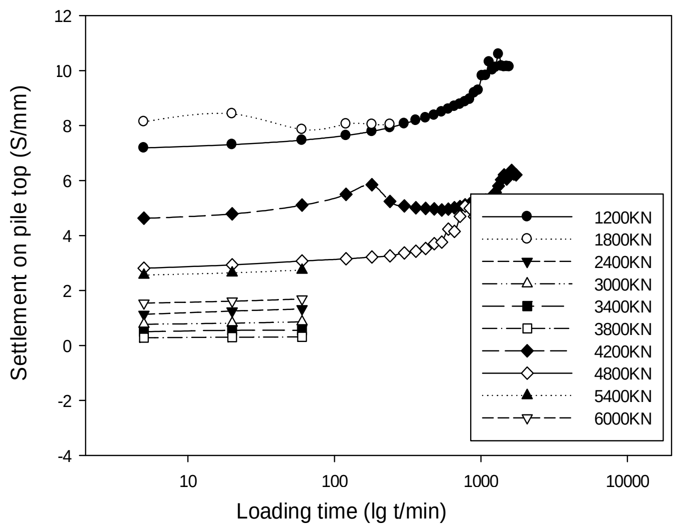


Figure 8. S-lgt curve of pile.

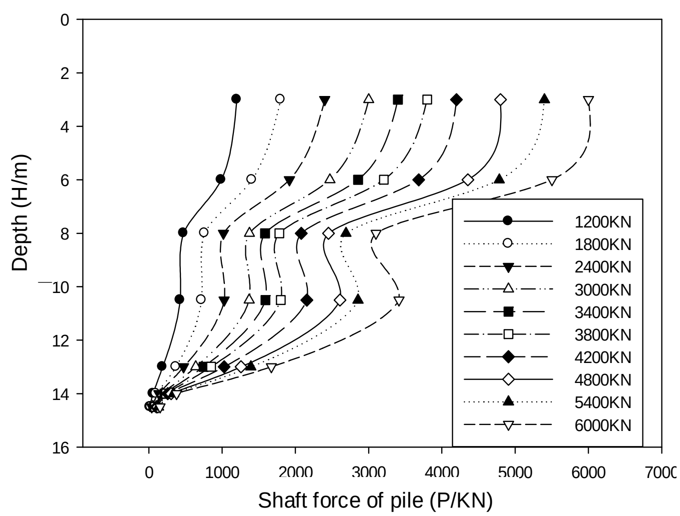


Figure 9. curve of pile shaft force with different load.

of supporting pile-shaft ultimate load in excess of about 6000 kN for the ground-temperature conditions prevalent during the test in the broadest sense. At a load level of 6000 kN, one test pile showed very high settlement rates immediately after load application. These pile capacities are quite low, even for piles in contact with icy permafrost. These findings prompted the designers to introduce other methods of foundation cooling in order to increase the allowable pile load capacities for the building. Allowable pile side stresses in permafrost would normally have been expected to be in the range of 407.19 kPa for these ground temperature conditions based on the above experience and theoretical considerations. The influence of rich-ice, and possibly also of pile installation method, have obviously introduced major reductions in pile capacity.

Load transfer characteristics of test pile side

Figure 9 presented curve of pile shaft force with different load. And curve of pile side resistance with different load is given in Figure 10. The pile shaft force and pile side resistance around test pile can be calculated according to the measuring values of steel instruments under different load conditions. Figure 9 shows that the pile shaft force along the test pile decays gradually under different loads. The pile tip resistance is very small and the increase of the pile tip resistance is slow relatively. It indicates that the vertical load on the pile top is balanced by the pile side resistance coming from the soil around test pile. With the increase of load on the top of pile, the attenuation trend of pile shaft force along test pile presents accelerating gradually. It is more obvious especially 0 to 10.5 m below the pile top. The attenuation trend of pile shaft force at the different depth is different even though under the same load. This may be related to the character of permafrost around test pile. For example, when the shaft force of test pile is loaded to 3200 kN, the attenuation trend of pile shaft force in strongly weathered marl layer 3 to 5 m below the top of test pile is significantly faster than intermediary weathered 7.5 to 10.5 m below the top of test pile. The attenuation trend of pile shaft force is relatively flat when load on the pile top is small. This is caused by the smaller deformation of the pile cross-section and pile side resistance not fully developing. There is a larger gradient change pile shaft force 3 to 5 m, 5 to 7.5 m and 7.5 to 10.5 m below the pile top, respectively. And the corresponding pile side resistance is larger, too. On the contrary, there is a smaller gradient change pile shaft force 0 to 3 m and 10.5 to 12.5 m below the pile top, respectively. And the corresponding pile side resistance is smaller, too.

