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A Field Study on the Freezing Characteristics of Freeze-Sealing Pipe Roof Used in Ultra-Shallow Buried Tunnel

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Featured Application: The freeze-sealing pipe roof (FSPR) can be applied to be the pre-supporting system in a shallow buried tunnel through water-rich strata or other tunnel constructions in which the disturbance of tunnel excavation must be strictly controlled.

Abstract: A new pre-supporting technology named the freeze-sealing pipe roof (FSPR) method was adopted in the construction of Gongbei tunnel (Zhuhai, China), a critical part of the Hong Kong–Zhuhai–Macau bridge (HZMB) project. The method combined pipe-roofing with artificial ground freezing (AGF). The pipe roof which included a number of large-diameter steel pipes was designed to play a primary role in load bearing, while the frozen wall between pipes was designed for water sealing. The refrigeration proceeded in two stages called the active freezing period and excavation period. This paper mainly focuses on the freezing characteristics of FSPR to explore how the frozen soil wall developed and changed over time during both periods based on field temperature data. The results show that the development of the frozen wall met the design requirements in fewer than 80 days of refrigeration considering the most unfavorable situation. The distribution of frozen soil along the entire tunnel was non-uniform. Frost heave and thaw weakening problems should be taken into account, since some of the bottom section of the frozen wall was more than 3 m. The frozen soil at the excavation side was visibly influenced by the replenishment of heat due to excavation, while the frozen soil outside the excavation face was much less influenced. The thermal effects of Hurricane Nicole on the frozen soil wall was also observed. The conclusions provide experience, reference, and guidance for the development of similar projects in the future.

Keywords: shallow buried tunnel; freeze-sealing pipe roof; ground freezing; freezing characteristics; field study; Hong Kong–Zhuhai–Macau bridge

1. Introduction

The pipe-roofing method is widely used in geotechnical engineering [1–5] since its appearance in Japan in 1971 [6]. This method is a kind of pre-supporting technology that can effectively control the disturbance of tunnel excavation, especially in shallow buried tunnel construction. For the general pipe roof, the pipe line is nearly straight and the water sealing problem between pipes is commonly solved by usage of a pipe lock combined with grouting. However, during the construction of Gongbei tunnel (Zhuhai, China), a critical part of the Hong Kong–Zhuhai–Macau bridge (HZMB) project, a curved pipe roof was used due to objective conditions [7]. Considering the complex geological conditions in the excavation area, such as water-rich strata and stagger distribution of soft sandy and silty clay, the above conventional method using a pipe lock is unable to ensure the water sealing

performance between the pipes. Thus, a new pre-supporting system for shallow buried tunnel, called “freeze-sealing pipe roof” (FSPR) [8], was adopted to solve this problem.

As a combination of the pipe-roofing method and the artificial ground freezing (AGF) method, the FSPR system consists of a steel pipe roof and a frozen soil wall between the pipes. The pipe roof is designed to play a primary role in load bearing, while the frozen wall is designed for water sealing. A similar method called pipe-shed freezing was adopted in Japan, Germany, and China before the FSPR was proposed [9]. In that method, the frozen soil wall mainly played a role in load bearing, which is essentially different from the FSPR method being discussed. Moriuchi and other researchers carried out experimental and theoretical research on the mechanical characteristics and the interaction between them [10–12]. The freezing scheme and the related thermal properties of the FSPR were studied by Hu’s research group through analytical solutions [13], numerical simulations [9,14], a large-scale physical model test [15,16], and an in situ test [17–19]. A laboratory test was also conducted by the group to study the mechanical behavior of the FSPR system [20].

Based on the field temperature data of the project, the freezing characteristics of the FSPR system used in Gongbei tunnel in different freezing periods were analyzed in this paper. The time-dependent changes of temperature of the frozen soil between two adjacent pipes are presented. The results verify the safety of the FSPR system and provide reference and guidance for similar projects in the future.

