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Preliminary communication

Cable Replacement Scheme for Low Tower Cable-Stayed Bridges Based on Sensitivity Analysis

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Abstract: Cable replacement is a key technique to solve the problems of cable corrosion and strand breakage. Cable removal causes structural changes. The choice of replacement method affects the safety of the bridge during cable replacement. A sensitivity analysis method was used to evaluate the force and deflection changes of Wohu Bridge. A reasonable method for the number and order of cable replacement was proposed; by comparing different cable removal schemes, it was revealed that the cable force and beam stress changes of the cables closest to the removed cable were the most significant. The results showed that the cable force increment of the surrounding cables was the largest when removing the longest cable. The structural impact change was small when removing the shortest cable. The maximum deflection at the top of the tower decreased with the decrease of the length of the removed cable. It was recommended to replace two cables symmetrically from the center of the tower, and the optimal replacement order was from the shortest cable to the longest cable. Furthermore, this paper also studied the influence of variable load on the cable replacement scheme, and demonstrated that the design scheme of opening part of traffic in this paper was safe and feasible.

Keywords: cable replacement process analysis; finite element analysis; low tower cable-stayed bridge

1 INTRODUCTION

Cable-stayed bridges are widely used because of their attractive shapes and strong spanning abilities [1, 2]. Statistics show that cable-stayed bridges built in China now account for 35% of the total number of cable-stayed bridges in the world [3]. Cables are the main force-bearing components of a cable-stayed bridge, and any damage to one or more cables may cause its structural failure. Now some early built cable-stayed bridges have suffered serious cable corrosion [4, 5]. Therefore, it would be necessary to conduct a comprehensive analysis of cable replacement that considers cable corrosion, damages, and even fractures during operation in the scheme design stage [6]. The cable replacement should be designed on the premise of minimum impact on normal traffic during cable replacement to minimize maintenance costs [7]. The removal of old cables and the re-tensioning of new cables during replacement inevitably lead to changes in the stress indicators of the bridge structure, including the bridge deck alignment, girder stress, and residual cable forces, which cause the redistribution of the internal forces of the cable-stayed bridge and adversely affect the bridge structure itself [8]. Through the establishment of independent finite element models for various cable replacement schemes and old cable removal sequences, the stress state indicators, such as cable force increment, main beam alignment, and main beam strain, under various schemes at the construction stage were compared, and the optimal cable replacement scheme was obtained to provide guidance for the construction process [9-11].

2 PROJECT OVERVIEW

The bridge investigated in this work is a 110 m single-tower single-cable plane concrete girder, cable-stayed bridge with a span of 55 + 55 m. Refer to Fig. 1 for the general layout of the bridge. The main tower is a rectangular reinforced concrete bridge tower, with a longitudinal width of 3.4 meters, a transverse width of 2.0 meters, a maximum tower height of 24.5 meters, and the bridge tower is fixedly connected with the main girder and the bridge pier. A high-strength low-relaxation steel strand cable system was adopted for the stay cable, and the diameter of a single steel strand was 15.24 mm.

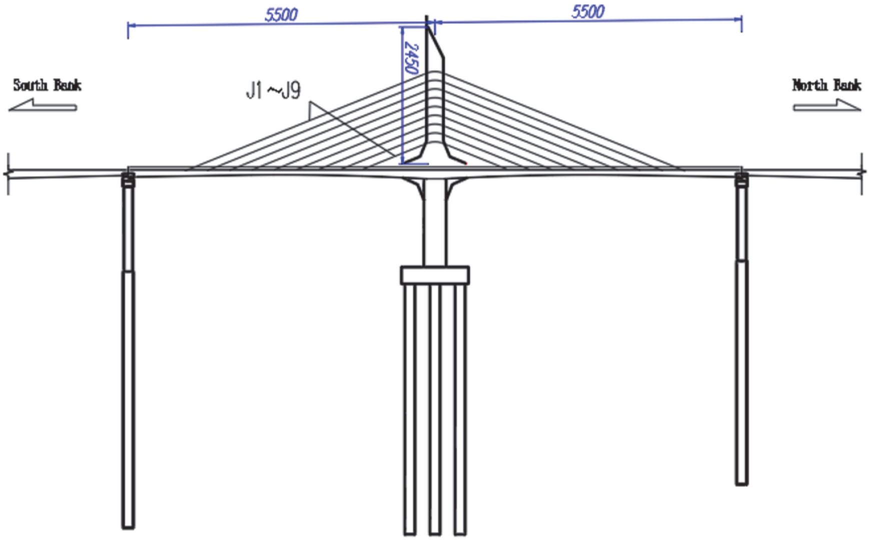


Figure 1 General layout of the bridge (unit: m)

