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Segmental Model Test on Ultimate Bearing Capacity of a Cable Tower Anchorage Zone in a Cable-Stayed Bridge

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Abstract: To determine the ultimate bearing capacity of the anchorage zone in a long-span cable-stayed bridge, a typical segment of the anchorage zone was selected for horizontal force mechanism analysis and model loading test, and its load-carrying behavior and deformation characteristics were subsequently analyzed. Results show that under the action of horizontal cable force, the bending moment is relatively large at the outer face of the end tower wall and inner face of the side tower wall, and the chamfer between the side and end walls becomes a critical section because of the combined action of tension, shear and bending. The failure mode is primarily concrete cracking: cracks appear in the end tower wall at 0.15P and penetrate the full height of the segment at 0.25P, while full-depth vertical cracks appear in the side tower wall at 0.20P. The horizontal load of the segmental mode increases nonlinearly with the deformation of the tower wall, and the ultimate bearing capacity reaches 329.70 kN, approximately 0.48P, indicating that the concrete tower wall alone cannot sustain the enormous horizontal cable force. Compared with the side tower wall, the concrete of the end tower wall is more sensitive to the horizontal cable force load. As the horizontal load increases, the load carried by the concrete is gradually transferred to the reinforcement because of the tensile cracking of the tower wall.

Keywords: composite bridge; cable tower anchorage zone; ultimate bearing capacity; concrete tower wall; model test

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Scientific Research

1 Introduction

In China, concrete cable towers are widely adopted in long-span cable-stayed bridges, and the tower-end anchorage of stay cables can be realized using circumferential prestressing tendons or composite anchorage systems such as steel anchor beams and steel anchor boxes [1-2]. Enormous cable force is transmitted through the confined space of the anchorage zone, subsequently diffused across the full cross-section of tower, and then transferred downward to the bridge foundation. Consequently, the load-carrying performance of the cable tower anchorage zone plays a critical role in ensuring the overall structural safety of a cable-stayed bridge.

Previous studies have commonly simplified the anchorage zone segment of a cable tower as a frame structure for mechanical performance analysis [3-7]. However, owing to the complex stress state within the cable tower anchorage zone, particularly under extreme loading scenarios such as cable breakage or cable replacement, ultimate load-carrying behavior remains insufficiently understood, and simplified analytical methods exhibit inherent limitations. Model test investigations focusing on the cable tower anchorage zone have typically involved full-scale or large-scale reduced-scale models extracted from the anchorage segment, with stay cable forces being simulated by means of jacks or prestressing to evaluate the structural performance of the anchorage zone [8-10]. Nevertheless, constrained by model dimensions and loading apparatus capacity, such tests are often limited to assessing load-carrying performance merely within the linear elastic stage or at the onset of the elastoplastic stage, failing to reveal mechanical behavior throughout the complete loading-to-failure process and thus to determine the ultimate bearing capacity of the anchorage zone.

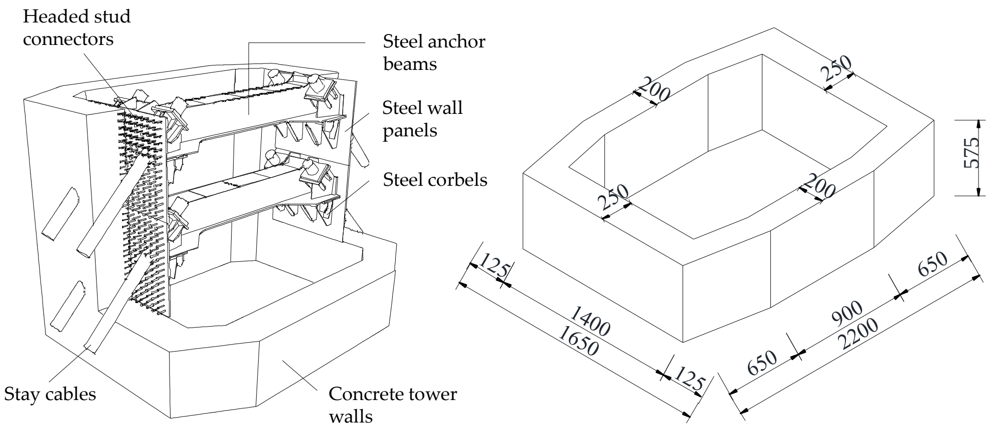
To obtain the ultimate bearing capacity of the cable tower anchorage zone under extreme loading conditions, a reduced-scale segmental model specimen was designed and tested based on a prototype long-span cable-stayed bridge with a main span of 780 m. Because the vertical component of the decomposed stay cable force is transferred via compressive bearing action on the tower wall, which is favorable to the concrete, only the horizontal component of the stay cable force was simulated and applied using jacks. Based on the segmental model test results, the ultimate bearing capacity, frame deformation behavior, and crack resistance performance of the concrete tower wall of the cable tower anchorage zone in the cable-stayed bridge were analyzed.

