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Flexural bearing capacity and stiffness research on CFRP sheet strengthened existing reinforced concrete poles with corroded connectors

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Abstract. In mountainous areas of China, concrete poles with connectors are widely employed in power transmission due to its convenience of manufacture and transportation. The bearing capacity of the poles must have degenerated over time, and most of the steel connectors have been corroded. Carbon fiber reinforced polymer (CFRP) offers a durable, light-weight alternative in strengthening those poles that have served for many years. In this paper, the bearing capacity and failure mechanism of CFRP sheet strengthened existing reinforced concrete poles with corrosion steel connectors were investigated. Four poles were selected to conduct flexural capacity test. Two poles were strengthened by single-layer longitudinal CFRP sheet, one pole was strengthened by double-layer longitudinal CFRP sheets and the last specimen was not strengthened. Results indicate that the failure is mainly bond failure between concrete and the external CFRP sheet, and the specimens fail in a brittle pattern. The cross-sectional strains of specimens approximately follow the plane section assumption in the early stage of loading, but the strain in the tensile zone no longer conforms to this assumption when the load approaches the failure load. Also, bearing capacity and stiffness of the strengthened specimens are much larger than those without CFRP sheet. The bearing capacity, initial stiffness and elastic-plastic stiffness of specimen strengthened by double-layer CFRP are larger than those strengthened by single-layer CFRP. Weighting the cost-effective effect, it is more economical and reasonable to strengthen with single-layer CFRP sheet. The results can provide a reference to the same type of poles for strengthening design.

Keywords: bearing capacity; carbon fibre reinforced polymer (CFRP) sheet; corroded connector; reinforced concrete pole; stiffness

1. Introduction

The southwest power transmission line built as a hybrid line of iron towers and concrete poles in 1959 is the first 220 kV voltage level transmission line in China. In 1982, it was found that

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concrete and steel in some joints were badly corroded, which threatens the safe operation of the line. The inspection results at that time showed that the steel near the joint of the pole was corroded in varying degrees: In the serious corrosion part, the concrete cover cracked and even spalled, and the main reinforcement suffered from serious corrosion and peeled off. In others, the concrete spalled but the steel didn't expose. After that, the line operated for more than 20 years, and kept in service far longer than original plan and also has far exceeded the load reference period at the time of construction. In terms of the situation of southwest power transmission line in Guangxi, on the one hand, most of the poles are located in remote mountainous areas, so replacing them is bound to increase the cost of manpower and material resources because of the complex geographical environment and inconvenient traffic. On the other hand, the reinforced concrete poles which are replaced result in creating large amount of waste concrete, and large amount of energy consumption will be generated for their subsequent disposal. To continue using the line, it is necessary to study its safety.

Affected by the erection environment, production, and transportation of electric poles, the whole member is often made in multiple sections and connected to each other by steel hoop connector, which is welded on site to satisfy the height required. Due to the long service life and environmental erosion, the mechanical properties of poles must be degraded to varying degrees, especially the steel hoop connector whose rust degree will determine the service life of the whole pole. Relevant studies have proved that the bearing capacity of this type of old ordinary reinforced concrete poles still have a high safety surplus factor (Xia *et al*. 2012a, b), and the damage of pole is mainly caused by concrete, which means the steel hoop connector still has high strength (Liu and Xia 2012). In order to make full use of this kind of pole, it can be considered to strengthen and then continue employing the poles.