The whole bridge had the following characteristics: standard tensile strength of the steel strands f_{pk} = 1860 MPa, E_y = 195 000 MPa, extruded high-density polyethylene sheath, single-cable plane system, harp-shaped arrangement, and nine pairs of stay cables. The cable spacing on the tower (vertical spacing between anchor points) was 1.6 m and that on the beam (transverse spacing between anchor points) was 4 m. The whole stay cables were tensioned, with the tensioning end set in the tower, and the longest stay cable (including anchorage) being 99.03 m. The prestressed tendons and cables in the box girder are made of standard steel strands. The concrete strength grade of the box girder and the tower is C50.

According to the design drawings, the material properties are shown in Tab. 1.

Table 1 Material properties

Name	Material	Elastic modulus / MPa	Poisson's ratio / MPa	Thermal expansioncoefficient / 1/°C	Strength / MPa
Concrete	C50	35000	0.2	0.00001	35
Cable	Steel stranded wire	195000	0.3	0.000012	1860

The main beam section was a three-cell section of a prestressed concrete single box, and the beam height showed a parabolic variation. The main dimensions of the main beam section are shown in Fig. 2.

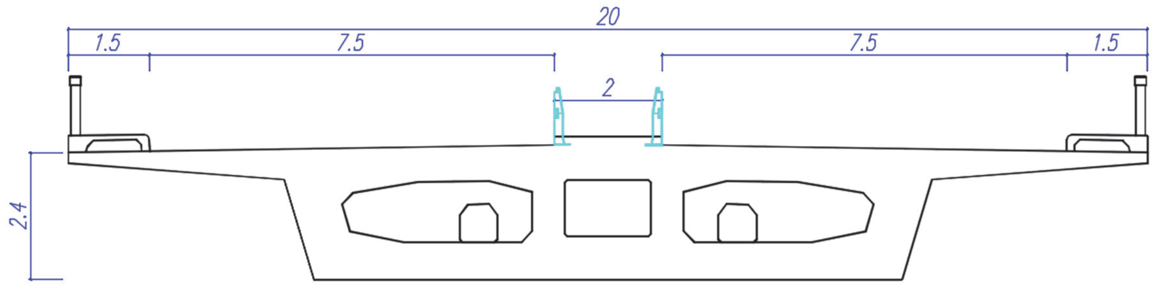


Figure 2 Section and dimension of the main beam (unit: m)

3 FINITE ELEMENT SIMULATION

Fig. 3 shows that the calculation model of the bridge is established with the aid of the bridge finite element software MIDAS/Civil. The bridge was divided into 461 beam elements, 18 truss elements (only tension elements), and 515 nodes. During modeling, beam elements were used to simulate the main beam and the main tower, and the truss elements to simulate the stay cables. Through cable adjustment and other technical means, the line shape and cable force of the model were approximated to those of the structure before cable replacement. The tower and beam were consolidated. The system temperature difference was calculated to be ±25 °C, and the temperature difference between the upper and lower edges of the main beam and the left and right sides of the cable tower ±5 °C. The initial tensions of the stay cable and longitudinal prestressed tendons at the upper and lower edges of the main beam were subject to the design data. A finite element model of rigid connection between beam and cable was established, in which the tower and main beam as well as pier were rigidly connected, and the farthest pier from tower and beam had a hinge connection. The boundary condition of fixed connection between foundation and pier or tower was considered. The sag of cable had an influence on the analysis. However, the impact existed before and after cable removal. Only the increment of cable force and the stress change of beam were considered. Therefore, the influence of cable sag was not considered in this paper. The finite element model is provided in Fig. 3.