2 Segmental Model of Cable Tower Anchorage Zone

Because factors such as loading conditions and stress states significantly influence outcomes of model tests, the anchorage zone segment subjected to the maximum stay cable force was selected for model loading test. Considering factors such as the model dimensions, test site conditions, and equipment capacity, a 1/4 scale reduction was applied to the anchorage zone segment, and the segmental model was designed in accordance with similarity principles, as illustrated in Figure 1. Within the cable tower anchorage zone, the stress states of individual segments are similar, and a segmental model with appropriately configured boundary conditions can realistically represent the horizontal mechanical behavior of a single anchorage segment in the prototype structure. Existing studies have extensively investigated the service performances of steel–concrete composite anchorage zones at the linear elastic stage. Once the plastic stage is entered, the load-carrying behavior of steel components can be determined with reasonable certainty through calculation, while the ultimate bearing capacity of a concrete tower wall remains unclear. Therefore, the tower wall was extracted as the primary test object for loading.

Based on geometric similarity conditions, the height of a single segment in the cable tower anchorage zone was scaled down from 2,300 to 575 mm, and the plan dimensions were reduced from 8,800 mm × 7,000 mm to 2,200 mm × 1,750 mm. The thicknesses of the end and side tower walls were scaled from 1,000 and 800 to 250 and 200 mm, respectively (as shown in Figure 1). The reinforcement in the tower walls was designed to maintain the same reinforcement ratio as that of the prototype structure.

According to physical similarity conditions, the model employed materials identical to those used in the prototype, and it was assumed that the stress distributions in both the model and prototype under loading were the same; consequently, the load ratio between the model and prototype was determined to be 1/16. In accordance with boundary similarity conditions, the top surface of the model was set free, and a thin layer of fine sand cushion was placed beneath the bottom surface to minimize frictional effects.



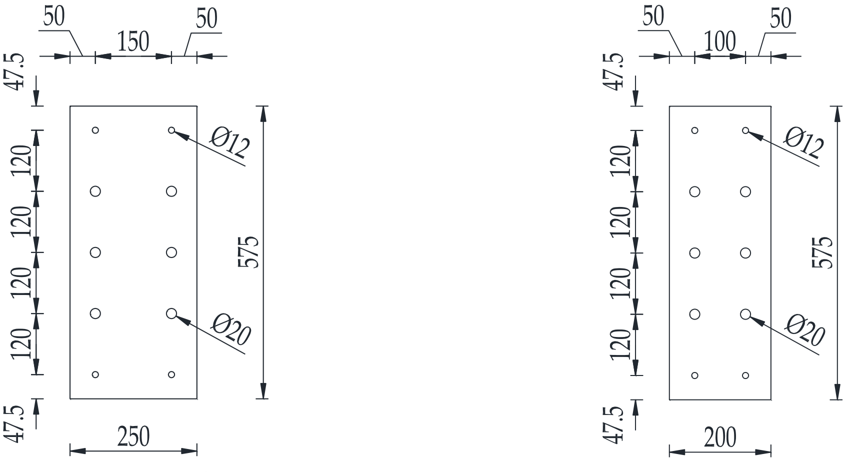
a）Prototype b）Model

Figure 1 Segmental model of cable tower anchorage zone（unit：mm）

3 Horizontal Force Mechanism of Anchorage Zone

3.1 Analysis of Tower Wall Section

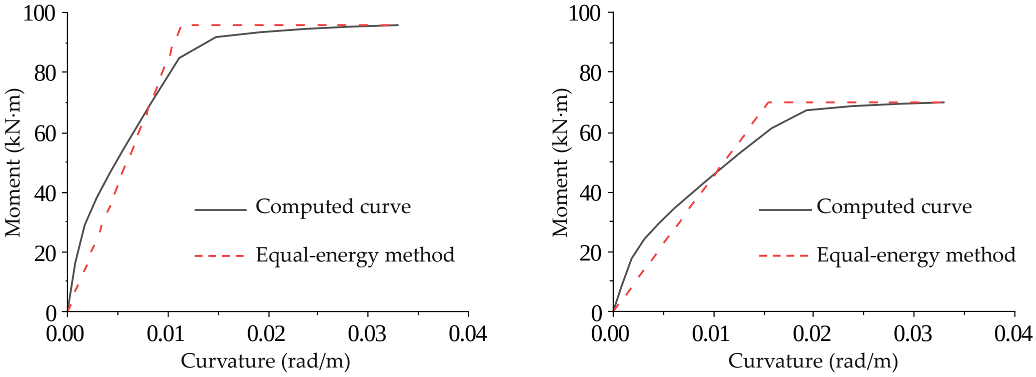
The cross-sectional reinforcement details of the end and side tower walls within the cable tower anchorage zone are illustrated in Figure 2. The concrete used for the tower walls was of Grade C50, with an axial compressive strength of $f_c = 32.4$ MPa and an elastic modulus of $E_c = 34.5$ GPa. The cross-sections were reinforced with HRB400 steel bars of Φ20 and Φ12 mm, possessing a yield strength of $f_y = 400$ MPa, an ultimate tensile strength of $f_s = 570$ MPa, and an elastic modulus of $E_S = 200$ GP. As shown in Figure 2, the stirrups (vertical reinforcement) in the end and side tower walls were Φ12@160 and Φ12@225, respectively. The corresponding stirrup reinforcement ratios were $\rho_{sv} = n \times A_{sv1}/(b \times s)$ =0.246% and 0.175%, respectively. The minimum required stirrup ratio was $\rho_{sv,min} = 0.24 \times f_t/f_{yv} = 0.126\%$. It is verified that $\rho_{sv} > \rho_{sv,min}$ is satisfied for both the walls, thereby meeting the cross-sectional shear resistance requirements.