2. The FSPR System and Temperature Measuring Method

2.1. The Pipe Roof of FSPR

Gongbei tunnel is a critical part for the Zhuhai link of the Hong Kong–Zhuhai–Macao bridge and it crosses the prosperous Gongbei port, which has the highest people flux in the world. It is designed as a double-decker integral tunnel with a large section constructed by the shallow tunneling method. The total length of the tunnel is 255 m and the excavation section is about 345 m² (20 m high and 19 m wide). The tunnel is buried in water-rich strata within soft sandy and silty clay, about 4 to 5 m below the ground surface.

Figure 1 shows the cross-section of the tunnel, and the five excavation layers are also illustrated in the figure, which are marked with letters A to E and divided into 14 small sections. It is shown that 36 steel pipes with a diameter of 1.62 m are arranged along the quasi-circular cross-section of the tunnel to form the pipe roof. The odd-numbered roof pipes are shifted inward by about 30 cm along the radial direction of the tunnel and the space between two adjacent pipes is about 35 cm.

2.2. The Freezing Scheme of FSPR

As mentioned above, for the FSPR system, the pipe roof mainly serves for load bearing, and the frozen soil wall is mainly used for water sealing. Therefore, the freezing scheme is obviously different from the general freezing method. It is necessary to consider both the requirements of water sealing between pipes and the control of surface settlement and uplift. That is to say, the minimum thickness of the frozen soil wall must meet the water sealing requirements; the maximum thickness of the frozen soil wall should satisfy the requirement of surface deformation control due to the frost heave. Hu and his group proposed the three concepts of freezing philosophy of FSPR, including rapid freezing, resistance against thermal weakening, and limitation of frost heave [21].

A new freezing scheme including three types of freezing tubes was established based on the concepts. It is impractical to embed the freezing tubes in soil because of the curvilinear tunnel axis; thus, the tubes were installed in the steel pipes. The layout of the freezing tubes in each steel pipe is shown in Figure 2. There are three types of freezing tubes: master freezing tubes, limiting freezing tubes, and enhancing freezing tubes. In each of the odd pipes, two circular master freezing tubes with a diameter of 133 mm were installed at the middle edge position as the major cold source, while one circular limiting freezing tube with a diameter of 159 mm was arranged at the middle upper position to control the frost heave by recycling brine with relatively high temperature when necessary. Furthermore, in each of the even pipes,

two fan-shaped enhancing freezing tubes were welded at the inner surface for freezing reinforcement. The odd-numbered pipes would be filled with concrete as soon as the corresponding freezing tubes are installed, while the even-numbered pipes would remain hollow until excavation. The design thickness of frozen soil wall was 2 to 2.6 m.

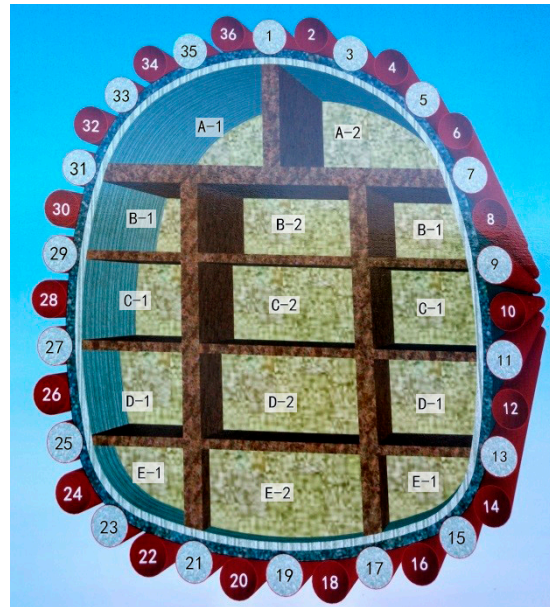


Figure 1. Schematic of the tunnel section.

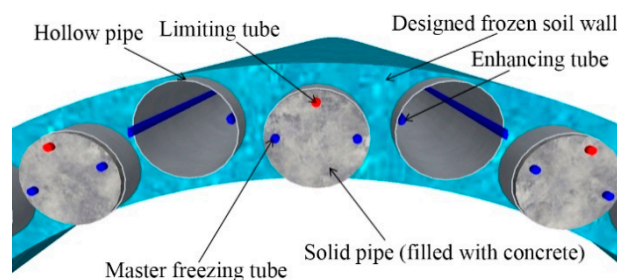


Figure 2. Schematic of the freezing scheme.