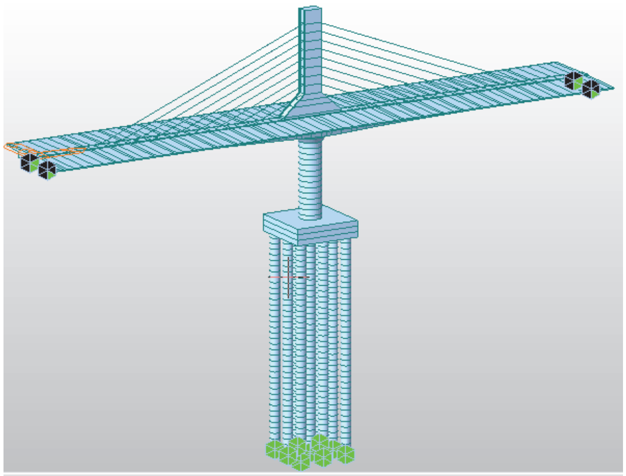


Figure 3 Finite element model of the cable-stayed bridge

Two construction stages were established: The first stage was the initial completion state before cable replacement that the bridge needed to recover to through cable adjustment, and the second stage was the removal of the corresponding stay cables, which was realized through element passivation.

Table 2 Cable replacement conditions under each scheme

Working condition of cable replacement	Scheme 1	Scheme 2	Scheme 3	Scheme 4	Scheme 5
1 Removal of old cable	J9	J9, J8	J9, J8, J7, J6	J1, J2	J1, J2, J3, J4
2 Installation of new cable	J9	J9, J8	J9, J8, J7, J6	J1, J2	J1, J2, J3, J4
3 Removal of old cable	J8	J7, J6	J5, J4, J3, J2	J3, J4	J5, J6, J7, J8
4 Installation of new cable	J8	J7, J6	J5, J4, J3, J2	J3, J4	J5, J6, J7, J8
……	……	……	……	……	……
17 Removal of old cable	J1	J3, J2		J7, J8	—
18 Installation of new cable	J1	J3, J2		J7, J8	—

At present, three replacement schemes for stay cables are commonly used: the first is to replace the stay cables separately; the second is to replace the stay cables symmetrically on both sides with the bridge tower as the center, and the third is to replace the stay cables in an antisymmetric manner on both sides with the bridge tower as the center. At the same time, to facilitate construction organization, two main ways are available to replace the cables: one is to replace the outer cables (longer cables) first and then the inner ones (shorter ones), and the other is to replace the inner cables first and then the outer ones.

Considering the structural layout characteristics of the single-tower and single-cable plane, this study compared the schemes of the symmetrical replacement of individual and multiple cables for the purpose of selecting the optimal cable replacement scheme. Then, based on the optimal scheme, the advantages and disadvantages of five replacement sequences from short to long cables or from long to short cables were compared to find the optimal replacement sequence of stay cables.

Scheme 1: remove one short cable in turn; Scheme 2: remove two short cables in turn; Scheme 3: remove four short cables in turn; Scheme 4: remove two long cables in turn; and Scheme 5: remove four long cables in turn. Refer to Tab. 2 for the specific cable replacement conditions.

Five finite element calculation models were established for the above schemes.