a）End tower wall b）Side tower wall

Figure 2 Tower wall cross-section in anchorage zone（unit：mm）

The Csicol program is a cross-sectional analysis software developed by Computers and Structures, Inc. (the developer of SAP2000) and is based on the finite element theory. Its core algorithms have been validated through engineering practices. Additionally, it can accurately compute the geometric properties of member cross-sections as well as stress distributions under various load combinations, making it suitable for mechanical analysis of both regular and complex cross-sectional configurations. The cross-sectional flexural capacity of the concrete tower wall in the cable tower anchorage zone was analyzed using the Csicol program, and the results are presented in Figure 3. In the figure, Curve 1 represents the computed moment – curvature relationship, and Curve 2 denotes the idealized elastic – perfectly plastic bilinear approximation obtained using the equal-energy method. From the moment–curvature relationship, the yield moment of the end tower wall is $M_{yl} = 84.93$ kN·m, and its peak moment is $M_{ul} = 96.05$ kN·m. Meanwhile, the yield moment of the side tower wall is $M_{y2} = 61.23$ kN·m, and its peak moment is $M_{u2} =$ 69.93 kN·m.



a）End tower wall b）Side tower wall

Figure 3 Analysis of tower wall cross-section

3.2 *Analysis of Yield Load*

As illustrated in Figure 4, the segment of the cable tower anchorage zone was simplified as a planar frame model to analyze the force mechanism under the action of the horizontal component of the stay cable force. Considering symmetry, a half-structure was adopted for modeling, with symmetry constraints imposed on the symmetry axes. The symmetry axis of the side tower wall was constrained against the longitudinal displacement and rotation, while that of the end tower wall was constrained against the transverse displacement. A unit horizontal cable force was initially applied at the midpoint of the end tower wall, yielding a maximum bending moment of 0.3012 kN·m. The yield load, F_y, was then determined as the ratio of the yield moment of the end tower wall (84.93 kN·m) to this unit moment, obtaining $F_y = 282.00$ kN.

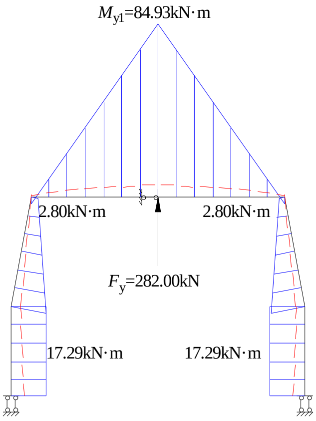


Figure 4 Yield load analysis of anchorage zone

3.3 *Peak Load Analysis*

Based on the sectional analysis results of the tower walls, the yield load and peak load of the end tower wall in the segmental model of the cable tower anchorage zone are 1.39 and 1.37 times those of the side tower wall, respectively. As revealed by yield load analysis, the end tower wall of the segmental model sustains a significantly larger bending moment, approximately five times that of the side tower wall. Thus, the failure of the segmental model of the cable tower anchorage zone is inferred to be initiated by the loss of the flexural capacity of the end tower wall, while the side tower wall has not yet reached its ultimate flexural state. As shown in Figure 5, the peak load $F_u = 318.94$kN was obtained by dividing the peak bending moment of the end tower wall, 96.05 kN·m, by the bending moment induced per unit horizontal cable force, 0.3012 kN·m.

