CIVIL ENGINEERING JOURNAL

Article no. 13

THE CIVIL ENGINEERING JOURNAL 2-2026

MAINTENANCE CASE STUDY OF PEDESTRIAN CABLE-STAYED BRIDGE WITH SPARSE CABLE SYSTEM

Dandan Hu[1], Yu Wang[1], Baozhen Yan [2]

1. *School of Intelligent and Construction Engineering, Harbin University, No.109 Zhongxing Road, Harbin, Heilongjiang Province, China; Hardandan@163.com*
2. *Heilongjiang Province Highway Bridge Survey Design Co. Ltd., No. 90 Qingbin Road, Harbin 150040, China*

Received: *05.12.2025*
Received in revised form: *02.07.2026*
Accepted: *14.07.2026*

ABSTRACT

The cable-stayed bridges built in the early days have some hidden dangers due to the corrosion of the cable and the looseness of the anchor. In this paper, the control process of cable replacement of pedestrian cable-stayed bridge is analyzed, the characteristics of different construction parameters in different construction stages are discussed, and the theoretical cable force value is compared with the measured cable force value by the finite element method. The results show that the error between the measured cable force and the theoretical cable force is within 2% of the standard requirement. The linear shape of the main beam has been greatly improved, with a maximum elevation of 4.9 cm, making pedestrians travel more comfortable. The maximum offset value of the main tower is 1.3 mm, which does not exceed the standard requirements when the construction starts and ends, and the cable replacement construction process meets the design requirements.

KEYWORDS

Pedestrian cable-stayed bridge, Replace the cable, Construction monitoring, Cable force

INTRODUCTION

Designers prefer cable-stayed bridge in the process of bridge development due to their aesthetic appeal and impressive span capacity. However, as cable-stayed bridges continue to evolve and proliferate, a myriad of corresponding issues arise. As the lifespan of cable-stayed bridges extends, the challenge of cable replacement also looms on the horizon. [1-4].

The cables of cable-stayed bridges serve as one of the primary load-bearing components, enduring significant tensile forces and supporting the entire bridge's weight. Over time, these cables undergo erosion from various natural elements (such as wind, rain, snow, and frost) as well as the impact of dead and live loads incurred during bridge operation, leading to gradual aging and damage.

[5,6]. If a damaged cable is not promptly replaced, it may compromise the structural integrity of the bridge, potentially leading to severe safety accidents. Therefore, to ensure the safety of the bridge structure, it is imperative to conduct regular inspections and replacements of the cables [7].

During the operation of a cable-stayed bridge, the cables are subjected to both tension and pressure, which can cause relaxation of the cables and subsequent deformation of the bridge structure. [9]. By replacing the cables, the internal force distribution of the bridge can be adjusted, thereby restoring and enhancing the overall performance of the bridge. Additionally, cable replacement can address issues such as bridge vibration and noise resulting from cable damage, ultimately improving driving comfort and the service quality of the bridge [10, 11]. Cable damage and aging are significant factors that shorten the lifespan of a bridge. By promptly replacing damaged cables, the service life of the bridge can be extended, and the costs associated with bridge repair and maintenance can be minimized. Furthermore, cable replacement enhances the durability and disaster resilience of the bridge, making it more stable and secure in the face of extreme weather conditions and natural disasters [12, 13].

The cable-stayed bridges built in the early days have some safety risks due to cable corrosion and loose anchorage fatigue, so the cable-stayed bridges have been strengthened by cable replacement [14,15]. Since most of the cable-stayed bridges have been in operation for many years, and the main bridge has been in a stable and balanced state, the principle of cable replacement is to try to make the stress state of the structure not to change greatly [16]. Replacing the original cable-stayed cables aims to optimize the internal forces of the main beam and ensure that the structural stress state and deformation remain within safe limits during the construction of the cable-stayed bridge. The primary challenges faced by the construction control of cable-stayed bridge replacement projects include achieving the anticipated linear shape of the main beam upon completion and maintaining the structure in an optimal stress state [17].

At present, the design load grade of the original bridge cannot meet the needs of the current urban traffic flow, and the protective cover of the cable is also cracked in many places, leading to the corrosion of part of the cable [18, 19]. When the vehicle passes over the bridge, there is an obvious tremor phenomenon, so in order to ensure the normal use of the bridge, it was decided to add two sets of parallel cables to the bridge [20]. At present, there are relatively few researches on cable replacement construction control of pedestrian cable-stayed bridge, mostly on highway, railway and urban bridge. Especially for cable-stayed bridges in sparse structure systems, the distance between the cables is relatively large, and a single cable plays a significant role in supporting the main girder. Based on this, this paper relies on the rainbow bridge cable replacement project, the pedestrian steel box girder cable-stayed bridge cable replacement construction monitoring system research. The sparse research results can provide reference and guidance for the replacement of guy wires in similar slender arch bridges.

PROJECT OVERVIEW

The bridge is a cable-stayed bridge featuring steel box beams as its main structural element. It is designed as a single-tower double-cable-plane cable-stayed bridge with a relatively streamlined and simple bridge tower design. The main bridge spans 2×47.0 m, while the approach bridges at both ends have a span arrangement of 3×12.5 m each. The elevation of the bridge is illustrated in

Figure 1.