2.3. Temperature Measuring Method

In this project, a remote monitoring system developed at Tongji University was used for temperature monitoring, including the temperatures of the frozen soil and the brine freezing loop. This remote monitoring method, based on a 1-Wire (Maxim Integrated Products, San Jose, CA, USA) bus system, makes automatic and continuous computer monitoring possible and contributes to observing the development of the frozen soil wall and judging the freezing effect of FSPR. More details about the monitoring system can be found in Reference [22].

For the monitoring of soil temperature, 32 measuring faces were arranged longitudinally along the tunnel, which were numbered as 1 to 32 from east to west. The distance between two measuring faces was about 8 m. A typical measuring face N is shown in Figure 3. The temperature measurement points of each measuring surface were classified as the wall temperature measuring point and the soil temperature measuring point. The latter was installed through the temperature measuring holes, which were drilled from inside the steel pipes. Furthermore, 3–7 soil temperature measuring points were arranged in each hole to determine the development and changes of the frozen soil wall. The naming conventions of measuring points were as follows: D + steel pipe number – measuring face number – S/B + measuring point number (where S means the soil temperature measuring point, and B means

the wall temperature measuring point). For example, D06-15-S3 represents the third measuring point installed from the sixth steel pipe at the 15th measuring face.

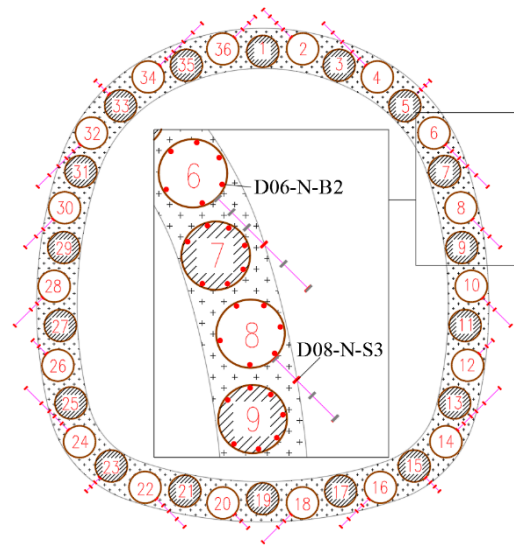


Figure 3. Schematic of a typical measuring face N.

The measured temperature data of different stages and multiple areas are analyzed later. Figure 4 is a schematic of the selected area and measuring points. The blue boxed area represents the selected area for analysis during the active freezing period, and the red one is that for the excavation period.

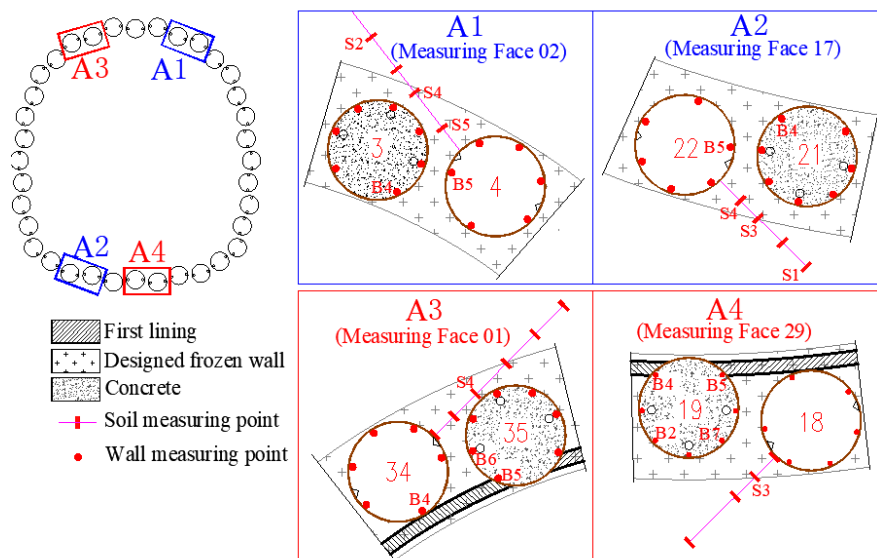


Figure 4. The analyzed areas and measuring points.