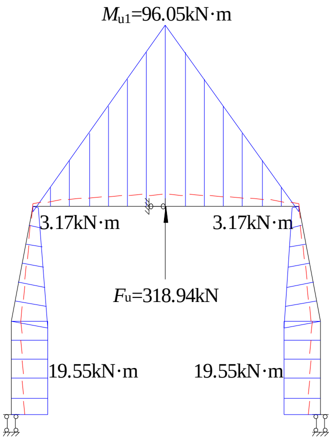


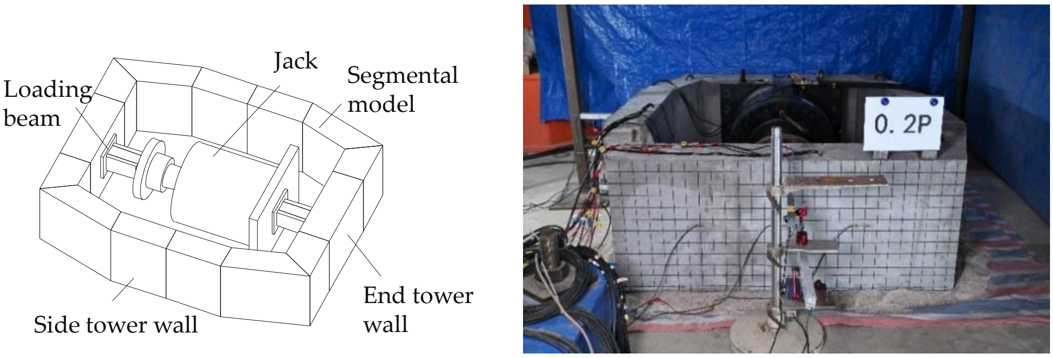
Figure 5 Peak load analysis of anchorage zone

4 Model Loading and Testing Scheme

4.1 Model Loading Scheme

As shown in Figure 6, a unidirectional loading scheme was adopted, in which the horizontal load was applied through self-balanced compression using a 5,000 kN servo-hydraulic jack. In the prototype structure corresponding to the segmental model, the force of a single stay cable is 6,390 kN. Considering the 1/4 geometric scale applied to the anchorage zone segment and principle of stress equivalence, the single-cable force load acting on the loading beam at a scale of 1× is calculated as 6,390 × 1/16 = 399.375 kN. Based on the angles between the stay cable and horizontal and vertical planes, the horizontal component of the single-cable force is further obtained as 345.75 kN through double projection. Because stay cables are arranged in a double-cable plane configuration, the design value of the horizontal load applied in the test is $P = 345.75 \times 2 = 691.50$ kN.

During installation of the model test, the center of the jack loading end was



a）Loading schematic b）Site layout

Figure 6 Model loading scheme

aligned with the transverse centerline of the bridge, and the loading end plane was kept vertical while ensuring close contact with the outer plate plane of the loading beam to maintain a relatively small horizontal thrust. A 3 cm thick cushion layer of fine sand was placed beneath the bottom surface of the segmental model to minimize the influence of the laboratory floor on the horizontal mechanical behavior of the model.

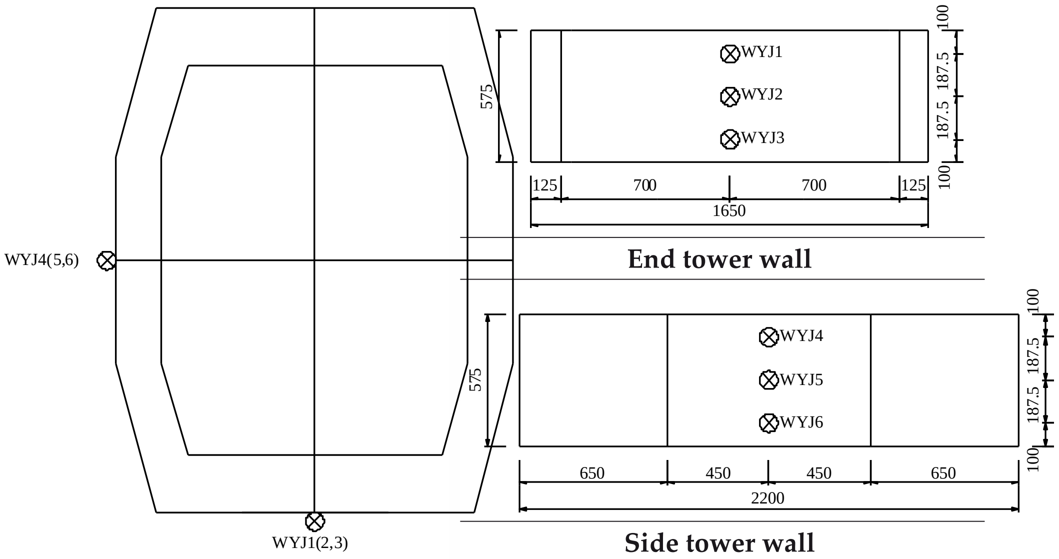
Prior to formal loading, 0.05 times the design load was applied to ensure the safety and reliability of the loading equipment; to calibrate the load, displacement, and strain acquisition instruments; and to eliminate initial nonlinear effects. The load value and loading stroke were continuously output and recorded using the jack controller.

