Analysis on local force of cable tower in low tower cable-stayed bridge

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Abstract. Concrete under pylon saddles in extradosed cable-stayed bridges is usually subjected to tremendous cable force, which might easily cause cracking or crushing of concrete and lead to potential structural security problems. At present, the finite element mechanical analysis model for the cable saddle is not accurate enough to analyse the real stress situation. A stress analysis method of concrete under the cable saddle based on the accurate finite element model is presented in this paper. In order to obtain the exact stress results of the concrete under the saddle, this paper modelled the filament dividers in the saddle one by one, and established the loading surface element on the surface of each filament divider layer to apply the equivalent surface force layer by layer. In addition, this paper also studied the stress characteristics of the lower tower column in the tower bridge, and put forward structural reinforcement suggestions for the force of the lower tower column. The results of this paper can provide reference for the design and construction control of pylons in extradosed cable-stayed bridge.

1. Introduction

The external prestressing force of the cable-stayed cable passes through the pylon continuously by installing saddles in the pylon of the extradosed cable-stayed bridge, under the action of enormous cable force, the cable saddle would cause large local compressive stress easily to the concrete on the lower part of the cable saddle which in the pylon due to external prestressing, consequently, it is easy to cause local concrete crushing at the lower part of the cable saddle, as well as large-scale cracks, which buries the hidden danger of structural safety. Domestic and foreign scholars adopted different methods to simulate and analyse extradosed cable-stayed bridges from different perspectives.

Zhang et al [1] used ANSYS to analyse the pylon stress of extradosed cable-stayed bridge, they used contact element to simulate the contact relationship between cable and steel pipe, but the saddle structure of sub-steel-pipe is not simplified as a single bundle that ignoring the influence of external steel pipe, and each steel strand interaction of the sub-steel-pipe can’t be simulated and analysed. Tan et al [2] carried out the model test that main tower saddle segment of prestressed extradosed cable-stayed bridge by using the improved cable saddle structure of the sub-steel-pipe, and a spatial finite element model is established to analyse the stress distribution of concrete under saddle and the stress characteristics of sub-steel-pipe by applying the actual cable force to the corresponding channels according to the surface pressure in the form of spatial paraboloid. Zhang et al [3] used
three-dimensional finite element software to establish the finite element model of extradosed cable-stayed bridge cable saddle area, selected the most disadvantageous load in the overall calculation for static analysis, the stress distribution of concrete and cable duct were obtained in cable saddle area of bridge pylon. Liu et al [4] carried out full-scale model tests of the segment in the cable saddle area combined with a low-tower cable-stayed bridge project, two kinds of double-sleeve cable saddles bond anchorage structures were selected and the transmission degree of unbalanced cable force was tested by low-cycle repeated loading of single-side cable. In addition, the mechanical characteristics of concrete continuum in cable saddle area are analysed and studied, the results show that two kinds of bond anchorage structures have strong and stable anti-slip effect. Zhang [5] used the spatial finite element method to calculate the stress of the saddle. The cable force was converted into parabolic surface pressure and applied to the corresponding channels, the comparison between the sub-steel-pipe and double sleeves shown that the actual effect was closer to the actual situation than the average method and the application method. Zhu [6] took a extradosed cable-stayed bridge as the research object and calculate the stress of cable saddle by space finite element method, the cable force was converted into parabolic surface pressure and applied on the corresponding channels, he considered that the actual effect of the deviation of the tunnel construction on the tower in operation stage was better, which was closer to the actual situation than the previous average method and application method. Wang [7] took an extradosed cable-stayed bridge that the twin-tower single-cable-plane with three-span prestressed concrete as the research background, and used Midas to establish the spatial finite element model of the pylon and saddle area, under the maximum cable force, the spatial force of the whole pylon and the local stress of the concrete in the pylon saddle area were analysed. Liu [8] made use of full-scale model test to study the value and distribution of splitting stress in concrete of main tower, the spatial finite element entity model was established by using ANSYS to analyse the stress distribution under saddle and the stress performance of the filament divider under cable force loading.

From what has been discussed above, although a large number of scholars have carried out a variety of entity analysis on the stress distribution law of saddle and the local concrete stress distribution law of saddle in the model test of extradosed cable-stayed bridge, and get a macroscopic grasp of the stress state about the local concrete with saddle, there are still some problems as follows: the saddle model established mostly uses approximate shape blocks, it fails to describe the concrete stress condition of the structure accurately under the condition that steel strand concentrated action of multiple sub-steel-pipes, the stress distribution of concrete under saddle is less accurate. Therefore, this paper uses ANSYS finite element software to analyse the interaction between the saddle and the concrete of the pylon by establishing the exact model of the cable-stayed saddle pylon and applying the cable force on the corresponding channels by means of the average method, which can provide reference for the design and construction control of the pylon of the extradosed cable-stayed bridge.