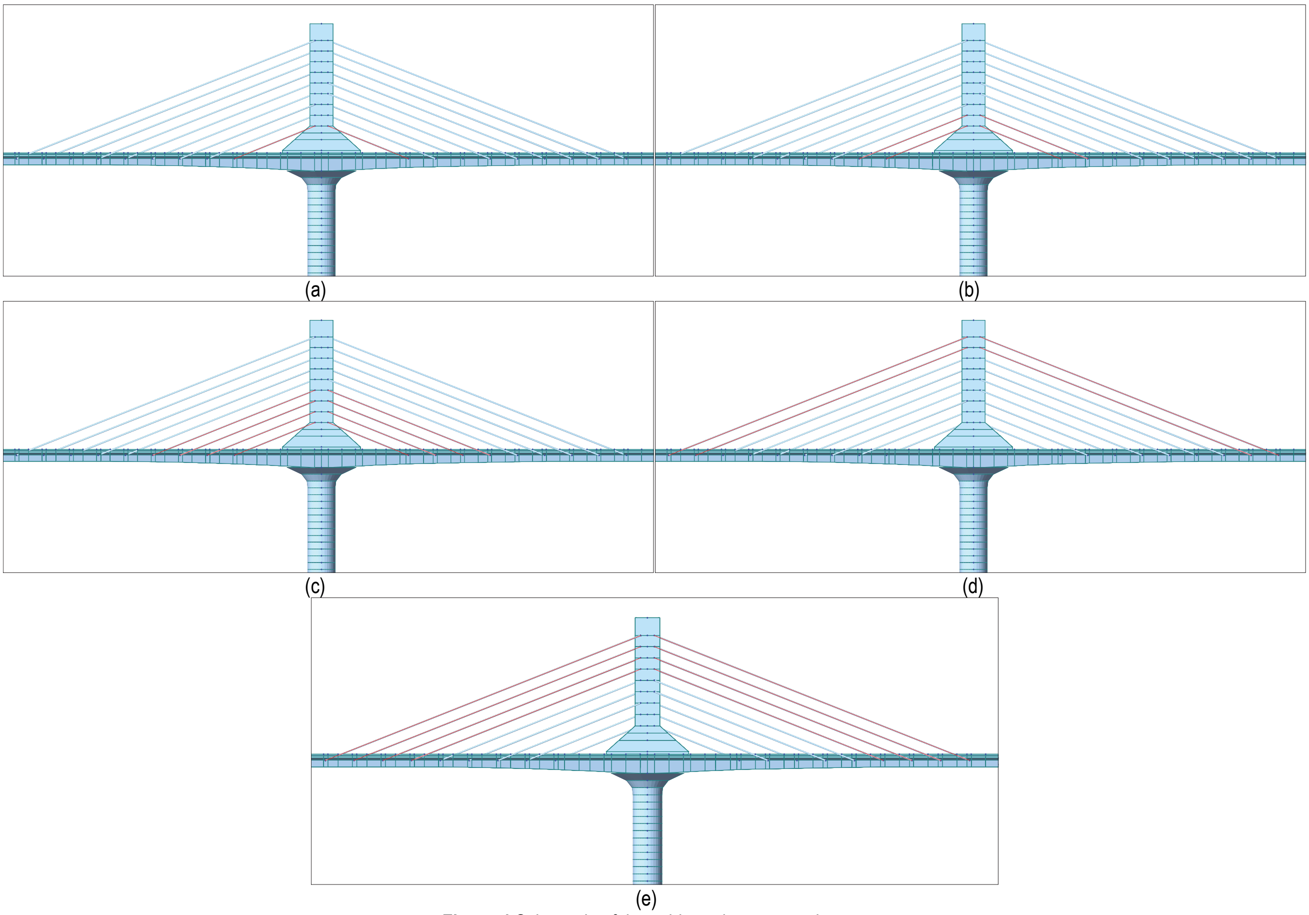


Figure 4 Schematic of the cable replacement schemes

4 COMPARISON OF CABLE REPLACEMENT SCHEMES

In a cable replacement operation, the cable-stayed bridge can be affected by many uncertain factors and especially highly vulnerable to external interference in a complex traffic environment. A reasonable cable replacement scheme must be designed to guarantee the safe and smooth cable replacement operation of the cable-stayed bridge and ensure that the bridge will meet the design requirements after cable replacement. Different cable replacement schemes have different effects on the internal force of the structure and the bridge alignment. This work mainly analyzed the cable force, main beam deformation and stress under each cable replacement scheme and studied the impacts of different operation sequences and numbers of cable replacements on the structure under closed traffic conditions.

4.1 Increment in the Vertical Displacement of the Main Beam

Fig. 5 shows the data when the vertical displacement of the beam reached the maximum under the corresponding cable replacement conditions in each cable replacement scheme. The peak values of displacement increment measured from the north side to the south side of the main beam are listed in Tab. 3.

Fig. 5 shows the displacement under Scheme 3 was the largest. Under the cable removal condition of Scheme 3, the maximum displacement occurred when cables J2, J3, J4, and J5 were removed simultaneously; under the cable removal condition of Scheme 5, the maximum displacement occurred when cables J1, J2, J3, and J4 were removed simultaneously.

Table 3 Maximum displacement increment of the main beam under each scheme unit: mm

Location	Scheme 1	Scheme 2	Scheme 3	Scheme 4	Scheme 5
Maximum displacement at the south side of the main beam	25.9	39.3	67.9	39.7	66.5
Maximum displacement at the north side of the main beam	32.5	45.7	73.3	45.8	71.3

Note: The actual displacement in the table is positive downward.