During formal loading, the load was applied in increments of 0.05P per step. After each loading increment, the load was held constant for 5 min to allow the readings of the loading equipment, strain acquisition system, and displacement transducers to stabilize before data recording. Concurrently, crack development was observed in the concrete tower walls.

4.2 *Data Measurement Scheme*

As shown in Figure 7, to investigate the deformation behavior of the segmental model under horizontal loading, three displacement transducers were arranged along the vertical centerline on the outer face of the end tower wall and another three on the outer face of the side tower wall to measure the deformation of the segmental model.

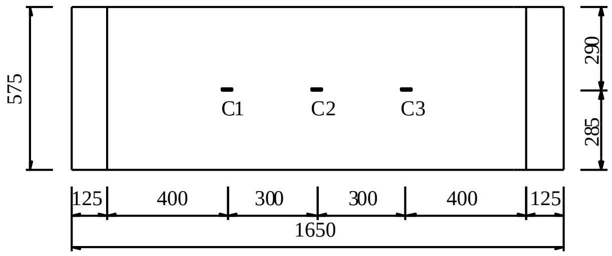
Figure 7 Deformation measurement scheme（unit：mm）



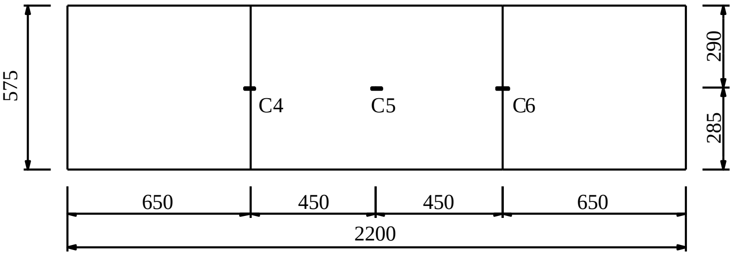
a) Plan layout b) Elevation layout

Under the action of the horizontal cable force, the maximum bending moment in the segmental model of the cable tower occurs along the axes of symmetry of the end and side tower walls, with the outer face of the end tower wall and inner face of the side tower wall subjected to tension. Accordingly, horizontal strain gauges were installed on the concrete surface at these two cross-sections to measure the stress distribution on the surface of the concrete tower walls, as shown in Figure 8.

Owing to the relatively low tensile strength of concrete, when the tensile stress in the tower wall concrete exceeds its tensile strength, cracks will form on the wall surface, the concrete will gradually cease to carry tension, and the tensile force sustained by the reinforcement will correspondingly increase. For this reason, intermediate-layer longitudinal-load-carrying reinforcing bars were selected from



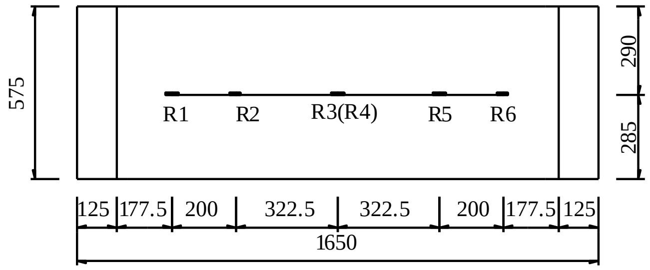
a）End tower wall



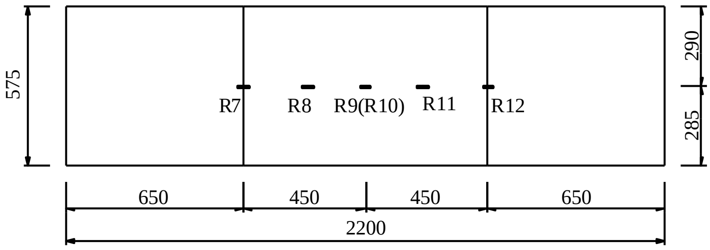
b）Side tower wall

Figure 8 Concrete strain measurement scheme（unit：mm）

the reinforcement cage of the segmental model, and strain gauges were attached at locations experiencing relatively high stress to measure the stress distribution in the ordinary steel reinforcement within the end and side tower walls, as shown in Figure 9.



a）End tower wall



b）Side tower wall

Figure 9 Reinforcement strain measurement scheme（unit：mm）

4.3 *Crack Observation Scheme*

Prior to loading test, a grid with a side length of 50 mm was marked on both the inner and outer concrete surfaces of the end and side tower walls of the segmental model using a marker pen, as shown in Figure 6. During the loading process, particular attention was given to monitoring the outer surface of the end tower wall concrete, the inner surface of the side tower wall concrete, and any other locations where cracks might appear. The crack distribution diagrams were constructed, variation in cracks with increasing load was monitored, and load level corresponding to appearance of each crack was recorded, thereby enabling determination of the cracking load of the segmental model of the cable tower anchorage zone.