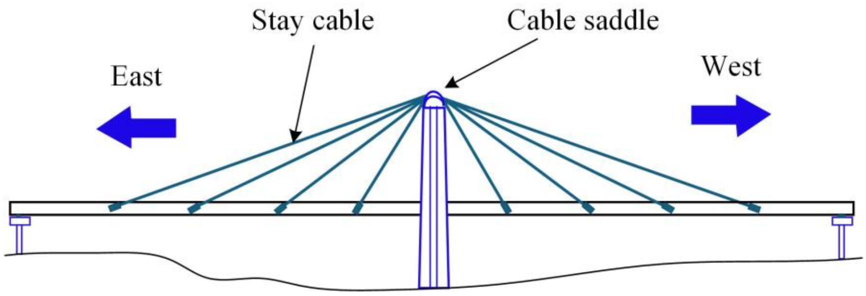


Fig. 1 - Elevation of the bridge

The tower of the cable-stayed bridge is constructed from reinforced concrete and features a semi-floating pier-tower-beam system. The tower pier forms a unified structure, with supports provided for both the pier and the bridge to support the main beam. The force distribution on the main beam resembles that of a continuous beam supported by elastic supports across its span. To accommodate intermediate supports, a beam is installed at the tower pier. The spacing between the cables is wide, and the tower height is moderately proportioned. The bottom of the main beam is equipped with a plate rubber support, while the substructure utilizes pile piers. The cable arrangement is radial, featuring a medium-density layout with spacing approximately 10 m apart.

The pedestrian cable-stayed bridge boasts a relatively low building height, which fully meets the requirements for clearance and aesthetics underneath, while minimizing the filling height of the approach road to reduce environmental impact. Each cable consists of two wire ropes, each with a diameter of 28.5 mm. The cable itself is woven from multiple strands of steel wire, offering high strength, wear resistance, corrosion resistance, and other beneficial properties. Typically, these cables are made from high-quality carbon structural steel or alloy structural steel. The cable saddle adopts curved clip, and the curvature radius of the seat plate is greater than 30 times the nominal diameter of the wire rope. The sleeve is poured at a high temperature to seal the wire rope at a temperature of about 450℃. The spacing of the cables is 10 m. There are 4 pairs on each side. Each cable is composed of 2 steel wire ropes produced by the Japanese KJkOKU Steel Works, with a nominal diameter of 28.5 mm and a non-twisted, ordinary right-twist (z) type. The breaking strength of each steel wire rope is 70 t. The design strength is set at 20 t. The newly replaced cable consists of 37 high-strength parallel steel wires with a diameter of 5 mm. The ultimate tensile strength limit is 1860 MPa, and the breaking strength is 121 t.

Each span of the upper structure of the ladder bridge at both ends of the main bridge is composed of four simple ordinary concrete rectangular section beams, and the stair step plates are set up between the beams. The substructure is made of bored piles, the piers are pillar piers, and the abutments are gravity abutments, see Figure 2.

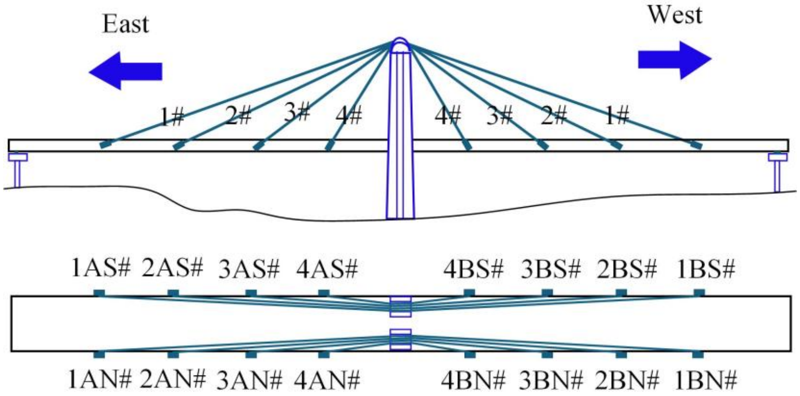


Fig. 2 - Bridge plane temporary cable number diagram (unit: cm)

Due to the limitation of the technical conditions during construction, the steel cable is coated with PE pipe. In recent years, the protective layer of the cable has been aging, and the steel cable and anchor have been damaged and corroded in different degrees. In order to ensure the service performance of the bridge, the safety and durability of the bridge are improved, and the cable-stayed bridge is replaced.

The bottom and sides of the uniform-section continuous steel box girder have extensive areas of rust paint peeling off and the steel plates are rusted. Water seepage occurs at the welding positions of the main girder, as shown in Figure 3.

Fig. 3 - Rusting condition of steel main beam

Fig. 4 - Anchor head protective cover fallen off

On both the north and south sides, there are 8 diagonal cables symmetrically distributed. Some of the cable anchor heads at the bottom have fallen off, exposing the anchor heads directly to the external environment. The simple PVC pipe covering of the bridge's cables has cracked to varying degrees, as shown in Figure 4.

There are local concrete damage and exposed reinforcing bars with rusting phenomena on the surface of Sota, as shown in Figures 5 and 6.