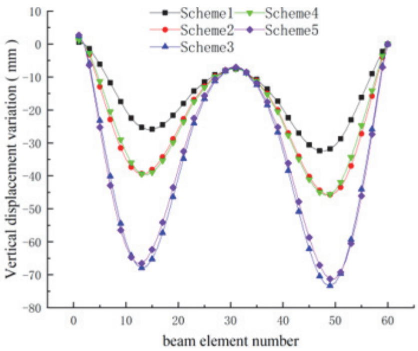


Figure 5 Comparison of the increment in the vertical displacement of the main beam under each scheme

Compared with other schemes, the scheme involving the removal of cables from shorter ones to longer ones or the one involving the removal of cables from longer ones to shorter ones represented dangerous working conditions if four cables were removed simultaneously. The figure also shows that the scheme where cables are removed individually had the minimum beam displacement. Under Schemes 2 and 4, the increment in the vertical displacement of the beam was related only to the number of cables replaced but not to the sequence of cable replacement.

4.2 Increment in the Horizontal Displacement of the Bridge Tower

The displacement of the main tower is one of the key points to be controlled during cable replacement. Fig. 6 shows the maximum tower displacement in each scheme under each cable replacement condition. Fig. 6 shows that the maximum horizontal displacement of the bridge tower occurred under the working conditions where cables J1 and J2 were removed in Scheme 4. However, given that the maximum horizontal displacement was less than 4 mm, the bridge tower displacement was not the main factor affecting the selection of the cable replacement scheme.

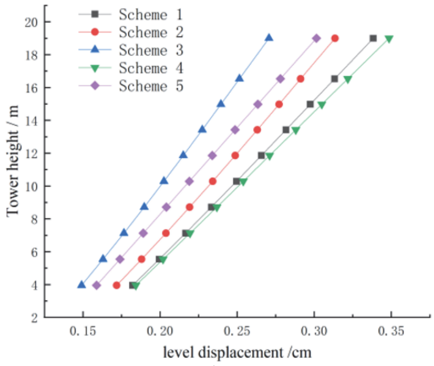
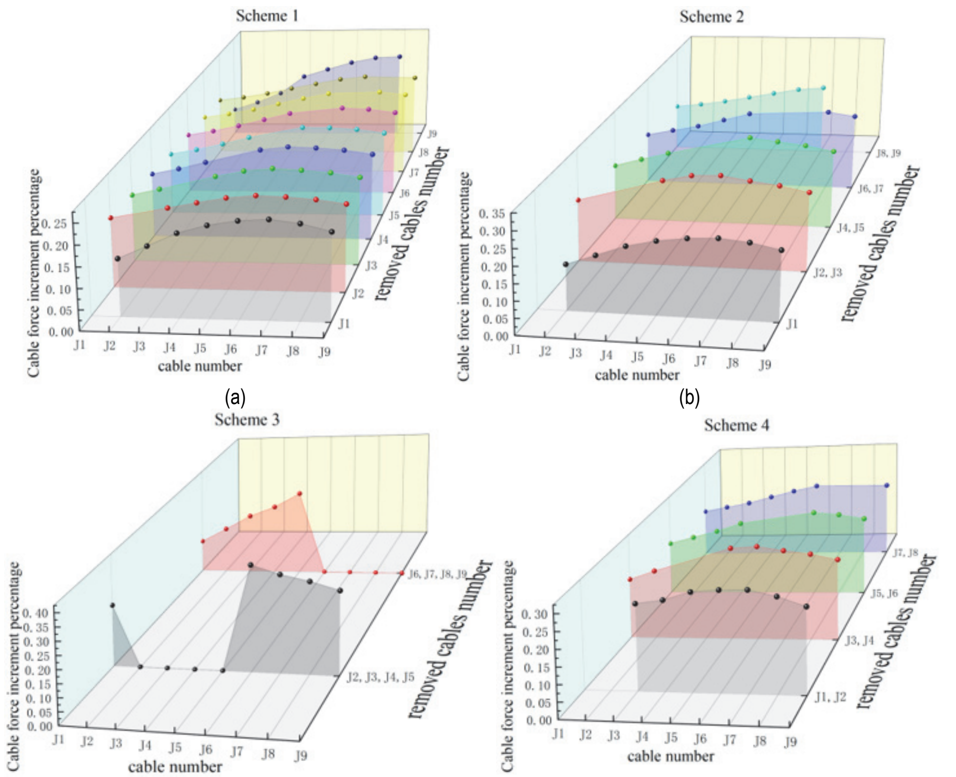