3. Results and Discussion

The whole construction can be separated into two stages: active freezing period (this period starts from the beginning of the brine circulation and ends before excavation) and excavation period. Specifically speaking, the active freezing period started in late February 2016, when the brine circulation at different freezing areas was getting started. A trial excavation was carried out from 19 June 2016, and the official excavation was started on 20 August 2016. The temperatures between steel pipes measured at both periods were analyzed and cloud charts of the temperature field before excavation were presented.

3.1. Active Freezing Period

Firstly, the temperatures of the measuring points in area A1 of Figure 4 were analyzed. The area was located at the top of the tunnel section, which was just 5 to 6 m below the ground surface. Moreover, the longitudinal measuring face 02 was also near the east working shaft. Therefore, this area belonged to the weakest area of the entire frozen curtain where thermal disturbance was fairly strong.

Figure 5 shows the temperature variation with time of the measuring points at measuring face 02 between the third and fourth pipe during the positive freezing period. It can be seen from the figure that, with the opening of the master freezing tubes in late February, the temperatures of the soil measuring points and the wall measuring points decreased, except for measuring point D04-02-S2 which was far from the freezing tube.

When the enhancing freezing tubes were opened on 11 March, the temperature of each measuring point decreased significantly, until 12 May when the temperature of D04-02-S4, which was near the design boundary of the frozen soil curtain (see Figure 4, A1 area, enlarged view), dropped to the freezing point of the frozen soil. This indicated that, after about 75 days of freezing, the development of frozen soil between the third and fourth pipes basically met the design requirements, and the water resistance performance between the pipes was initially satisfied. Since then, due to the maintenance work of the refrigeration system, the temperature of the brine was raised, and the temperature drop of each measuring point slowed down or even started rising. After the salt water temperature was lowered again on 1 June, the temperature of each measuring point began decreasing again. Then, due to the influence of trial excavation (see the next section), the temperature of measuring points D04-02-S4 and D04-02-S5 had a relatively obvious rise, while the temperature of measuring point D04-02-B5 was basically unaffected because it was much closer to the enhancing tube than others.

It is worth mentioning that, on 1 August, Hurricane Nicole struck; thus, the refrigeration system was forced to close, and the brine circulation stopped. The temperature of each measuring point then increased, especially the temperature of measuring point D04-02-B5, which obviously increased. When the hurricane was over and the brine circulation was restored, the temperature of the relevant measuring points dropped rapidly until official excavation began on 20 August.