Fig. 5 - Concrete Damage in Sota (1)

Fig. 6 - Concrete Damage in Sota (2)

CALCULATION OF THE STRESS-FREE LENGTH OF THE CABLE

This paper intends to calculate and verify the stress-free length of the new cable using the catenary line method. The stress-free length S_0 of the new cable can be obtained by subtracting the elastic elongation amount $\triangle S$ of the cable under the tension T from the catenary line length S of the cable, and considering the elastic elongation correction amount $\triangle$ caused by the change in cable specifications or cable force.

$$S_0=S-\triangle S-\triangle \tag{1}$$

When calculating the stress-free length of the new cable using the catenary method, accurate tower and girder anchor point coordinates and cable tension are the key to solving the problem. The determination of tower, girder anchor point coordinates and cable tension can be done in two ways: first, it can be clearly defined based on the original design and construction drawings of the bridge; second, measurement work can be carried out on the bridge, and the stress-free length of the cable can be calculated using the measured values. The former requires full consideration of whether the original basic data of the target bridge is complete and accurate. The measurement of the tower, beam anchor points' coordinates and the tension of the stay cables for the already constructed bridges poses certain technical challenges.

Furthermore, if the stress-free length of the old cable is known, the calculation can be corrected based on the stress-free state method of the stay cable and in combination with the actual situation of the stay cable anchorage. The stress-free length of the new cable S_0 can be obtained by subtracting the correction amountδdue to the stay cable anchorage from the stress-free length L_0 of the old cable, and considering the elastic elongation correction caused by the cable specification, the main bridge alignment adjustment, and the change in cable force.

$$S_0= L_0-\triangle-\delta \tag{2}$$

When using the stress-free state method to calculate the stress-free length of the new cable, it is necessary to clearly specify the stress-free length of the old cable, the actual extension of the anchorages at the tower and beam ends, and the ideal installation position of the original anchorages. This can be calculated by comparing the ideal installation position of the original anchorages with the measured values. Compared with the catenary line method, by using this method to correct and calculate the stress-free length of the stay cables only depends on the anchoring conditions of the stay cables in the current state, and is not affected by the actual cable force and the actual anchor

DOI 10.14311/CEJ.2026.02.0013

Article no. 13

THE CIVIL ENGINEERING JOURNAL 2-2026

point coordinates.

CABLE REPLACEMENT PLAN

The design unit, the construction unit, the monitoring unit and the supervision unit have determined the construction technical plan of the cable replacement.

① Installation and tensioning temporary cables ;
② Remove old cables;
③ Installation and tensioning new cables ;
④ Removal of temporary cables;
⑤ According to the measured data of cable force and elevation, the cable force is adjusted after calculation.

The construction calculation conditions of cable replacement process are shown in Table 1. The cable replacement process is shown in Figure 7.

Tab. 1 - Construction calculation condition table of cable replacement process

Operating condition number	Description of working condition
1	Before replacement
2	Tensioning temporary cable
3	Removal of original cables
4	Installation new cables
5	Removal of temporary cables

（a） （b） （c）

Fig. 7 - Cable change process diagram: （a）Install temporary cable，（b）Install New cable and remove temporary cable，（c）Tensioning the new cable

CABLE FORCE MONITORING

Temporary cable installation and old cable removal

Since the bridges whose cables need replacement have been in operation for many years, various factors can alter their dead load during actual use, including wear and tear on the bridge deck pavement and the addition or removal of ancillary facilities. These changes can significantly impact the overall stress state of the bridge structure, subsequently affecting the stress state of the

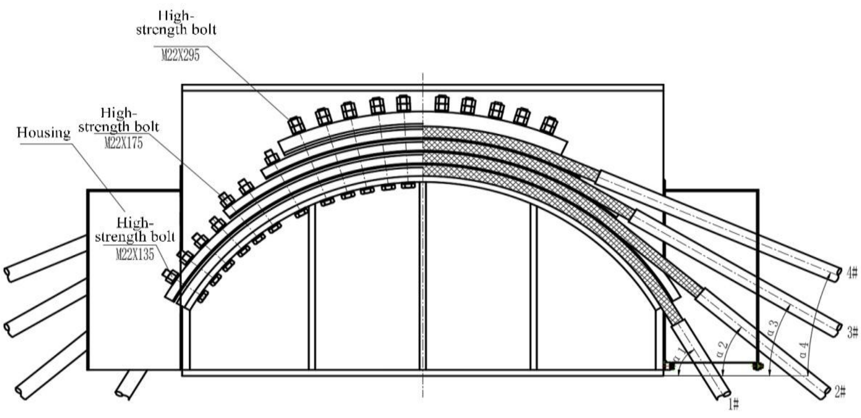
CIVIL ENGINEERING JOURNAL

cables. Over time, the cable material undergoes aging, leading to a reduction in its elastic modulus and causing stress relaxation. Stress relaxation decreases the load-carrying capacity of the cables, thereby compromising the safety and stability of the bridge structure. Furthermore, it exacerbates the overall deformation of the bridge structure, posing a significant threat to operational safety. For double-plane cable-stayed bridges, it is imperative that not all cables in the same group are removed simultaneously when uninstalling the original cable force. The process of monitoring the unloading cable force during cable replacement construction is indeed intricate and crucial.

Monitoring and adjustment of new cable forces in the precise tensioning stage

Firstly, the cable force of each cable in the two groups is measured using the dynamic measurement method. The measured cable forces are then compared with the design cable forces, and the construction unit is informed of the necessary adjustments for each cable force. The process of adjusting the cable forces involves manipulating the anchor nut to attain the desired cable force state. This is not a one-time process but requires multiple iterations of adjustments. After each adjustment, the cable force of each cable is remeasured and compared again with the design cable force. Through continuous fine-tuning, our ultimate objective is to minimize the difference between the measured and design cable forces of each cable to a predefined limit value. By doing so, we can ensure the stability and safety of structures like cable-stayed bridges. New suspension is shown in Figure 8.