It is seen from Figures 10 that the pile side resistance increases with the increasing of loading in the same soil. And there are limit value of pile side resistance within 3 to 7.5 m and 7.5 to 11 m below the pile top under every load level. The limit value of pile side resistance within 3 to 7.5 m and 7.5 to 11 m below the pile top were separately 378.39 and 407.19 kPa, respectively when loading reached 6000 kN. From the view of geological data, the geologies within 3 to 7.5 m and 7.5 to 11 m below the pile top were separately strongly weathered marl layer and intermediary weathered marl layers. According to the ground temperature at that time, the strongly weathered marl layer within 3 to 7.5 m below the pile top had so higher ground temperature that the pile side resistance was extremely small. The intermediary weathered marl layer within 7.5 to 11 m below the pile top had lower ground temperature. So the pile side resistance was extremely great. These should be related with the ground temperature.

Curve of the pile side resistance varied with load under

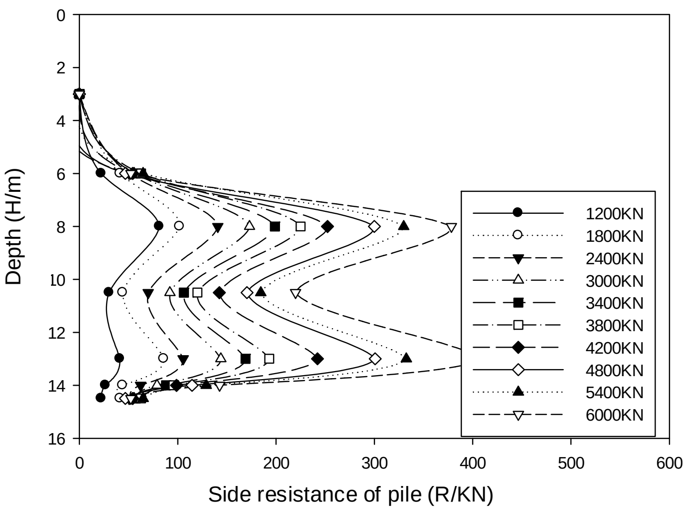


Figure 10. Curve of pile side resistance with different load.

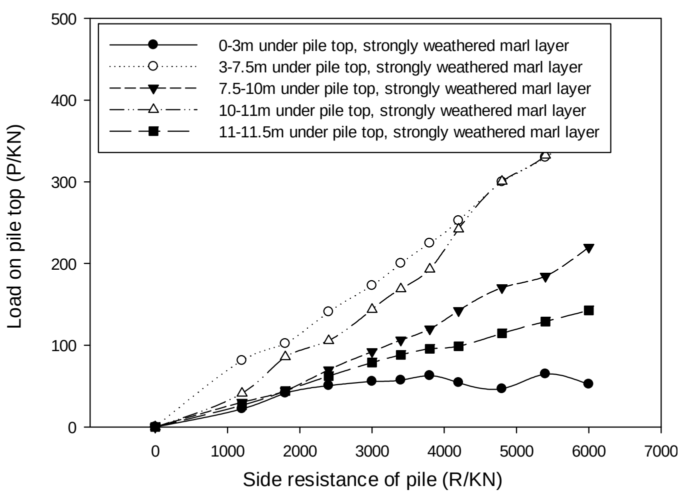


Figure 11. Pile side resistance curve of different soil layer with loading.

the different layers, as shown in Figure 11. Except for soil between within 0 to 3 m below the pile top, pile side resistance presented a linear increase with load on the pile top before loading reached the ultimate load 6000 kN. The increase of pile side resistance in the strongly weathered marl is the fastest within 3 to 5 m below the pile top. This may be related to the strongly weathered marl with a temperature always below 0°C. But the pile side resistance in strongly weathered marl layer had almost no any increasing within 0 to 3 m below the pile top. It indicated that the soil around test pile within 0 to 3 m below the pile top had reached the limit prior to other soil. This is because the strongly weathered marl within 0 to 3 m below the pile top had a temperature always over 0°C. In addition, pile tip resistance and loading sharing ratio of pile tip resistance increase with load on the pile top, as shown from Table 1. Load sharing ratio of pile tip resistance almost had no increase when the load increased from 4800 to 6000 kN, respectively. It also indicated that the load on the pile top is mainly borne by the pile side resistance.