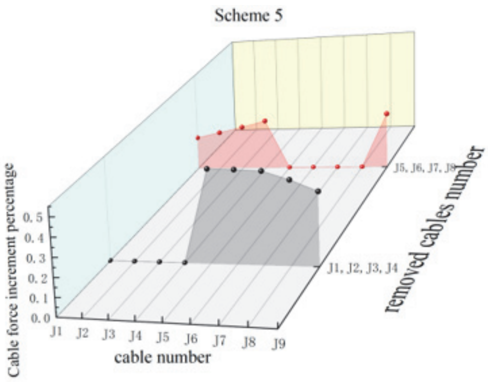
Figure 6 Horizontal displacement of the bridge tower under each scheme

4.3 Increment in the Cable Force of the Stay Cables

Fig. 7 shows the percentages of the increments in cable force under different schemes. The percentage of cable force increment is the ratio of the cable force increment to the original cable force. Fig. 7 shows that, after the removal of some cables, the forces of the cables at other positions increased.



(a) (b) (c) (d)



(e)

Figure 7 Percentages of cable force increments under different schemes

The closer the cables to the removed ones, the more significant the cable force increment. Under Scheme 1, the maximum cable force increment originated from the removal of cable J4 - the cable force increment of cable J6 was 735 kN, which was 22.96% higher than the original cable force. When the cables of this bridge were replaced individually, the operation of replacing the cables from shorter ones to longer ones showed the same pattern with the operation of replacing the cables from longer ones to shorter ones. In contrast to that under Scheme 4, the maximum cable force increment under Scheme 2 occurred during the removal of longer cables regardless of the cable removal sequence. The maximum percentage of cable force increment under Scheme 2 occurred in cable J4 - 25.98%, and that under Scheme 4 occurred in cable J3 - 25.51%. Under Schemes 3 and 5, four cables were removed. Fig. 7c shows that when cables J5, J4, J3, and J2 were removed, the cable force of cable J6 increased by 1252 kN, accounting for 39.11% of the original cable force. Fig. 7e shows that when cables J1, J2, J3, and J4 were removed simultaneously, the cable force of cable J5 increased by 48.26%, reaching 1544 kN. Based on engineering experience, removing four stay cables at the same time is inappropriate. From the figures, it can be seen that the cable force increment of the remaining cables when the shorter cables were removed before the longer ones was smaller than that when longer cables were removed first.

4.4 Increment in the Stress of the Main Beam Section

Given the large deflection of the 1/2 section of the side span during cable replacement, the changes in the stress of the bottom plate of the control section should also be paid attention to. The five schemes all focused on the symmetrical cable replacement with the bridge tower as the center. Analysis of the deflection changes demonstrates that Schemes 2 and 4 can be implemented within the deflection limits while achieving rapid cable replacement. Therefore, this study only analyzed the stress changes in the main beam control section under Schemes 2 and 4. The maximum tensile and compressive stress variations of each main beam control section are presented in Tab. 4.

The changes in the tensile stresses of the upper and lower edges of the main beam under the two schemes were very small and thus can be ignored. During cable replacement under Scheme 2, the maximum compressive stress of the upper edge of the main beam changed to 1.08 MPa, while that of the lower edge of the main beam 1.85 MPa. During cable replacement under Scheme 4, the maximum compressive stress of the upper edge of the main beam changed to 13.50 MPa, while that of the lower edge of the main beam 14.50 MPa.

Table 4 Maximum stresses of the upper and lower edges of the main beam (MPa)

Position	Tensile and compressive stress	Scheme 2	Scheme 4	Stress difference
Upper edge of the main beam	Tensile stress	0.0573	0.0298	−0.0264
	Compressive stress	1.08	13.50	12.42
Lower edge of the main beam	Tensile stress	0	0	0
	Compressive stress	1.85	14.50	12.65

Through comparison of the two schemes, it can be seen that the maximum compressive stresses of the upper edge of the main beam under Scheme 4 increased by 12.42 MPa and that of the lower edge of the main beam increased by 12.65 MPa, indicating that Scheme 2 is more reasonable than Scheme 4.