（a）



（b）

Fig. 8 - New suspension：（a）Real image，（b）Mounting diagram

FINITE ELEMENT ANALYSIS

Model building

Midas/Civil finite element modeling software was utilized to create a spatial model for comprehensive calculation and analysis. The discrete diagram of the computational structure is presented in the accompanying figure. Within this model, the superstructure was subject to calculations, where the main beam and pier of the entire bridge were represented by two distinct elements, while the cables were simulated using truss elements. Specifically, the bridge comprised 172 units and 178 nodes. The elastic modulus of the cable is 2×10^5 MPa. The bottom surface of the bridge piers is in a consolidated form. An elastic connection was employed between the main beam and pier clumn. The finite element diagram depicting this arrangement is illustrated in Figure 9.

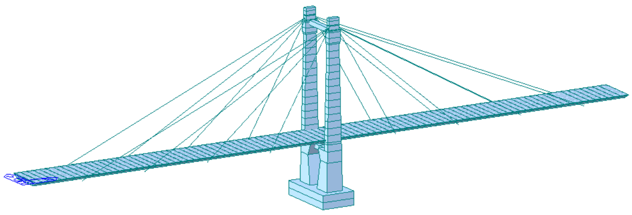


Fig. 9 - Overall calculation model

Result of finite element calculation

Under Load Combination 1, the stress diagrams for the upper and lower edges of the steel box girder are depicted in Figures 10 and 11, respectively. The upper edge experiences maximum tensile and compressive stresses of 39.80 MPa and -9.75 MPa, while the lower edge experiences maximum tensile and compressive stresses of 54.10 MPa and -30.80 MPa.

Under Load Combination 2, the corresponding stress diagrams for the upper and lower edges of the steel box girder are presented in Figures 12 and 13. In this scenario, the upper edge has maximum tensile and compressive stresses of 19.00 MPa and -40.3 MPa, respectively, while the lower edge experiences maximum tensile and compressive stresses of 45.9 MPa and -53.4 MPa, respectively. Notably, both the tensile and compressive stresses on the lower edge are below the design strength value of 310 MPa.

DOI 10.14311/CEJ.2026.02.0013

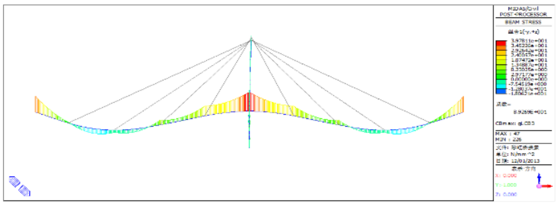


Fig. 10 - Upper edge stress diagram (load combination 1)

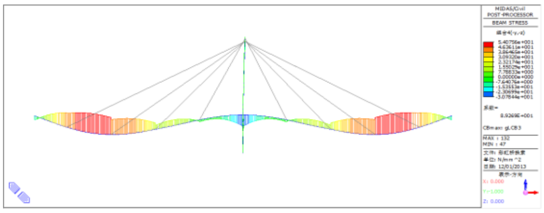


Fig. 11 - Lower edge stress diagram (Load combination 1)

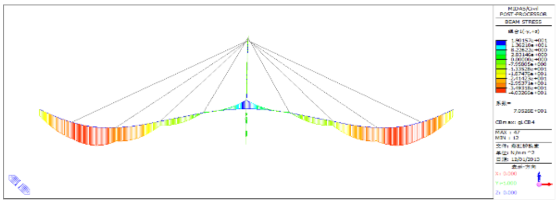


Fig. 12 - Upper edge stress diagram (Load combination 2)

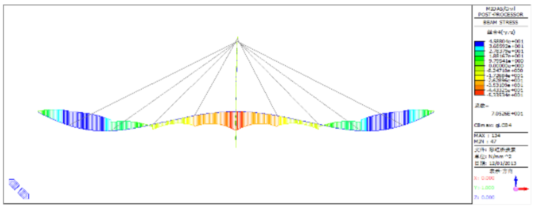


Fig. 13 - Lower edge stress diagram (Load combination 2)

Under load conditions, the vertical displacement diagram of the steel main beam is shown in Figure 14. The maximum deflection of the main beam is 31.9mm, and the allowable deflection value is 78.75 mm, which is less than the allowable deflection value, indicating that the strength meets the requirements. When the main beam deflection is small and the stiffness is good, it usually means that the bridge structure has excellent bearing capacity and stability, and can resist changes caused by external loads. When the main beam deflection is small and the stiffness is good, it usually means that the bridge structure has excellent bearing capacity and stability, and can resist changes caused by external loads. The horizontal displacement diagram of the tower is shown in Figure 15. The maximum horizontal displacement of the tower is 5.04 mm, and the allowable horizontal displacement is 157.5 mm, which indicates that the bridge tower has sufficient stiffness. When the measured deformation of the tower is smaller than the theoretical deformation and the stiffness meets the requirements, it usually indicates that the performance of the bridge structure in actual operation is better than expected, and it has sufficient bearing capacity and stability. When the rigidity meets the requirements, it indicates that the structure can maintain a stable shape under load and is not prone to excessive deformation. The rigidity meeting the requirements means that the structure has sufficient safety in actual operation and can resist the influence of various adverse factors.