2. General situation of the engineering

As shown in figure 1, there is an extradosed cable-stayed bridge with the span arrangement of 90+165+90 m, because of the tower bridge beneath the cable saddle has a large force, it is easy to crush the concrete bridge tower locally. This paper carries out force analysis on the saddle locally stressed area of the bridge. The detailed geometric structure of cable tower shown in figure 2, which including the elevation, side and section. The interaction between the steel cable saddle and the concrete of the cable tower is analysed by accurately establishing the model of the steel cable saddle concrete pylon. In addition, the stress characteristics of the lower tower column which belongs to the tower bridge was studied in this paper, besides, this paper also puts forward structural reinforcement suggestions for the force of the lower tower column.
3. Finite element analysis of cable saddle and tower column
The steel cable saddle adopts solid 45 element, each sub-steel-pipe is represented by solid element, the material properties are defined as steel, and the holes in sub-steel-pipe are ignored, besides, the solid 45 unit is also adopted by the concrete of cable tower, and the joint coupling is adopted for the contact between steel saddle and concrete tower.

Due to the meticulous consideration about the cable saddle sub-steel-pipe, there is a large number of model elements if we establish the sub-steel-pipe accurately by entity elements, it will more difficult to establish the model of cable tower completely. Therefore, the analysis range of the cable tower selected in this paper is C1 and C2 (the number of the cable is C1-C12, which varies from the side of the tower to the middle of the span) corresponding to the cable saddle of the cable tower, the steel wire in the steel cable saddle is not simulated. Besides, the force acting on the cable saddle is
simulated by the normal surface force perpendicular to the cable saddle sub-steel-pipe, and the surface force can be calculated by formula (1). The calculation results of normal equivalent forces on the surface of cable saddle sub-steel-pipe corresponding to C1 and C2 cable saddles are shown in table 1, the element type used in the model is shown in table 2.

\[ q_s = \frac{F_s}{R} \]  

(1)

Where, \( F_s \) is cable force, \( R \) is saddle radius.

Table 1. Normal equivalent forces on the surface of cable saddle sub-steel-pipe corresponding to C1 and C2 cable saddles.

| Surface of sub-steel-pipe with C1 | Surface of sub-steel-pipe with C2 |
|-----------------------------------|-----------------------------------|
| The magnitude of cable force (kN) | 4170                              | 4237                              |
| Radius of cable saddle (m)        | 3.9                               | 4.0                               |
| The magnitude of equivalent force (kN/m) | 1069.2                       | 1059.3                           |
| The magnitude of equivalent surface load (kN/m²) | 1204.1                        | 1193.0                           |

Table 2. Model / Element table adopted.

| Name of component | Types of element | Modulus of elasticity (MPa) |
|-------------------|------------------|-----------------------------|
| Steel saddle      | Solid45          | 2.1×10^5                    |
| Concrete pylon    | Solid45          | 3.45×10^4                   |
| Loading element   | Shell63(without thickness) | —                         |

Table 3. Vertical equivalent forces of cables loaded on the upper surface of cable saddle finite element model.

| Cable number | Horizontal inclination angle of cable \( \theta \) | \( \sin \theta \) | Cable force kN | Vertical force composition of unilateral cable force kN | Vertical component of cable force kN |
|--------------|---------------------------------|----------------|----------------|-------------------------------------------------|-----------------------------------|
| C3           | 22.7                            | 0.385906       | 4272           | 1649                                           | 3297                              |
| C4           | 21.4                            | 0.364877       | 4306           | 1571                                           | 3142                              |
| C5           | 20.4                            | 0.348572       | 4339           | 1512                                           | 3025                              |
| C6           | 19.5                            | 0.333807       | 4368           | 1458                                           | 2916                              |
| C7           | 18.8                            | 0.322266       | 4394           | 1416                                           | 2832                              |
| C8           | 18.2                            | 0.312335       | 4418           | 1380                                           | 2760                              |
| C9           | 17.7                            | 0.304033       | 4439           | 1350                                           | 2699                              |
| C10          | 17.2                            | 0.295708       | 4454           | 1317                                           | 2634                              |
| C11          | 16.8                            | 0.289032       | 4467           | 1291                                           | 2582                              |
| C12          | 16.4                            | 0.282341       | 4472           | 1263                                           | 2525                              |
Figure 3. Finite element model of cable tower. (a) 1/4 model of cable tower (whole), (b) 1/4 model of cable tower (local) and (c) Global boundary conditions of the model.

Figure 4. Cable saddle and loading plate element.