The concrete pole is a component widely used in power transmission lines and its working performance directly affects power transmission. Therefore, it is crucial to study the force mechanism of the pole. To this end, scholars had conducted the mechanical behavior of poles in laboratory experiments and numerical simulations. Vivek *et al*. (2017) conducted loading and unloading tests on tapered concrete poles embedded in various footing. They found that the loading capacity of the pole is improved by about 54% by increasing the footing dimension by only 20%. Braik *et al*. (2019) study the behavior of poles when subjected to tornadoes. They proposed a framework to perform fragility analysis on utility poles subjected to tornado hazard and strength deterioration. Saboori and Khalili (2011) carried out the linear static analysis of circular thin-walled conical FRP transmission pole by ANSYS. They found that more fibre volume fraction and wall thickness of the FRP pole, caused smaller maximum deflection and stress of pole. Zeynalian and Khorasgani (2018) carried out a numerical study on a 7-span distribution line to investigate the effects of harsh weather conditions on it and found out the weakest structural part of the network. Baghmisheh and Mahsuli (2021) developed finite element models of the H-type reinforced concrete poles and investigated the damage and collapse pattern of poles under earthquake excitations. The results showed that the most vulnerable segment of the pole is the first 0.5 m. Kliukas *et al*. (2018) conducted tests on deteriorated concrete poles that were under service for more than 30 years. They revealed that collapse was mainly caused by the failure of longitudinal reinforcement. In some cases, the collapse was associated with crushing of the concrete in compression. However, there are few studies on the mechanical properties and reinforcement methods of the old concrete poles with longer service time. Therefore, this study can only refer to strengthening methods of other concrete members to select an economic and effective strengthening method of concrete poles, and verify the effectiveness of the selected method.

There is a lot of research on the reinforcement of concrete components and the common reinforcement methods include prestressed reinforcement method, steel-enveloped strengthening method, bonding steel plates method, fiber reinforced polymer (FRP) reinforcement and so on (Terrasi *et al*. 2014, Shan *et al*. 2020, Abu-Obeidah *et al*. 2019, Zhu *et al*. 2014, Sumathi 2017, Mohammed and Fattah 2020, Li *et al*. 2018, Roberts *et al*. 2012). The durability and corrosion resistance of FRP can ensure the long-term operation of components, so FRP is widely used in strengthening components and structures.

Many scholars have carried out research on strengthening concrete with FRP. Some scholars adopted the near surface mounted (NSM) FRP method to strengthen beams and pointed out that the method had obvious strengthening effect. Rahal and Rumaih (2010) found that beams strengthened in shear using NSM CFRP bars increased the shear capacity by 37% to 92%, reduced the width of the diagonal cracks and rendered the beams to develop significant flexural ductility. The experimental results of Zhu *et al*. (2019) revealed that the flexural capacity and ductility of beams strengthened with side near surface mounted (SNSM) CFRP were significantly higher than that of control specimen. In addition, Abdel-Kareem *et al*. (2019) had conducted research on shaping with different end anchorage of glass fiber reinforced polymer (GFRP) (strips and rods) and indicated that beams strengthened with NSM GFRP rods have shown a greater improvement in the capacity than GFRP strips. However, NSM FRP methods need to slot in the reinforced part, which will cause damage to the component. It can be avoided by pasting an external FRP sheet to strengthen the specimen. Sim and Park (2005) reported that basalt fiber reinforced polymer (BFRP) sheet has better weathering performance than GFRP sheet, and the improvement effect of BFRP sheet on the ultimate strength of beam depended on the number of layers, which can be improved by up to 27%. Huang *et al*. (2013) compared the strengthening effects of BFRP, CFRP and GFRP, and the results validated that the performance of beam strengthened by CFRP sheet was better than that of other materials. Ahmad *et al*. (2018) applied CFRP sheets to strengthen concrete specimens damaged by mechanical loading and elevated temperature. The results show that CFRP wrap reinforcement can recover the lost strength and even make the concrete strength greater than the initial value.

It can be seen from the above literature that surface damage of the specimen can be avoided by pasting an external FRP sheet to strengthen the specimen, and the performance of beams reinforced with CFRP sheet is better than other types of FRP sheets. In addition, CFRP sheet reinforcement can obviously improve the performance of damaged components. Therefore, this study adopts the CFRP sheet to strengthen the poles with connector in service, and comprehensively compares the strength, stiffness and economy of the poles after adopting this strengthening method. Most of the existing reinforcement studies are based on the reinforcement of man-made pre-damaged beams or columns, but there are few reports on the reinforcement of the annular concrete pole with connector which is damaged by 50 years of service. This paper can provide a new idea and reasonable reference for solving the aging problem of concrete poles in mountain areas and strengthening design of similar concrete poles.