5 EFFECT OF VARIABLE LOAD ON CABLE REPLACEMENT

The above research shows that Scheme 2 is more reasonable than Scheme 4. On this basis, the effect of two schemes based on partial traffic opening on cable replacement was studied to see whether they can meet traffic demands. In this study, the effects of crowd and vehicle loads on cable replacement construction were designed and analyzed. The standard lane load specified in the *General Specification for Design of Highway Bridges and Culverts* (JTG D60-2015) was used for layout calculation.

The scheme for partial traffic opening is as follows: with the bridge tower as the center, one lane was closed respectively on the left and right sides of the bridge tower as the construction area, and the other two lanes and sidewalks were opened for traffic, as shown in Fig. 8. The design lane load of the bridge was used for calculation of the changes in the vertical displacement of the bridge under the lane load and analysis of the impact of variable load on the cable replacement.

The actual positions of the stay cables individually corresponded to the cable node numbers in the model, of which nodes 0 and 60 were the end points of the main beam, as shown in Fig. 9. Fig.10a shows the changes in the overall vertical displacement of the bridge deck when cables were replaced under closed and semiclosed traffic conditions in Scheme 2. Fig. 10b presents the changes in the overall vertical displacement of the bridge deck when cables were replaced under closed and semiclosed traffic conditions in Scheme 4.

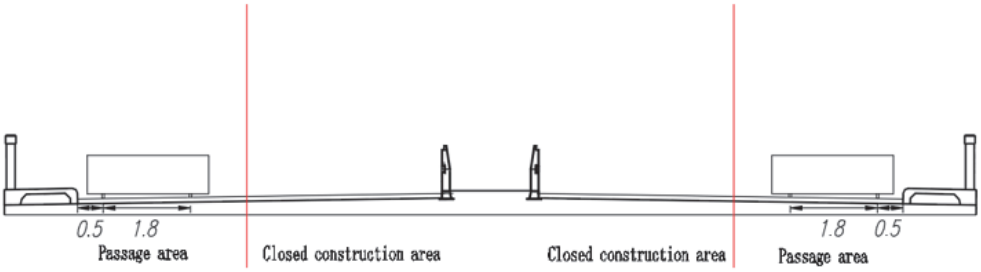


Figure 8 Transverse layout of the lanes under partially opened traffic schemes unit: (m)

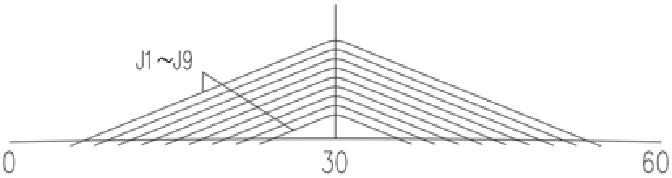
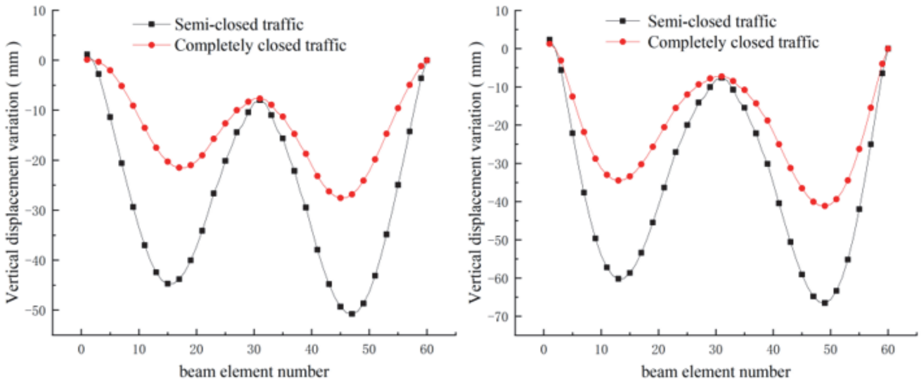
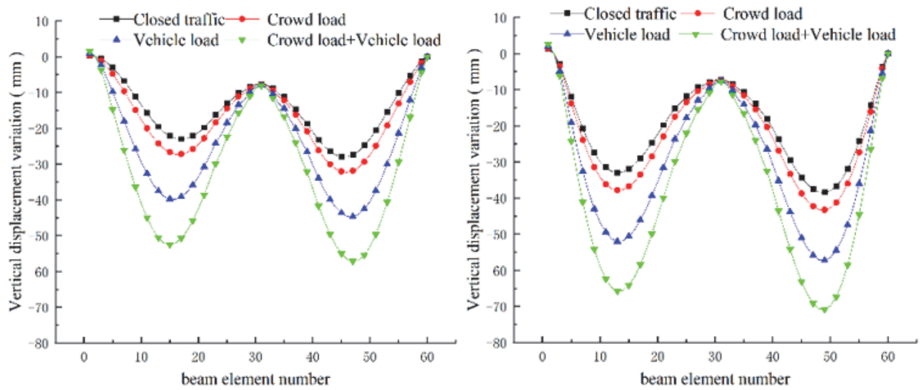


