Temperature and Strain Monitoring Analysis of Self-Compacting Concrete Bridge Decks with Mechanized Sand

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Abstract: The temperature control of self-compacting concrete with machine-made sand is an important research element related to the safety of actual projects. A reasonable and feasible temperature and strain monitoring scheme was designed for the bridge abutment project of Honghezhou Special Bridge in Yunnan Province, and the monitoring results of the concrete abutment on site were analyzed. The results show that the temperature of self-compacting concrete with mechanism sand has the characteristics of large temperature rise and rapid cooling compared with ordinary concrete, and the concrete surface layer is prone to cracking from 1.5d to 4d of age, and the concrete foundation part is prone to cracking at about 9d of age. It can provide reference for the temperature crack prevention and control of similar mechanism sand self-compacting concrete projects.

1. Introduction
Mechanized sand self-compacting concrete has the advantages of local material extraction, cost saving and environmental protection, and is widely used in some complex and special projects, which is a trend of sustainable development in the concrete industry[1]. In practical engineering, it often faces the problem of temperature cracks due to the heat of hydration of cement, which seriously affects the safety of the project, and the high content of stone powder and the large amount of cementitious materials in the mechanism sand self-compacting concrete play a certain role in promoting the heat of hydration of cement, which makes it more likely to produce temperature cracks in the early stage of concrete. Therefore, it is important to study the temperature and strain changes of self-consolidated mass concrete of mechanism sand for crack prevention and control[2].

In this paper, a reasonable and feasible temperature and strain monitoring scheme is designed with the background of Yunnan Honghezhou special bridge project, and the monitoring results of the bridge abutment concrete on site are analyzed to obtain the temperature and strain change law of the bridge abutment concrete and the temperature characteristics of the mechanism sand self-compacting concrete, which can provide reference for the crack prevention and control of similar projects.

2. Project Overview
Yunnan Honghezhou Special Bridge is located in the southwest of Yunnan Province, which is a mountainous plateau terrain with scarce natural river sand resources and abundant rock resources[4].
Combined with the actual situation of the project, the 0# abutment of the approach bridge was cast with self-compacting concrete of mechanism sand. The 0# abutment of the approach bridge is 25.48m long and has a symmetrical width structure of left and right, mainly consisting of a 1.8m wide and 1.7m high abutment cap, ear walls distributed at both ends of the abutment and 1.6m diameter piers.

The 0# abutment of the approach bridge adopts the sequence of using C30 strength ordinary concrete for the structure other than the left abutment cap first, and then using C30 strength mechanism sand self-compacting concrete for the left abutment cap structure after 30d of casting, and the construction drawing of 0# abutment casting is shown in Figure 2. The mechanism sand of the abutment is prepared from the mountain stone produced by the tunnel mining near the project, the main component is limestone, the stone powder content is 11%, and the mechanism sand self-compacting concrete pouring formula is shown in Table 1.

|           | Cement P·O 42.5 | Fly ash Grade II | Mechanized sand | Coarse Aggregate 4.75mm-9.5mm | 9.5mm-19mm | Water | Water Reducer |
|-----------|-----------------|------------------|-----------------|-------------------------------|-------------|-------|--------------|
| Quantity  | 402             | 77               | 826             | 486                           | 324         | 211   | 6.71         |

3. Bridge platform measurement point arrangement

In order to obtain the early temperature and strain distribution and change law of the bridge abutment concrete, it is necessary to select a reasonable location of the measurement point for monitoring. Combined with the structure shape and reinforcement of the abutment cap as shown in Figure 3, two vertical sections in the middle position and pier position were selected as the monitoring surface for temperature and strain monitoring, and the specific positions of the two sections in the abutment are shown in Figure 1. In each monitoring section, 15 measurement points were selected to determine the temperature and strain data in different directions of the section, and a temperature sensor was arranged.
in the middle layer of the pier section at 5 cm from the surface and around the abutment to determine the concrete surface temperature (ST) and environment temperature (ET) data, respectively, and the specific location of each measurement point is shown in Figure 4. Considering the feasibility and economic applicability of field operation, BGK2100 vibro-strain gauge was selected to determine the temperature and strain data at the center of the two sections, and PT100 platinum resistance temperature sensor was selected to determine the temperature data at the rest of the measurement points, and the respective use of the two sensors is shown in Figure 5.

![Table cap elevation reinforcement drawing](image1)

(a) Table cap elevation reinforcement drawing

![Table cap plan reinforcement drawing](image2)

(b) Table cap plan reinforcement drawing

Figure 3. Table cap reinforcement diagram

![Pier section measurement point layout](image3)

(a) Pier section measurement point layout (unit: cm)

![Intermediate section measurement point layout](image4)

(b) Intermediate section measurement point layout (unit: cm)

Figure 4. Arrangement of temperature measurement points of the bridge deck section (unit: cm)
4. Analysis of bridge deck temperature and strain monitoring results

After reasonably selecting the abutment temperature and strain monitoring points, arranging the corresponding monitoring elements at the measurement points, and conducting on-site monitoring for 19 d, the abutment temperature and strain variation data were obtained. In this section, the monitoring results are analyzed to obtain the early temperature and strain distribution and change law of the concrete abutment, and to provide reference for crack prevention and control.