Figure 5. Equivalent force loading of steel wire on cable saddle loading plate element.
The load on the model surface must also include the self-weight of the tower, which is 0.260 N/m², and the vertical force of the cables which causes the pressure load on the model surface is 4.736 N/m². The total of the two loads is 4.996 N/m².

(1) The calculation results of the contact position between the lower part of the cable saddle and the concrete are shown in figures 6-10.

From the calculation results in figures 6-10, it can be seen that the regular vertical (along Y axis) compressive stress distribution appears in the lower part of saddle under the cable saddle force action, and the magnitude of compressive stress is 7.31 MPa-11.41 MPa; The inner side of the saddle arc in the X direction is basically under compression, the tensile stress with a maximum of 1.15 MPa in the X direction will appear at the junction of the arc section and the straight section of the cable saddle.

**Figure 6.** Y-direction stress nephogram on the outer surface of concrete tower column at saddle position.

**Figure 7.** Y-direction stress nephogram of the section of cable saddle of concrete tower column (right side).

**Figure 8.** Y-direction stress nephogram of the section of cable saddle of concrete tower column (left side).

**Figure 9.** X-directional stress in the section of cable saddle of concrete tower column.
The stress calculation results at the variable section of the main tower are shown in figures 11-16. It can be seen from figures 11 and 12, viewing from the force acting on the whole cable tower in the X-direction, there is a large area of tensile stress along the X-axis direction at the cross section of the cable tower (the junction between the upper tower column and the lower tower column), as shown in figure 13. The range of tensile stress is between 0.41 MPa and 8.10 MPa (mean value is 4.2 MPa), especially in a small area at the top of the arc section, the maximum tensile stress is 8.10 MPa, this is due to the change of section at the upper and lower pylons junction and the eccentric force at the lower pylon section. Figure 15 is the overall Y-direction stress diagram of the pylon, and figure 16 is the overall deformation diagram of the pylon in the X-direction. As can be seen from figure 15, the Y-direction stress in the upper part of the lower pylon in the inner side is larger than the outer side, while the case in the lower part of the lower pylon is contrary, the Y-direction stress in the inner side is smaller than the outer side. From Fig. 16, it can be seen that the bending deformation of the lower pylon in the overall direction of X-direction is the largest with the value of 0.3 mm at a distance about 5-6 meters from the bridge deck. Prestressed tension or transverse reinforcement infill treatment is needed to prevent crack propagation, or a tension member is installed at the lower tower column 5-6 meters away from the bridge deck, which can also reduce the large transverse tension stress at the variable section.

(2) The stress calculation results at the variable section of the main tower are shown in figures 11-16. It can be seen from figures 11 and 12, viewing from the force acting on the whole cable tower in the X-direction, there is a large area of tensile stress along the X-axis direction at the cross section of the cable tower (the junction between the upper tower column and the lower tower column), as shown in figure 13. The range of tensile stress is between 0.41 MPa and 8.10 MPa (mean value is 4.2 MPa), especially in a small area at the top of the arc section, the maximum tensile stress is 8.10 MPa, this is due to the change of section at the upper and lower pylons junction and the eccentric force at the lower pylon section. Figure 15 is the overall Y-direction stress diagram of the pylon, and figure 16 is the overall deformation diagram of the pylon in the X-direction. As can be seen from figure 15, the Y-direction stress in the upper part of the lower pylon in the inner side is larger than the outer side, while the case in the lower part of the lower pylon is contrary, the Y-direction stress in the inner side is smaller than the outer side. From Fig. 16, it can be seen that the bending deformation of the lower pylon in the overall direction of X-direction is the largest with the value of 0.3 mm at a distance about 5-6 meters from the bridge deck. Prestressed tension or transverse reinforcement infill treatment is needed to prevent crack propagation, or a tension member is installed at the lower tower column 5-6 meters away from the bridge deck, which can also reduce the large transverse tension stress at the variable section.
4. Conclusion

In this paper, a stress analysis method of concrete under saddle based on the exact finite element model is presented. The feasibility of this method is verified by the analysis of engineering cases, and the following conclusions are drawn:

- Under the action of local cable saddle force and upper load, regular vertical compressive stress distribution (along Y axis) appears at the lower part of cable saddle with the value of 7.31 MPa to 11.41 MPa. The inner side of the cable saddle arc in the X direction is basically under compression. At the junction of the arc section and the straight section of the saddle, the X direction tensile stress will appear, and the maximum tensile stress is 1.15 MPa, which meets the strength design requirements of C50 concrete.

- From the overall X-direction stress of the pylon, there is a large area of tension stress along the X-axis at the variable section of the pylon (the junction of the upper and lower pylons), this is due to the change of section at the upper and lower pylons junction and the eccentric force at the lower pylon section. Prestressed tension or transverse reinforcement is needed to prevent crack propagation.
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