2. Experimental program

2.1 Test specimens

In order to reveal the bearing capacity of the CFRP sheet strengthened reinforced concrete poles with steel hoop connector, 4 typical poles were selected as the test specimens, and one of them that was not strengthened was set as the control specimen (named as J0). All specimens were 4.2 m in length, with an external diameter of 400 mm and an internal diameter of 300 mm in annular cross section. Before strengthening, the outer concrete layer of the poles and the outer rust of the steel hoop connector were worn off by an angle grinder and cleaned with acetone, and then steel hoop connectors were filled with cement mortar until they were flush with the main part of poles. The reinforcement method is as follows: First, CFRP sheets were pasted along the poles longitudinally, in which one specimen was strengthened with double-layer CFRP sheets (named as J2) and the other two specimens were strengthened with single-layer CFRP sheet (named as J1 and J1' respectively). It is worth noting that J1 and J1' are both specimens strengthened with single-layer CFRP sheet, and they have the same parameters. J1' is the original specimen reinforced with single-layer CFRP sheet. When the strain gauge quality of specimens was checked after pasting FRP, it was found that most of the strain gauges in the steel hoop connector of J1' were damaged. This means that the stresses on the connector of the specimen reinforced with single-layer CFRP sheet cannot be accurately obtained by J1'. In order to ensure that the test analysis can be carried out, a specimen with the same parameters was added before loading, and the specimen was named J1. In the subsequent analysis of specimens reinforced with single-layer CFRP sheet, the strain data were obtained from J1, and the average value of the performance indices of J1 and J1' is taken for the rest analysis.

Then the circumferential CFRP was pasted on outer surface of longitudinal CFRP sheet to act as a hoop. The width of the CFRP hoop was 50 cm, and the net distance between the two CFRP hoops was 20 cm. The sticking sequence of the CFRP hoop was from both sides of the steel hoop connector to two ends of the specimen with a net distance of 20 cm.

2.2 Materials

The test poles are rods with corroded steel hoops at the midspan joints, which come from a road section of Southwest line and have been in service for 50 years. The average concrete compressive strength of specimens measured by the rebound method is 22.8 MPa. The average values of measured yield strength, ultimate strength and elongation of longitudinal reinforcement that is made of HPB235 grade reinforcement with a diameter of 8 mm are 401 MPa, 475 MPa and 27.5%, respectively. The steel hoop with a thickness of up to 8 mm and a length of 20 cm is welded together.

The reinforced materials are CJ300 CFRP sheet and YZJ-CQ fiber composite impregnated adhesive produced by Wuhan Changjiang Reinforcement Technology Co. LTD. The tensile strength and elastic modulus of CJ300 CFRP sheets are 3035.2 MPa and 2.3×10^5MPa, and elongation is 1.5%. The tensile strength, compressive strength, flexural strength and tensile bond strength with concrete of YZJ-CQ fiber composite impregnated adhesive are 40.6 MPa, 92.4 MPa, 74.3 MPa and 4.0 MPa, respectively.

2.3 Load equipment and test method

The specimens were all loaded at three dividing points through the distribution beam and hydraulic jack. The load value and strain of each specimen were collected automatically by Japan TML system and DTS602 data acquisition system. Since the specimens were concrete poles with annular cross section, two special semi-circular supports were installed on the contact surface