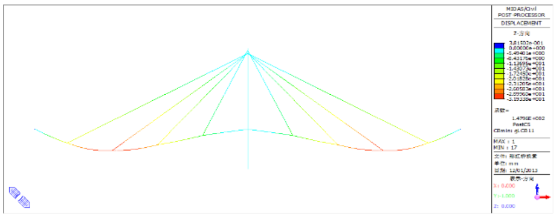


Fig. 14 - Deflection diagram of steel box girder

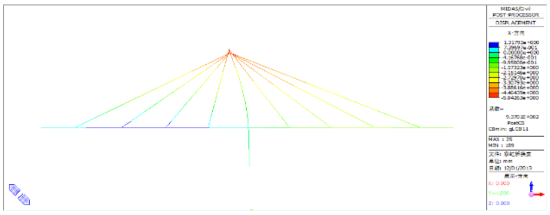


Fig.15 - Displacement diagram of concrete bridge tower

DOI 10.14311/CEJ.2026.02.0013

DETECTION RESULT ANALYSIS

Monitoring results of cable force during cable replacement

The measured cable force of the original cable, prior to replacement and following temporary cable tensioning, is depicted in Figure 16. Conversely, Figure 17 illustrates the measured cable force of the new cable after it has been stretched and the temporary cable has been removed. Prior to cable replacement, the measured cable force distribution lacked a discernible pattern and exhibited asymmetry. Specifically, the maximum cable force was observed in 3 AN#, while the minimum was recorded in 4 AN#. Given that the bridge's cable-stayed cables had been in operation for 30 years before replacement, some degree of damage had occurred, resulting in varying degrees of downward torsion in the main beam. Consequently, the cable force distribution was deemed normal under these circumstances.

Before cable replacement, if the measured cable force distribution was found to lack a discernible pattern and displayed left-right asymmetry, it typically indicated an abnormal cable force distribution within the bridge structure. During bridge operation, external factors such as pedestrian and wind loads can cause uneven cable force distribution. This becomes more pronounced when such loads are unevenly distributed across the left and right sides of the bridge, leading to more pronounced left-right asymmetry in the measured cable force. Uneven cable force distribution can impact the bridge's bearing capacity, particularly when certain cable forces become excessively large, potentially damaging the cables and their connecting components, thereby compromising the bridge's overall structural integrity.

To align the bridge's actual stress state with the design specifications, slight adjustments to the cable forces are necessary to ensure that the new cable forces meet the stipulated requirements.

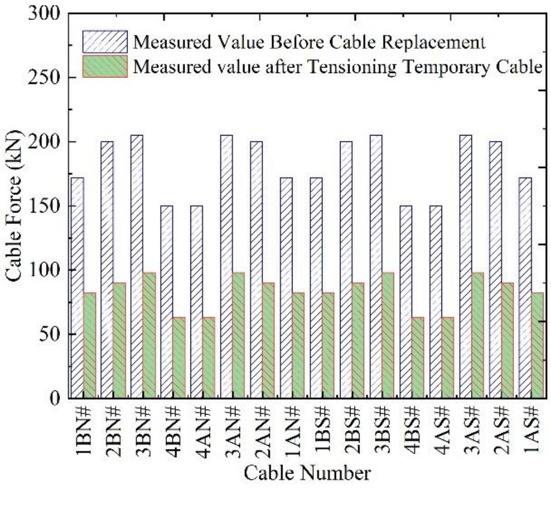


Fig. 16 - Measured cable force of the original cable (before cable replacement and after temporary cable tensioning)

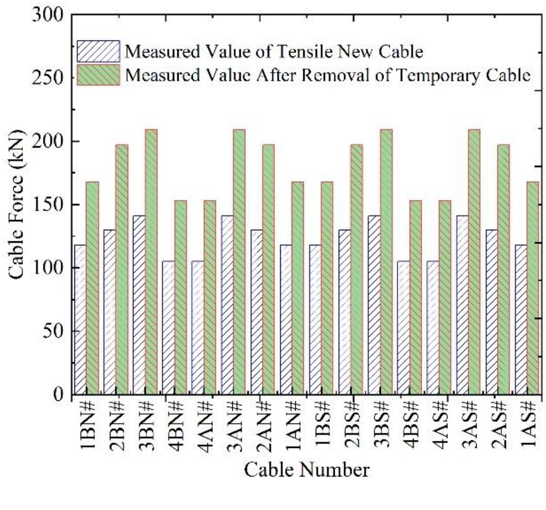


Fig. 17 - Measured cable force of new cable (after stretching new cable and removing temporary cable)

The measured cable force of the temporary steel cable is depicted in Figure 18. Upon stretching the temporary cable, the weight of the steel box girder is jointly supported by both the temporary and old cables, resulting in a significant reduction in the original cable force, ranging from 1.5% to 69.5%. When a new cable is installed in the bridge structure, it disrupts the existing cable force balance.

The incorporation of this new cable alters the stress state of the entire structure, leading to a redistribution of cable forces. This redistribution influences not only the new cable's force but also the forces of other cables, including the temporary ones.

After the removal of the original cable, the temporary cable force begins to increase, with a maximum increase rate of 22.2%. Once the old cable is removed, the entire dead weight of the steel box girder is borne solely by the temporary cable, and the increase in cable force meets the design specifications. Subsequently, upon installing the new cable, the temporary cable force decreases by 34.8% to 48.7%. The installation of the new cable results in it sharing some of the load previously borne by the original cable (including the temporary cable), thereby reducing the load on the temporary cable and decreasing its force.