5 Model Test Results and Analysis

5.1 Failure Mode of Segmental Model

The failure mode and progression of the cable tower anchorage zone under horizontal cable force loading, primarily characterized by gradual cracking of the concrete in the end and side tower walls with increasing load level are presented in Table 1 and Figure 10. The segmental frame model of the cable tower anchorage zone was tested under self-balanced loading, and initial cracks propagated rapidly upon appearance, making it impossible to measure the initial crack width. After complete failure of the specimen, the crack width at the centerline of the outer surface of the end tower wall is measured as 15 mm and that at the right side of the outer surface is 18 mm. The chamfer between the side and end tower walls, subjected to the combined action of tension, shear, and bending, constituted a critical section.

5.2 Load–Deformation Relationship Curve

As shown in Figure 11, the horizontal load of the segmental model of the cable tower anchorage zone exhibits a nonlinear relationship with the tower wall deformation. In the early stage of loading, the stiffness of the model specimen is relatively high, and the applied load is essentially linearly related to deformation. With the progressive increase in the horizontal load, the deformation of the tower wall continues to increase. Once the horizontal load exceeds the yield load, the stiffness of the model specimen gradually decreases, whereas the displacement continues to increase with further loading. This behavior is attributed to cracking of the concrete tower wall, whereby cracks gradually propagate from the outer face of the end tower wall toward the inner face, accompanied by an increase in the stress carried by the tower wall reinforcement. After reaching the peak load point, the horizontal load sustained by the model specimen decreases.

The load – deformation relationship curve of the cable tower anchorage zone shown in Figure 11 is simplified to an idealized elastoplastic response. Using the equal energy method, the yield load is calculated as 277.70 kN. The secant stiffness corresponding to the yield load is taken as the initial stiffness, which is 14.14 kN/mm, and the corresponding tower wall deformation is defined as the yield deformation (19.64 mm). The peak load on the load–deformation curve is 329.70 kN, and the corresponding tower wall deformation is the peak deformation (53.88 mm). The failure deformation of the model specimen when loading is terminated is 61.15 mm, which is 2.74 times the yield deformation; this ratio is taken as the ductility factor.

From the model test results, the peak load is regarded as the ultimate bearing capacity of the segmental model of the cable tower anchorage zone, which is 329.70 kN, approximately 0.48 times the design value of the horizontal cable force load. This finding indicates that in the anchorage zone of a long-span cable-stayed bridge, the concrete tower wall alone cannot sustain the enormous horizontal component of the stay cable force. If a circumferential prestressing anchorage scheme is adopted, a large number of prestressing tendons will be required, which may be impractical because of construction constraints. If composite anchorage schemes such as steel anchor beams or steel anchor boxes are employed, it is necessary to thoroughly verify extreme load cases such as cable breakage or cable replacement to prevent accumulation of excessively high local stress in the tower wall concrete, which can lead to cracking and crushing.

5.3 Stress State of Tower Wall Concrete

The variation in the concrete stress in the tower wall of the segmental model with increasing load level is illustrated in Figure 12. As shown in Figure 12 a), at a

Table 1 Failure mode of segmental model

Load level	Failure of end tower wall	Failure of side tower wall
0.15P	A fine crack with an approximate length of 23 cm appeared on the upper surface extending upward from the midpoint of the outer face.	No significant phenomenon observed.
0.20P	An approximately 18-cm long crack appeared at the centerline of the upper surface, propagating from the outer to the inner side.	Two vertical through-thickness cracks appeared on the left side of the outer face, extending to the upper surface.
0.25P	A through-thickness crack along the height direction appeared at approximately one-quarter of the width on the right side of the concrete.	Three vertical fine cracks appeared on the inner face, two of which had a length of approximately 40 cm, with one located at the corner.
0.30P	Slight cracking sounds were heard; a through-thickness crack along the height direction appeared at approximately one-quarter of the width on the left side of the concrete and propagated inward.	Two additional vertical cracks, with an approximate length of 40 cm, appeared on the inner face.
0.35P	Crack at the middle of the outer face of the concrete showed a tendency to expand.	One additional vertical through-thickness crack appeared on the outer face and extended to the top surface; a horizontal branch crack appeared at the lower part.
0.40P	Continuous cracking sounds were heard; the middle crack on the outer face of the concrete continued to propagate inward on the inner side of the upper surface.	Crack at the left corner expanded, and concrete spalling occurred on the upper surface and at the bottom.
0.45P	Crushing appeared above the loading beam on the inner face; a horizontal crack with a length of approximately 30 cm appeared on the opposite side, leading to spalling.	Severe concrete spalling occurred on the outer face of the side tower wall, with small pieces of concrete falling off.
0.50P	Cracks on the upper surface of the concrete extended to the loading end on the inner face; localized concrete crushing occurred; multiple horizontal fine cracks appeared along the original vertical crack on the outer face of the end tower wall.	Horizontal spalling appeared at the bottom of the left corner on the inner face of the side tower wall.