4.1 Analysis of the overall change in temperature of the bridge deck section

The overall temperature changes of pier section and intermediate section within 19 days after the completion of concrete placement on the bridge abutment are shown in Figure 6. As can be seen from the figure, the time course of the two sections of the temperature change pattern is basically the same, both first sharp rise, reach the peak temperature after a rapid decline, after the cooling rate slowed down, gradually converge to the ambient temperature. Both sections are the highest temperature in the middle, the bottom layer is second and the top layer is the lowest temperature. This is due to the top contact with the outside air, the fastest heat dissipation, the bottom layer contact with the ground and concrete, heat dissipation is slower and the middle layer location of the heat of hydration is larger. Before the age of 5d, both cross-sectional height directions show the law that the middle layer temperature is greater than the bottom layer temperature, and the bottom layer temperature is greater than the top layer temperature. After reaching the temperature peak at the age of 1.5d, the cooling rate of the middle layer temperature is greater than the cooling rate of the bottom layer and the top layer, which is because the temperature of the middle layer is higher, and the difference between the temperature gradient and the outside world is larger, and more heat is transferred within the same time, and the cooling rate is faster. All layers in the thickness direction of the two cross-sections show the pattern that the temperature of the central layer is greater than the temperature of the sub-outer layer, and the temperature of the sub-outer layer is greater than the temperature of the outer layer. The temperature gradient between the center layer of the cross-section and the sub-outer layer is smaller than the temperature gradient between the sub-outer layer and the outer layer, indicating that the heat exchange rate between the concrete and the outside air is greater than the heat exchange rate between the center layer and the sub-outer layer inside the concrete, and the temperature stress in the outer layer of the concrete is greater and most likely to produce temperature cracks. The overall temperature of the two cross-sections basically converged with the ambient temperature at the age of 18d, and the temperature changes in the later period were mainly influenced by the external environment.

From Figure 6, the overall temperature change curve of the pier section and the middle section and the corresponding distribution of measurement points, the temperature index of the two sections can be obtained as shown in Table 2. 36h after the completion of the abutment concrete, the pier section and the middle section reached the temperature peak of 70.8°C and 72.8°C respectively at the center, and
the temperature peak lasted about 4h, after which the temperature began to cool down faster from 2d to 4d of the age, and the maximum cooling rate was 8.2°C/d and 9.2°C/d respectively, which was much larger than the "Standard for construction of mass concrete" (hereinafter referred to as "Standard") maximum The cooling rate does not exceed the requirement of 2.0°C/d. No obvious cracks were found at the site, indicating that the mechanism sand self-compacting concrete has the characteristics of rapid cooling, and good insulation measures are needed for the bridge deck concrete during the age period of 2d to 4d[5]. The maximum temperature rise of the pier section and the middle section are 40.4°C and 42.1°C respectively, which meet the requirement of the Standard that the maximum temperature rise should not be greater than 50°C. The temperature difference between the corresponding positions of the two sections is not large and the overall pattern is consistent, indicating that the temperature change of the bridge deck concrete in the longitudinal direction is small, and in the length direction is susceptible to expansion to produce tensile stresses, resulting in concrete cracking.

![Temperature curve](image)

(a) Course curve of temperature change of pier section and distribution of measurement points

![Temperature curve](image)

(b) Course curve of temperature change of intermediate section and distribution of measurement points

Figure 6. Time course curves of temperature change in pier section and intermediate section and distribution of corresponding measurement points

| Table 2. Comparison of temperature index of pier section and intermediate section |
|-------------------------------------------|-----------------|-----------------|
| Monitoring cross section               | Pier section   | Intermediate section |
|-------------------------------------------|-----------------|-----------------|
| Maximum temperature/°C                  | 70.8            | 72.8            |
| Maximum temperature occurrence time/h   | 36              | 36              |
| Maximum temperature duration/h           | 4               | 4               |
| In-mold temperature/°C                  | 30.4            | 30.7            |
| Maximum temperature rise/°C              | 40.4            | 42.1            |
| Maximum cooling rate/°C/d               | 8.2             | 9.2             |
4.2 Analysis of the temperature difference between the inside and outside of the surface
When the abutment concrete has a large temperature difference between inside and outside, it will lead to uncoordinated concrete deformation. These uncoordinated through the internal constraints of the members to form temperature stress. When the temperature stress is greater than the same period of concrete tensile strength, the concrete will produce temperature cracks. So the analysis of the abutment concrete temperature difference value changes, the temperature crack judgment and prevention is of great significance[6].