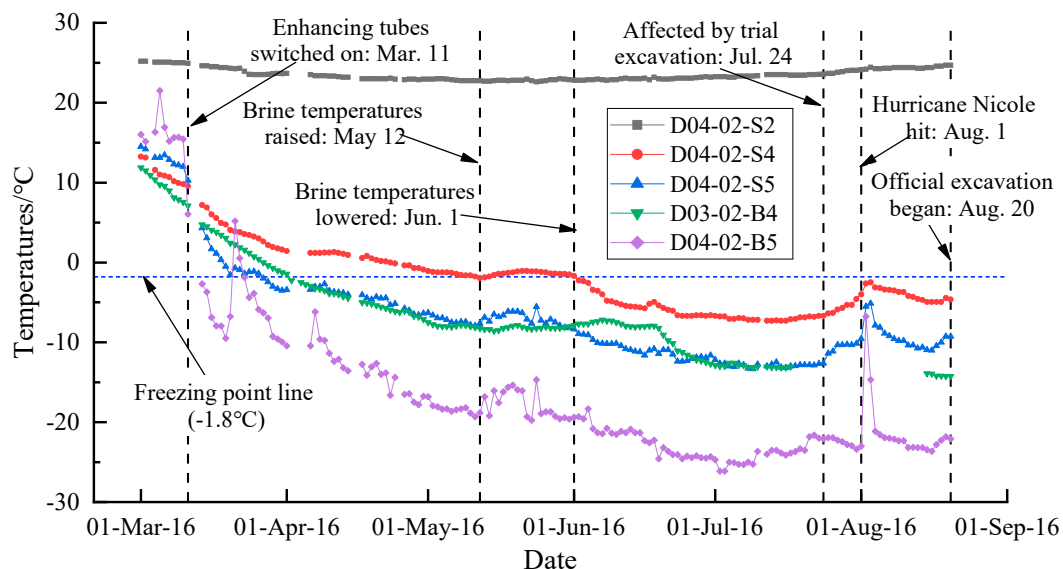


Figure 5. Temperature variation with time of measuring points at area A1 (see Figure 4).

From the perspective of the whole process of the active freezing period, the frozen soil curtain developed to the design thickness after more than two months of refrigeration, despite being affected by the trial excavation and hurricane. The temperature of the relevant measuring points was basically kept below the freezing point temperature, and the thickness of the frozen soil at area A1 was

able to satisfy the design requirements. This section is an analysis of the temperature variation of the measuring points in area A2 of Figure 4. The area was located in the lower part of the tunnel section, and, in the longitudinal direction, the measuring face 17 was located in the middle of the tunnel. Therefore, the A2 area was less disturbed by external heat and the development condition of the frozen soil was good.

Figure 6 shows the temperature variation with time of the measuring points at measuring face 17 between the 21st and 22nd pipes during the positive freezing period. The temperature change trend of each measuring point in the figure was similar to that of Figure 5 as a whole. The difference was that, despite measuring point D22-17-S1 being far away from freezing pipes, the temperature of the point decreased as the brine circulated and finally fell below freezing point. The spot of measuring point D22-17-S1 was about 0.8 m away from the boundary of the designed frozen wall, which means that the frozen wall in this area was beyond 3 m. This phenomenon is consistent with the above judgment that the A2 area was less thermally disturbed. In addition, the temperature of measuring point D22-17-S3 near the design boundary of the frozen soil curtain dropped to the freezing point of the frozen soil around 2 May, which was about half a month earlier than the A1 area. Moreover, after the brine temperature was raised around 16 May, the temperature of each measuring points except D21-17-B4 kept decreasing but slowed down slightly. On 1 June, after the brine temperature was lowered again, the temperature of each measuring point began falling again until the official excavation began around 20 August. Since the area of the trial excavation (see Figure 1; areas A1 and A2) was located in the upper part of the tunnel, it had little effect on the formation and development of the frozen curtain in area A2. The impact of Hurricane Nicole on 1 August was not significant.