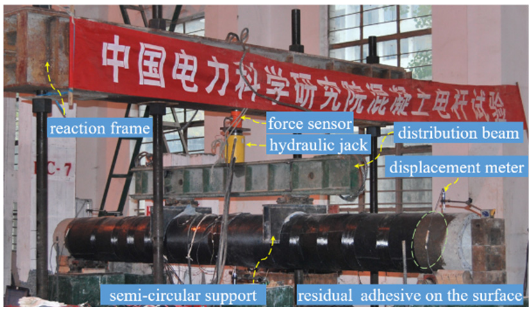
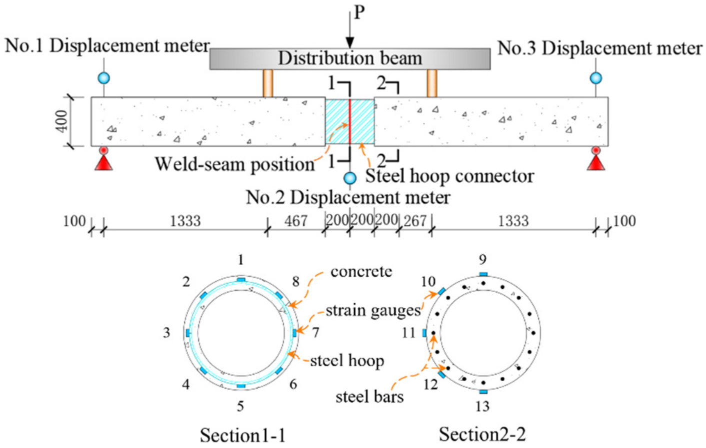
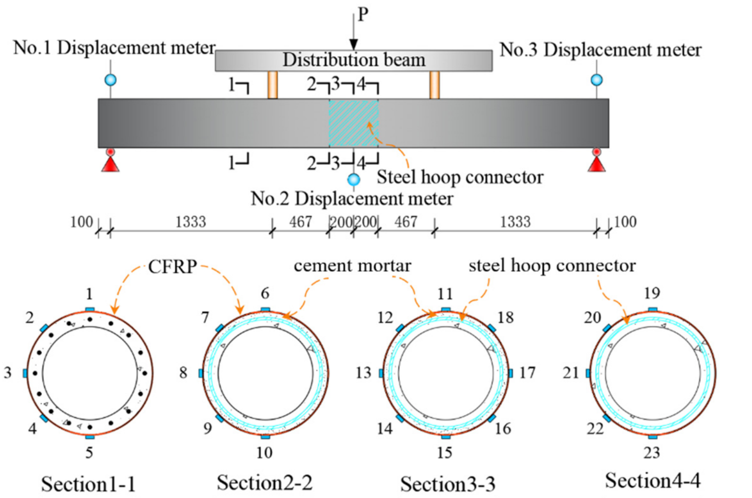


Fig. 1 Photo of test setup



(a) Schematic diagram of measuring point in J0 (mm)



(b) Schematic diagram of measuring points in reinforced specimens (mm)

Fig. 2 Schematic diagram of measuring point between distribution beam ends and the pole, and two roller supports under specimen ends were also tailor-made for ensuring the free rotation of the ends. In order to acquire the significant data such as deformation, internal force distribution, crack development and strain distribution, electronic displacement sensors were placed at the supports and midspan, and resistance strain gauges were arranged in the corresponding positions of the steel hoop and main part of pole. The loading device is given in Fig. 1, and the arrangements of displacement meters and strain gauges are presented in Fig. 2.

The load was applied in a load increment control method at a stage loading of 9.8 kN and the specimens were pre-loaded 2 times before loading. To ensure that the specimen had been fully deformed, after each load was finished, the load was maintained for 5 to 10 minutes before real-time data collection. When the load was close to the cracking load, each stage loading became 4.9 kN until the load could not be increased.