The center of the concrete section of the abutment has the highest temperature, and the difference between the center of the section and the surface temperature is the maximum temperature difference in the surface, which can best reflect the temperature change law of concrete, so this paper mainly analyzes the maximum temperature difference in the surface of the section. Figure 7(a) shows the time course curve of the maximum temperature difference between the two sections, and it can be seen from the figure that the maximum temperature difference between the two sections is basically the same, which again proves that the temperature of concrete in the longitudinal direction is basically the same. The temperature difference between the two sections reaches the peak at the age of 2d, and then decreases rapidly from the age of 2d to the age of 5d, and decreases slowly after 10d. The maximum temperature difference in the pier section is 28.1℃, and the maximum temperature difference in the intermediate section is 30.0℃. During the period from 1.5d to 2.5d, the temperature difference in the two sections does not meet the requirement of "Standard" that the maximum temperature difference in the surface should not be more than 25℃.

As the surface concrete temperature of the abutment is influenced by the environment and the temperature difference between the two sections in the longitudinal direction is not large, combined with the field test conditions, the side of the pier section was selected for the test to monitor the surface temperature, the change in the temperature difference between the surface and the outside is shown in Figure 7 (b). As can be seen from the figure, the surface temperature difference reached a peak of 22.6℃ at the age of 2d. During the age of 1d to 2d, the surface temperature difference does not meet the "standard" maximum surface temperature difference should not be greater than 20℃, during which the concrete surface layer is prone to cracking, the need to do a good job of insulation and control measures.

(a) Time course curve of the maximum inside surface temperature difference change  (b) Time course curve of temperature difference change outside the surface

Figure 7. Time course curves of temperature difference between inside and outside of the surface

4.3 Analysis of strain variation in bridge deck sections
Considering the operational feasibility and economic feasibility of the field test, this test monitored the early strain changes in the center layer of the two monitoring surfaces of the abutment, and the monitoring results are shown in Figure 8. From Figure 8, it can be obtained that:
(1) The compressive strain appears first at all measurement points, and increases gradually with the warming of the concrete by the heat of hydration, reaching a maximum of 180 με and 220 με at about 1.5 d for the pier and column sections and the middle section, respectively, and decreases gradually after 2 d. At 9.5d of age, the tensile strain is generated at measurement point ZZ-1, and reaches a maximum of 13.38 με at about 14.5 d of age. After 9 d, the strain variation decreases and gradually stabilizes, and fluctuates with the change of external environment.

(2) Due to the arrangement of vertical reinforcement in the pier section, the thermal expansion coefficient is larger, which makes the tensile strain in the height direction of the pier section larger than the middle section, and the coordination of concrete deformation shows that the longitudinal compressive strain in the bottom concrete of the pier section is larger than the middle section, while the middle and upper concrete of the pier section are slightly smaller than the middle section due to the influence of the larger expansion in the height direction of the bottom concrete at the same time, so the tensile strain of the pier section The compressive strains in the middle and upper layers of concrete in the pier column section are slightly smaller than those in the middle section.

(3) The longitudinal compressive strain of the concrete of the abutment is larger during the age period 1.5d to 3d, indicating that the tensile strain in the thickness and height directions is larger and prone to the risk of cracking. At around 9d of age, the concrete is subjected to external restraint of the foundation, which generates certain tensile stresses and a greater risk of cracking in the foundation.

Figure 8. Time course curve of longitudinal strain variation in the center layer of pier section and intermediate section and distribution of measurement points

5. Conclusions

(1) After 1.5d of the mechanism sand self-compacting concrete abutment casting, in the center of the middle section to reach the peak temperature of 72.8 ℃, and keep the peak for about 4h. In the 2.5d to 4d period, for the rapid cooling period, easy to produce temperature cracks and should do a good job in time to prevent and control measures. After 10d of the overall temperature change slowly, into the slow cooling period, and finally tend to ambient temperature.

(2) The maximum temperature rise of the bridge deck concrete is about 42.1℃, the maximum cooling rate is 9.2℃/d, the maximum inside surface temperature difference is 30.0℃ and the maximum outside surface temperature difference is 22.6℃. The maximum cooling rate, the maximum temperature difference between inside with outside and the maximum temperature difference between the surface with the outside all exceed the corresponding limit value of the ordinary concrete GB 50496-2018.
Standard for construction of mass concrete", but it is not found in the site. However, no obvious cracks were found at the site, which indicates that the mechanism sand self-compacting concrete has the characteristics of high temperature rise and rapid temperature drop compared with ordinary concrete.

(3) The longitudinal direction of the abutment concrete temperature difference is small, early to compressive strain. In the age of 1.5d to 4d period, the abutment concrete surface layer is prone to cracking. In the age of about 9d, the abutment concrete by the foundation outside the restraint effect, easy to produce cracking, and should be good crack prevention and control measures.

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