The addition of new cables may cause some deformation in the bridge structure. This deformation alters the geometry and length of the cables, subsequently affecting their forces. In the case of a temporary cable, if its length changes (such as shortening), it will lead to a decrease in its cable force. Once the temporary cable is removed, the dead weight of the steel box girder is fully supported by the new cable, causing the cable force to increase by 36.5% to 72.8% compared to its original state. Ultimately, the cable force approaches the design cable force value.

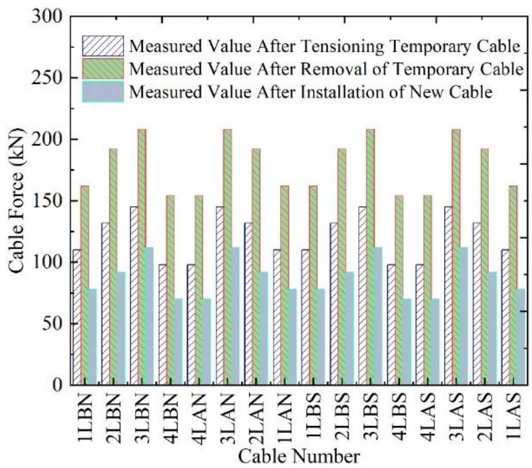


Fig. 18 - Measured cable force of temporary cable

Result of cable force after cable replacement

According to the design requirements, after the end of each working condition, all the cable force must be monitored to check whether the cable force after installation is close to the design value.

The ratio between the measured cable force and the designed cable force after cable replacement is shown in Figure 19. The elastic modulus, cross-sectional area and other parameters of the cable material may be different from the designed values, resulting in the cable force deviation. The nonlinear behavior and relaxation factors of cable may be different in practical application, which is not consistent with the design expectation. After cable replacement, the final cable force of the bridge has a certain deviation from the designed cable force. In order to make the stress state of the bridge after cable replacement consistent with the designed internal force, it is necessary to further adjust the cable force. The cable force deviation may affect the stability of the bridge structure,

CIVIL ENGINEERING JOURNAL

Article no. 13

THE CIVIL ENGINEERING JOURNAL 2-2026

resulting in structural instability or excessive vibration. The cable force deviation may affect the stability of the bridge structure, resulting in structural instability or excessive vibration. It is a complicated phenomenon that there is a certain deviation between the final cable force and the designed cable force after cable replacement, and many factors need to be considered comprehensively. In order to ensure the stability and bearing capacity of the bridge structure, a series of countermeasures are needed to reduce the deviation and strengthen the monitoring and maintenance work. After the cable adjustment is completed, the error between the measured value and the design value of the cable is controlled within 2%, which is very close to the design cable force, as shown in Figure 20.

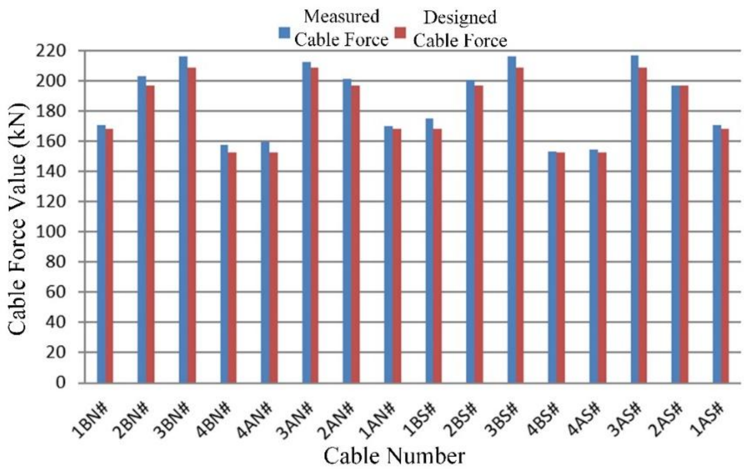


Fig. 19 - Ratio between measured cable force and designed cable force after cable replacement

From the comparison diagram contrasting the measured cable force with the design value, it is evident that the majority of cable forces closely align with the design cable force. However, a few cable forces exhibit a slight deviation, surpassing the design value. The maximum discrepancy between the measured and design values amounts to 4.4%.

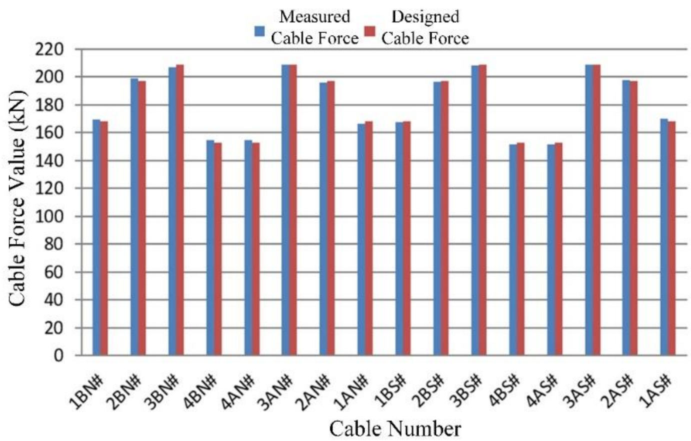


Fig. 20 - Cable force value of the new cable

The actual measured cable force of the cable is close to the design cable force, the maximum is 1.1%, and the cable force is basically close to the design value, not more than 2% of the code requirements. The close approximation between the cable force and the design value indicates that the bridge structure is in accordance with the design expectation in terms of force, thus ensuring the safety of the structure. This usually means that the cable force adjustment of the bridge has reached a relatively ideal state. However, it is still necessary to continuously monitor and maintain the safety of bridge structures, and constantly improve the design and technical methods to improve the quality of engineering. After the cable has just been replaced, the actual measured cable force is not much different from the designed cable force, and the force of the whole bridge cable is relatively uniform.