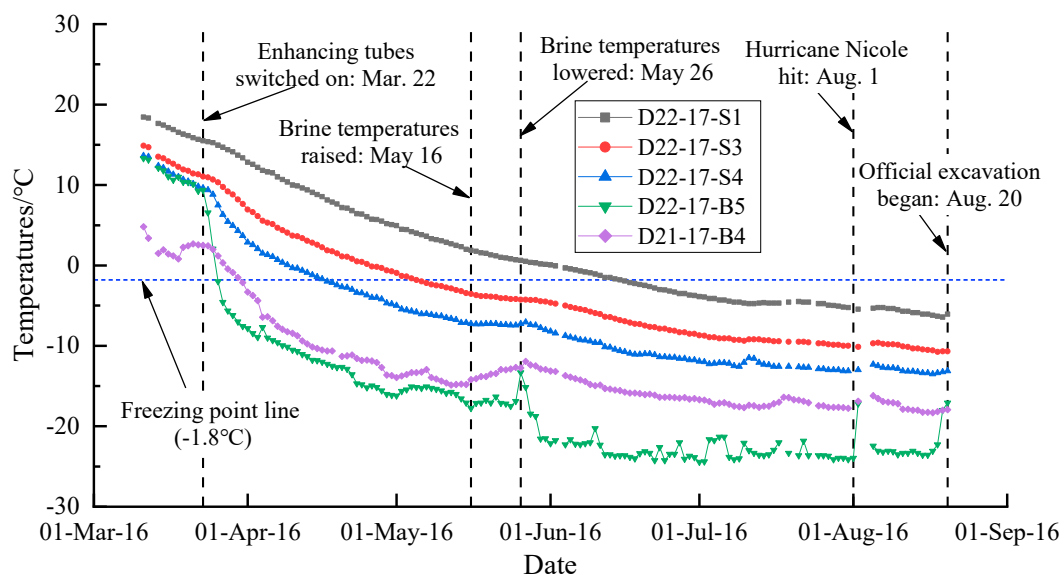


Figure 6. Temperature variation with time of measuring points at area A2 (see Figure 4).

Figure 7 represents two cloud charts of the temperature field of the two representative measuring faces 02 and 20, which were obtained using the interpolation method according to the temperature of the measuring points just before excavation. The charts also indicate that the distribution of frozen soil in the entire section was not uniform. Subjected to thermal disturbance of the ground surface, the top of the frozen soil curtain was thinner, while the lower part of the frozen soil curtain was thicker; the frozen soil curtains at the ends of the tunnel were thinner, while the middle part was thicker. However, whether it was at the upper part or the lower part, the inter-tube frozen soil could develop to the design requirements during the active freezing period, such that the freezing effect of FSPR remained satisfactory.

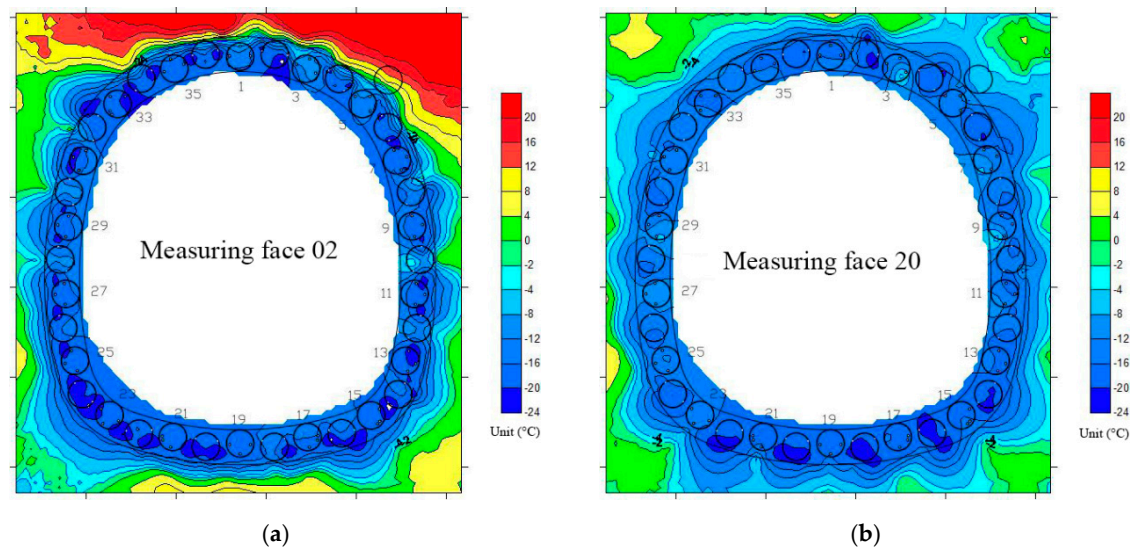


Figure 7. Cloud charts of the temperature field of typical measuring faces: (a) measuring face 02; (b) measuring face 20.

3.2. Excavation Period

The excavation of the Gongbei tunnel is a continuous and long process considering its short length. The bench cut method was adopted because of the huge excavation section. A trial excavation was conducted from 19 June, and lasted about one month. The official excavation started from 20 August 2016 and ended on 10 April 2017. The excavation speed was relatively slow and the process lasted a long time. During the excavation, some frozen soil was dug out, and the “thermal attack” from the air at the excavation face of the frozen soil was much fiercer than before. It had a negative effect on the freezing effect of the FSPF. Moreover, various excavation steps such as welding and lining also resulted in a non-negligible thermal disturbance of the frozen soil. Two typical areas (A3 and A4 in Figure 4) were selected to study the temperature variation near the boundary of the excavation face. Figures 8 and 9 show how the temperature at the selected areas changed over time during the excavation period.