load level of 0.05*P*, the stress at the mid-height of the centerline on the outer surface of the end tower wall concrete is 1.88 MPa. At 0.15*P*, cracks begin to appear in the concrete on both lateral sides of the end tower wall. As shown in Figure 12 b), prior to applying 0.25*P*, the outer surface concrete of the side tower wall is subjected to tension, with a maximum tensile stress of 2.75 MPa. As the load level increases, the concrete tends toward compression, reaching a maximum compressive stress of −13.22 MPa at 0.40*P*. The test results indicate that the concrete of the end tower wall is more sensitive to the horizontal cable force load than the side tower wall, and appropriate measures should be taken to enhance its crack resistance performance.

a) End tower wall b) Side tower wall

Figure 10 Failure mode of segmental model

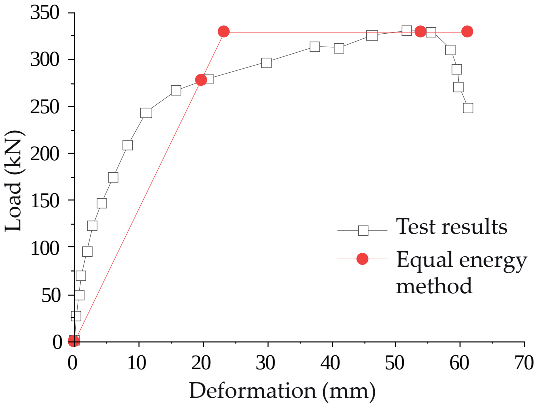
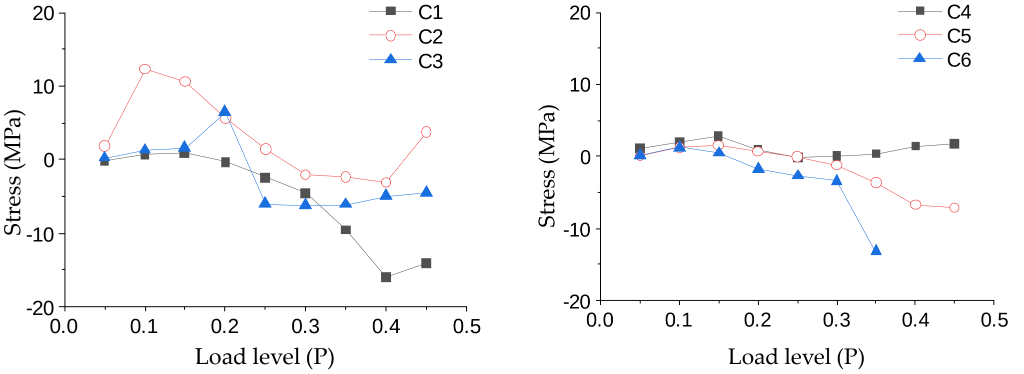