Monitoring and analysis of elevation during cable replacement

During the cable replacement construction process, Figures 21 and 22 illustrate the elevation changes on the south and north sides of the steel box girder, respectively. The main beam's elevation undergoes notable variations under different operational conditions. Prior to cable replacement, the steel box girder of the cable-stayed bridge exhibited a distinct downward deflection, with the bridge's stiffness being considerably lower than that of the completed structure. The maximum mid-span deflection reached 13 cm.

Over the extended usage of bridge materials, such as steel, aging and deformation occur due to environmental and age-related factors. The deterioration of material properties, like a decrease in the elastic modulus, contributes to a reduction in the bridge's stiffness. Persistent and repetitive loads can cause vibrations and displacements in the bridge, leading to downward deflection. Construction errors and imperfections, such as inadequate welding quality and inaccurate component dimensions, may also compromise the stiffness and stability of the bridge. Environmental variables, including temperature and humidity fluctuations during construction, can impact the bridge structure, resulting in downward torsion.

During the tensioning and removal of temporary cables, the mid-span deflection of the steel box girder increased significantly, indicating that the elevation of the main beam improved post-cable replacement, enhancing the overall stiffness of the bridge. After stretching the temporary cables, the maximum deflection of the steel box girder rose by 0.095 m.

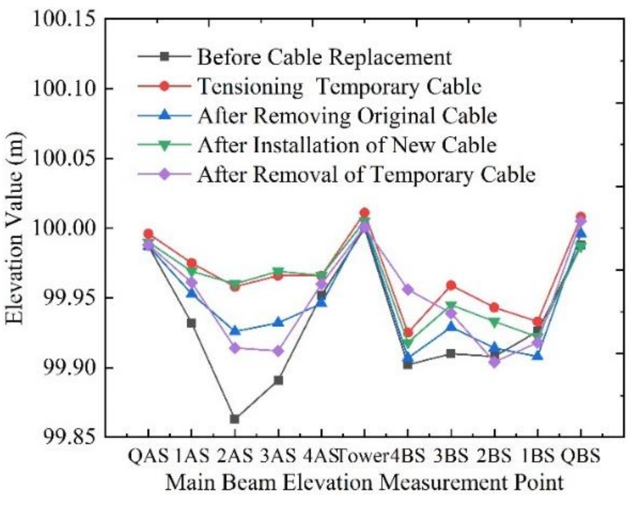


Fig. 21 - Elevation change value on the south side of the main beam during cable replacement

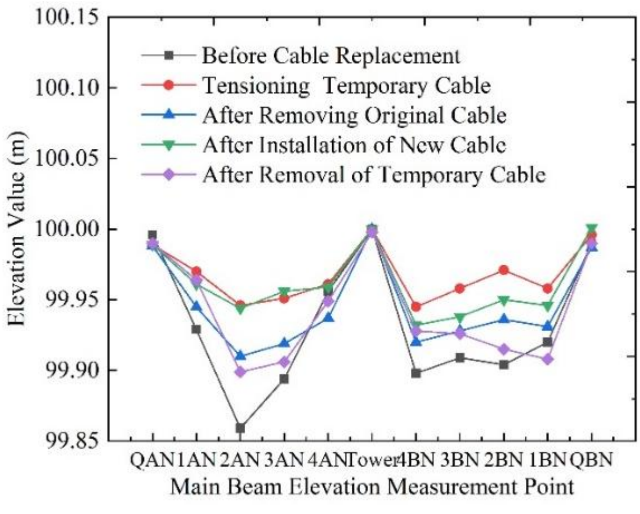


Fig. 22 - Elevation change of the north side of the main beam during cable replacement

DOI 10.14311/CEJ.2026.02.0013

After removing the original cable, the relative deflection of the steel box girder is reduced by 0.036m. After the temporary cable is removed, the relative deflection of the steel box girder is reduced by 0.057 m. After the main beam elevation is changed, the mid-span deflection of the steel box beam is increased by 0.054 m, the linear shape of the main beam is improved, and the design requirements are met. In the process of cable replacement, the stress state of the main beam can be changed significantly by adjusting the cable force of the cable, so as to improve its linear shape. Especially for those main beams that already have obvious downwarping or deformation, cable replacement is an effective repair and reinforcement method. The cable replacement not only improves the alignment of the main beam, but also improves the overall performance of the bridge. This helps extend the service life of the bridge, reduce maintenance costs, and improve the safety and reliability of the bridge.

CONCLUSION

1. Based on an analysis of the cable replacement control process, the characteristics of various construction parameters across different stages have been examined. The results indicate that the trend of cable force changes aligns closely with theoretical values, while the alignment of the main beam and deflection of the main tower remain within reasonable limits.

2. The discrepancy between the measured cable force and the theoretical maximum cable force stands at 1.1%, adhering to the standard requirement of no more than 2%. The linear shape of the main beam has significantly improved, with a maximum elevation deviation of 4.9 cm, enhancing pedestrian comfort. Furthermore, the maximum offset value of the main tower is 1.3 mm, which is within standard limits at the initiation of construction. Hence, the cable replacement process fulfills the design specifications.