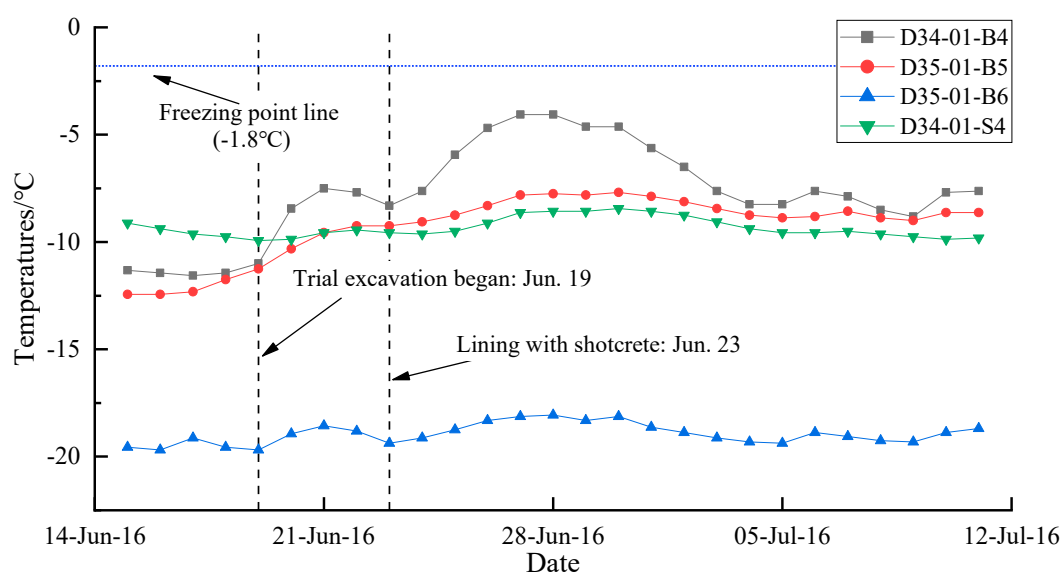


Figure 8. Temperature variation with time of measuring points at area A3 (see Figure 4).

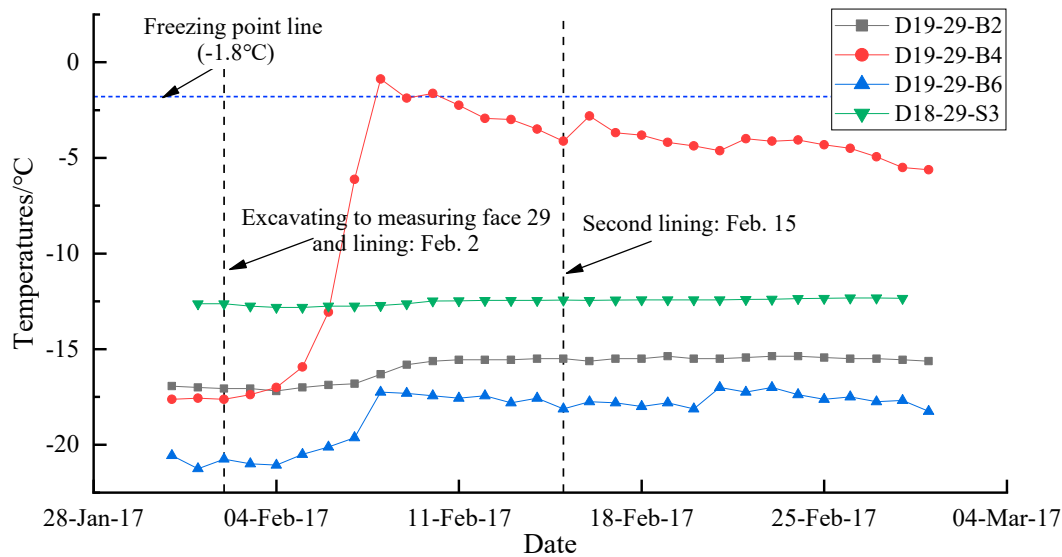


Figure 9. Temperature variation with time of measuring points at area A4 (see Figure 4).