Figure 9 Numbered position of cable-stayed bridge units



(a) Scheme 2 (b) Scheme 4

Figure 10 Changes in the vertical displacement of the main beam under semi-closed and closed traffic conditions



(a) Scheme 2 (b) Scheme 4

Figure 11 Effect of variable load on the vertical displacement of the main beam

From Fig. 10a, it can be seen that the maximum displacement of the south side of the bridge was 44.7 mm and that of the north side was 50.8 mm when traffic was partially opened under Scheme 2. The maximum displacement of the south side of the bridge was 60.2 mm and that of the north side was 66.5 mm when traffic was partially opened under Scheme 4. The displacement of the beam was significant in the presence of vehicle load regardless of whether short or long cables were removed. For example, when two short cables were removed, the difference between the maximum displacements of the main beam in the partially opened traffic and closed traffic conditions reached 25.7 mm. And judging from Fig 10a and Fig. 10b, under the partially opened traffic condition, the vertical displacement of the main beam under Scheme 4 was considerably larger than that under Scheme 2.

At the same time, the effect of crowd load on cable replacement was also studied. Fig. 11a and Fig. 11b illustrate the effects of crowd load on the vertical displacement of the bridge under Schemes 2 and 4. Fig. 11a suggests that the maximum vertical displacement of the beam caused by crowd load was only 32.2 mm. While crowd load had a small effect on the cable replacement operation, vehicle load had a larger effect - the maximum displacement of the beam caused by the latter reached 50.7 mm. The displacement was even aggravated under the combined action of crowd and vehicle loads, in which case, the maximum vertical displacement of the beam reached 58.9 mm. Accordingly, a similar conclusion can be drawn from Fig. 11b, which showed the situation under Scheme 4, where the effect of variable load on the vertical displacement of the main beam was highly obvious, also indicating that Scheme 2 is more reasonable than other schemes.

Although the downward displacement of the main beam was more significant under the partially opened traffic condition than that under the closed traffic condition, it is still below the allowed vertical displacement (δ = 11000/500 = 22 cm) of the main span as specified in *Design Specification for Highway Cable-stayed Bridge* (JTG/T 3365-01-2020) - the maximum displacement of the main beam was no greater than 22 cm. Therefore, when cable replacement was carried out under the established partially opened traffic scheme, the changes in the overall and local vertical displacement of the bridge deck met the requirements.

6 CONCLUSIONS

In this study on the design of cable replacement sequences, the bridge state before cable replacement was first determined and the stress states of the bridge under various cable replacement schemes were calculated. Then, different cable replacement schemes were compared and evaluated from the perspectives of cable force, main beam alignment, stress, and horizontal displacement of the bridge tower. The following conclusions were drawn:

(1) In the vicinity of the cable undergoing dismantlement, a conspicuous fluctuation in both cable tension and beam stress can be observed. Disregarding the spatial orientation of the beam, the distribution of beam stress variation exhibits symmetry. The most significant alteration in beam stress arises from the elimination of the longest cable (or its nearest neighbor). Among all the scenarios investigated in this study, a reduction in the length of the detached cable corresponds to a decrease in the maximum deflection experienced at the top of the tower.

(2) In the absence of accounting for the random distribution of cable damage severity, an optimal strategy for cable replacement is presented. It is proposed to systematically replace two cables, in a symmetrical manner, with the bridge tower as the focal point. Drawing upon the findings of the main beam stress analysis, it is recommended to prioritize the replacement sequence from the shortest to the longest cable.

(3) In consideration of the effect of variable load on cable replacement, the semi-closed traffic scheme mentioned in this work is able to meet the relevant requirements.

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