3. Given the site constraints and the nature of the bridge as a roadway, the cable-stayed bridge in this project is of modest scale. Consequently, a temporary cable-stayed system has been employed for cable replacement. This approach has proven feasible and may serve as a valuable reference for similar endeavours.

REFERENCE

[1] Fu Z, Ji B, Yang M,. 2015. Cable replacement method for cable-stayed bridges based on sensitivity analysis. Journal of Performance of Constructed Facilities, vol. 29, n. 3, p. 04014085. ttps://doi.org/10.1061/(ASCE)CF.1943-5509.0000460

[2] Mehrabi A B,. 2016. Performance of cable-stayed bridges: Evaluation methods, observations, and a rehabilitation case. Journal of Performance of Constructed Facilities, vol. 30, n. 1, p. C4014007. https://doi.org/10.1061/(ASCE)CF.1943-5509.0000715

[3] Yang Y, Wang X, Wu Z,. 2020. Long-span cable-stayed bridge with hybrid arrangement of FRP cables. Composite Structures, vol. 237, p. 111966. https://doi.org/10.1016/j.compstruct.2020.111966

[4] Wang X, Wu Z,. 2010. Integrated high-performance thousand-metre scale cable-stayed bridge with hybrid FRP cables. Composites Part B: Engineering, vol. 41, n. 2, p.166-175. https://doi.org/10.1016/j.compositesb.2009.09.001

[5] Mehrabi A B,. 2006. In-service evaluation of cable-stayed bridges, overview of available methods and findings. Journal of Bridge Engineering, vol. 11, n. 6, p. 716-724. https://doi.org/10.1061/(ASCE)1084-0702(2006)11:6(716)

[6] Xie X, Li X, Shen Y,. 2014. Static and dynamic characteristics of a long-span cable-stayed bridge with CFRP cables. Materials, vol. 7, n. 6, p. 4854-4877. https://doi.org/10.3390/ma7064854

[7] Martins A M B, Simões L M C, Negrão J H J O,. 2020. Optimization of cable-stayed bridges: A literature survey. Advances in Engineering Software, vol. 149 , p. 102829. https://doi.org/10.1016/j.advengsoft.2020.102829

[8] Li H,. 2020. Structural assessment of concrete cable-stayed bridge after replacement of closure segment: The service stage. Practice Periodical on Structural Design and Construction, vol. 25, n.3, p. 04020023. https://doi.org/10.1061/(ASCE)SC.1943-5576.0000508

[9] Li H, Ou J,. 2016. The state of the art in structural health monitoring of cable-stayed bridges. Journal of Civil Structural Health Monitoring, vol. 6, p.43-67. https://doi.org/10.1007/s13349-015-0115-x

[10] Zhao Y, Wu J, Wang T,. 2020. Cable Replacement Scheme for Low Tower Cable-Stayed Bridges Based on Sensitivity Analysis. Tehnički vjesnik, vol. 30, p. 5, p. 1603-1610. https://doi.org/10.17559/TV-20230227000385

[11] Sandberg J, Hendy C R,. 2010. Replacement of the stays on a major cable-stayed bridge. Proceedings of the Institution of Civil Engineers-Bridge Engineering. Thomas Telford Ltd, vol. 163, n. 1, p. 31-42. https://doi.org/10.1680/bren.2010.163.1.031

[12] Feng B, Wang X, Wu Z,. 2019. Fatigue life assessment of FRP cable for long-span cable-stayed bridge. Composite Structures, vol. 210, p.159-166. https://doi.org/10.1016/j.compstruct.2018.11.039

[13] Zeng Y, Zheng H, Jiang Y,. 2022. Modal analysis of a steel truss girder cable-stayed bridge with single tower and single cable plane. Applied Sciences, vol. 12, n. 15, p. 7627. https://doi.org/10.3390/app12157627

[14] Akhoondzade-Noghabi V, Bargi K,. 2016. Study on different cable configurations of cable-stayed bridges using developed seismic risk assessment. Structural Engineering International, vol. 26, n. 4, p. 312-323.

[15] Lu N, Liu Y, Beer M,. 2021. System reliability evaluation of in-service cable-stayed bridges subjected to cable degradation. Reliability and Safety of Cable-Supported Bridges, p. 59-84.

[16] Huang Y, Wang Y, Fu J,. 2018. Measurement of the real-time deflection of cable-stayed bridge based on cable tension variations. Measurement, vol. 119, p. 218-228. https://doi.org/10.1016/j.measurement.2018.01.070

[17] Wen J, Han Q, Xie Y,. 2021. Performance-based seismic design and optimization of damper devices for cable-stayed bridge. Engineering Structures, vol. 237, p. 112043. https://doi.org/10.1016/j.engstruct.2021.112043

[18] Ma Y, Peng A, Wang L,. 2023. Structural performance degradation of cable-stayed bridges subjected to cable damage: Model test and theoretical prediction. Structure and Infrastructure Engineering, vol. 19, n. 9, p. 1173-1189.

[19] Wang X, Wu Z,. 2010. Evaluation of FRP and hybrid FRP cables for super long-span cable-stayed bridges. Composite Structures, vol. 92, n. 10, p. 2582-2590. https://doi.org/10.1016/j.compstruct.2010.01.023

[20] Hoang V, Kiyomiya O, An T,. 2018. Experimental and numerical study of lateral cable rupture in cable-stayed bridges: Case study. Journal of Bridge Engineering, vol. 23, p. 6, p. 05018004. ttps://doi.org/10.1061/ (ASCE)BE.1943-5592.0001227