Figure 8 shows the temperature variation of the measuring points with time at measuring face 01 between the 34th and 35th pipes (see Figure 4, A3 area) during the trial excavation, and it can be seen that, since the trial excavation started from 19 June, the temperature of measuring point D35-01-B4 increased slightly from -12°C . After shotcrete lining, its temperature rose to the highest value of about -8°C around 28 June, then decreased steadily, and finally wobbled around -8°C . The remaining measuring points were slightly affected by the trial excavation. From the enlarged view of A3 in Figure 4, the B4 and B5 measurement points were closer to the excavation surface than other measuring points. The largest temperature rise of point D34-01-B4 was about 8°C . Their temperature was lower than the freezing point of the frozen soil, indicating that the frozen soil between the 34th and 35th pipes existed during the trial excavation. This is because the construction progress during trial excavation was slower than that of official excavation, and the cut face was small. Furthermore, the excavation construction control was strict, and there was basically no over-excavation; thus, the corresponding air thermal erosion and construction thermal disturbances were small.

Figure 9 shows the temperature change of the measuring point with time between the 18th and 19th pipes (A4 area in Figure 4) during the official excavation. It can be seen that the temperature of measuring point D19-29-B4 was most affected by the excavation, and the maximum temperature rose to about 0°C . Its total temperature increase was about 16°C , 8°C higher than point D34-01-B4 in Figure 8. The temperature change of D19-29-B5 was almost the same as that of B4; thus, it is not shown in the figure. Combined with the enlarged view of A4 area in Figure 4, it is known that B4 and B5 were located on the excavation boundary. Thus, the part of the pipe near these points will be exposed to the air if there exists over-excavation. This is why the temperature of the B4 and B5 measuring points rose sharply during the excavation, even exceeding the melting point of the frozen soil. The excavation had limited effects on the remaining measuring points. The temperature rise of measuring point B6 did not exceed 5°C , and it could maintain -15°C or below even at its highest value. The influence of the excavation on the frozen soil close to measuring point S3 was almost negligible.

Most of the temperature changes of measuring points between the pipes were similar to those in Figure 9. The measuring point closest to the excavation surface was considerably affected, but the points farther away were almost unaffected. The thickness of the frozen soil tended to decrease to less than 20 cm due to excavation thermal disturbances. Considering that the whole thickness of the frozen wall was 2 m or more, the effect of the 20-cm reduction on the frozen wall between steel pipes was very limited. Therefore, the freezing effect of the FSPR during excavation was also satisfactory.

4. Conclusions

The freezing effect of the FSPR adopted in the construction of Gongbei tunnel was analyzed in this paper based on the field temperature data. The results showed that FSPR maintained satisfactory water resistance performance during both the positive freezing period and excavation period. The following conclusions about freezing characteristics of FSPR can be drawn:

For the positive freezing period,

1. The development of frozen wall met the design requirements in less than 80 days of freezing considering the most unfavorable situation.
2. The distribution of frozen soil along the entire tunnel is non-uniform. Some of the bottom section of the frozen wall is more than 3 m, which may cause frost heave and thaw weakening problems.
3. Extreme weather like hurricanes only has a limited effect on the top of the frozen soil wall. The safety of the construction work was under control.

For the excavation period,

1. The strict control of over-excavation was helpful in reducing the thermal disturbances during excavation. There was an 8 °C difference of the largest temperature rise between the trial excavation and the official excavation. The “thermal attack” from the air at the excavation face on the frozen soil was much fiercer than before when the near frozen soil was dug out.
2. The frozen soil at the excavation side was visibly influenced by the replenishment of heat due to excavation. The thickness reduction of the frozen soil due to excavation thermal disturbances was less than 20 cm, whereas the frozen soil at the outside of the excavation face was much less influenced.

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