Influencing factors analysis of testing pile bearing

It is not difficult to speculate from the above analysis that the primary unknown in this performance analysis involves the properties and behavior of the frozen-back permafrost around test pile. If competent bedrock is within practical piling distance of the ground surface then end-bearing pile is preferred. In fact, friction pile is preferred because competent weathered marl is within practical piling distance of the ground surface. The frozen soil around test pile will have adequate bearing capacity because of freezing force of pile side if the ground temperature below test pile top is always below 0°C. If this approach is either impractical or impossible then the piles must develop adequate bearing capacity in frozen or unfrozen soils. Permafrost is insensitive to the outside temperature when the ground temperature is near 0°C. Settlement and strength properties of warm frozen soils (greater than 0°C) are still poorly defined. Moreover, the thermal regime of such soils is in a delicate state of equilibrium. Therefore it is recommended that the pile foundation be determined only for the length of the pile bored in permafrost colder than 0°C. Consequently, in marginal permafrost areas where the ground temperature is warmer than 0°C, special precautions must be taken. If the permafrost is thaw-stable then the design may be based on the unfrozen soil properties. If the permafrost is thaw unstable then the soil may be pre-thawed and compacted. The warm frozen soil temperature must be lowered using artificial refrigeration to keep the stability of pile foundation in permafrost. Pile design is then identical to that for cold permafrost. For a friction pile the allowable pile load is determined from both settlement and strength considerations.

However, some studies only considered the ultimate bearing capacity of pile under disturbed permafrost not the original undisturbed state (Wang et al., 2013). It means that the ultimate bearing capacity of pile does not fully play although the pile has a high vertical bearing capacity and a small deformation under a non-fully refreezing condition. So the ultimate bearing capacity of pile must be considering a fully refreezing condition.

The allowable adfreeze strength may be estimated from Figures 9 and 10. It is emphasized that the allowable adfreeze strength must consider the soil condition at the pile interface during test pile loading. Thus, for the bored pile poured in winter, the moisture migration to the pile during freeze back will result in an ice lens at the pile-soil interface and therefore the allowable adfreeze strength should be equal to that of a

Table 1. Loading sharing ratio of pile tip resistance.

Load levels/kN	1200	1800	2400	3000	3400	3800	4200	4800	5400	6000
Pile tip resistance /kN	15.83	26.37	39.55	58	71.17	84.33	102.75	121.17	131.68	152.78
Load sharing ratio/%	1.3	1.5	1.6	1.9	2.0	2.2	2.4	2.5	2.4	2.5

pile in ice. Further, the adfreeze strength must be determined for the warmest soil conditions throughout the design life. In order to satisfy settlement criteria in ice-poor soils it is necessary to determine the flow law parameters. The approaches are almost certainly conservative but this should not preclude the application of a factor of safety. It is recommended that the safety factor be applied to the bored area. Its magnitude will depend upon the construction control and, more importantly, the accuracy of the soil property values used in the analysis. Finally it is noted that the net allowable structure load per pile is equal to the allowable axial pile load as determined from the down drag loads within the active layer. The in-situ ground temperature tests and static loading tests were used for the pile foundation design of the Qingshuihe Bridge located at the Qinghai-Tibet railway during its construction.

Conclusions

From the aforementioned results and analyses, we can find several significant conclusions for bearing characteristics and influencing factors analysis of bored pile under refreezing condition in permafrost region.

(1) It is as far as possible to avoid disturb permafrost by construction when the bored pile is constructed in the unstable underground thick ice layer of high temperature permafrost region. It is very difficult to recovery once permafrost is disturbed.
(2) The load-settlement process can be divided into three stages based on P-S curve of the field loading test. They are separately elastic, elastic-plastic and plastic. So the deformation development process of frozen soil of pile side and the interaction process between pile and pile side soil is analyzed.
(3) Pile tip resistance accounted for the entire load is just 2.5% when load on pile top reached 6000 kN. Pile tip resistance is not too important for the bearing capacity of test pile.
(4) The test result of shaft force and pile side resistance indicate that the attenuation characteristics of shaft force and distribution of pile side resistance along test pile is closely related to the load on the pile top, the properties of permafrost around test pile and ground temperature.

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Conflict of Interests

The authors have not declared any conflict of interests.

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