Figure 11 Load–deformation relationship curve

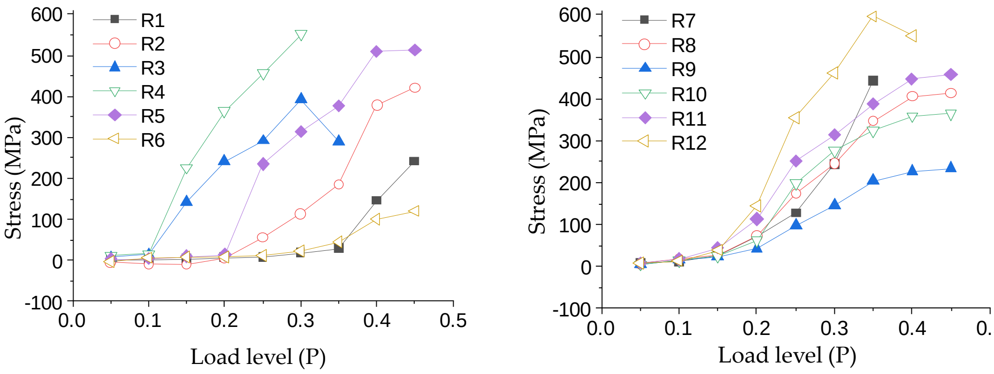


a) End tower wall b) Side tower wall

Figure 12 Concrete tower wall stress

5.4 Stress State of Tower Wall Reinforcement

The variation in the reinforcement stress in the tower wall of the segmental model with increasing load level is illustrated in Figure 13. As shown in Figure 13 a), the reinforcement in the end tower wall is subjected to tension, with the stress decreasing from the centerline toward both sides. At 0.25P, the average reinforcement stress along the centerline of the end tower wall is 374.34 MPa. As shown in Figure 13 b), the reinforcement in the side tower wall is under tension, and the stress increases with the load level. At 0.25P, the average reinforcement stress along the centerline of the side tower wall is 146.21 MPa; this value increases to 300.57 MPa at 0.45P. The test results indicate that as the horizontal cable force load increases, tensile cracking of the tower wall concrete leads to a gradual transfer of load to the reinforcement.



a) End tower wall b) Side tower wall

Figure 13 Tower wall reinforcement stress

6 Conclusions

Based on the design scheme of the cable tower anchorage zone for long-span cable-stayed bridges, an analysis of the horizontal force mechanism of the anchorage zone and a segmental model loading test were conducted. The following conclusions are drawn from this study.

(1) Based on the plane frame model analysis, under the action of the horizontal cable force, the bending moment values at the outer face of the end tower wall and inner face of the side tower wall are relatively large, constituting critical sections where the concrete is susceptible to tensile cracking. The chamfer between the side and end tower walls, subjected to the combined action of tension, shear, and bending, becomes a weak section. The end tower wall bulges outward, while the side tower wall deforms inward, with the maximum deformation occurring at the axis of symmetry.

(2) The failure mode of the cable tower anchorage zone under the horizontal cable force is primarily characterized by gradual cracking of the concrete in the end and side tower walls with increasing load level. Cracks appear in the end tower wall at 0.15P and penetrate the full segment height at 0.25P, while full-depth vertical cracks appear in the side tower wall at 0.20P.

(3) The horizontal load of the segmental model of the cable tower anchorage zone increases nonlinearly with the deformation of the tower wall, and the ultimate bearing capacity is 329.70 kN at approximately 0.48P. The concrete tower wall alone cannot sustain the enormous horizontal cable force; therefore, it is necessary either to provide a considerable number of circumferential prestressing tendons or to adopt a composite anchorage scheme after verifying extreme load cases such as cable breakage or cable replacement.

(4) The concrete of the end tower wall is more sensitive to the horizontal cable force load than that of the side tower wall, and appropriate measures should be taken to enhance its crack resistance performance. As the horizontal cable force load increases, it is gradually transferred from the concrete to the reinforcement because of tensile cracking of the tower wall concrete.

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References

1. Huang W.; Pei M.; Liu X.; Wei Y. Design and construction of super-long span bridges in China: Review and future perspectives. *Frontiers of Structural and Civil Engineering* **2020**, 14, 803–838. https://doi.org/10.1007/s11709-020-0600-5
2. Zhang X.; Liu Y. Cable-Tower Composite Anchorage; China Communications Press: Beijing, China, 2010.
3. Chen, K. Mechanical mechanism of the steel anchor housing in the anchorage zones of the cable pylons. *China Railway Science* **2008**, 29, 58–64. (in Chinese)
4. Zheng S.; Liu Y.; Xu H. Structural analysis of steel bracket-concrete walls in cable-tower composite anchorage. *Engineering Mechanics* **2014**, 31, 197–202. (in Chinese)
5. Liu Y.; Chen C.; Zheng S. Analysis of force mechanism of pylon anchorage structure of fixed-end steel anchor box type. *Bridge Construction* **2015**, 45, 33–38. (in Chinese)
6. Zhao J.; Chang Y.; Zhang J. Study of design of steel anchor beams in cable-pylon anchorage zone of Qingshan Changjiang River Highway Bridge in Wuhan. *Bridge Construction* **2020**, 50(S1), 82–88. (in Chinese)
7. Zhang M.; Tang S.; Liu M. Parametric sensitivity analysis for composite cable-pylon anchor structure with PBL connection. *Bridge Construction* **2019**, 49, 72–77. (in Chinese)
8. Zhang Q.; Li M. Segmental model test study of steel and concrete composite anchor zone on pylon of cable-stayed bridge. *Bridge Construction* **2006**, 16–19. (in Chinese)
9. Xu H.; Liu Y.; Li Y.; Zheng S. Experimental study on stress mechanism of exposed steel anchor box cable-tower

anchorage. *Journal of Tongji University (Natural Science)* **2014**, 42, 672–676. (in Chinese)

10. Zheng S.; Fu Q.; Wang D.; Li H. Horizontal load transfer mechanism of composite cable tower anchorage zone with exposed steel anchor box. *Engineering Mechanics* **2024**, 41, 101–110. (in Chinese) AUTHOR BIOGRAPHIES