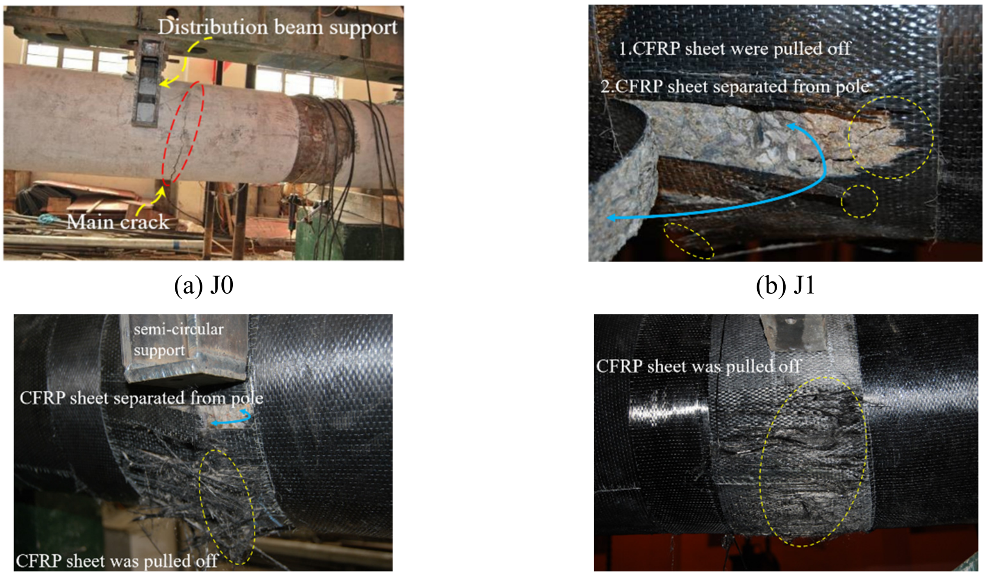
3. Test results and discussion

3.1 Test observation

The failure mode of each pole is given in Fig. 3, and the main failure phenomena are described as follows:

1) Specimen J0: at the initial stage of loading, there was a linear relationship between the increase of load and the change of displacement, and the specimen had no circular crack. As the load rose to a certain level, visible circular cracks began to appear in the concrete at the bottom of the pure bending section, and the crack width was small at this time. Thereafter, with the increase of the load, the circular cracks increased and widened, and began to develop towards both ends of the specimen. After the concrete cracked, the tensile stress was mainly borne by the steel bars. When the load increased to a certain value, a main crack was formed between 10 and 15 mm near the distribution beam support, with a width up to 0.4mm. At this time, the specimen can still be loaded. The width and depth of the main crack also increased with the load increasing. After that, the midspan deflection of the specimen increased rapidly. When the load could not be increased, the loading ended.

2) Specimen J1 and J1': at the initial stage of loading, as the specimen was in the elastic state, the increase of load had a linear relationship with the deformation, and no CFRP sheet cracking occurred at any part. As the load increased to a certain level (J1 was 98 kN, J1' could not be characterized because some strain gauges were damaged), the steel hoop in the tensile zone of the midspan began to yield. When the load continued increasing to a certain value (J1 was 137.2 kN, J1' failed to obtain this characteristic value), the upper steel plate of the steel hoop in the compression zone also reached yield. Thereafter, when the loads on the two specimens were increased to 186.2 kN and 196 kN, respectively, some sounds of the CFRP sheet separating from the concrete were heard near the distribution beam support, but no CFRP sheet fracture was observed. And the load continued increasing, the tearing sound was heard at each additional level of load. When the loads on the two specimens were increased to 245 kN and 272.4 kN respectively, a loud sound was heard at the bottom of the distribution beam support, and the longitudinal CFRP sheet between two CFRP hoops was pulled off, exposing the broken carbon fiber and the concrete powder. The deflection of specimens increased dramatically and the load could not be increased, which means the test was over.



(a) J0 (b) J1

(c) J1' (d) J2

Fig. 3 Failure mode of each specimen with connector

3) Specimen J2: the destruction process of the specimen with double-layer CFRP sheets was different from the two specimens with single-layer CFRP sheet described previously. It was only when the load rose to 205.8 kN that tearing sound of the CFRP sheet separating from the concrete was heard near the distribution beam support. Later, the upper steel plate of the steel hoop in the compression zone reached yield load of 225.4 kN, while the lower steel plate of the steel hoop in the tensile zone did not yield. As the load increased to 313.6 kN, a loud noise was heard underneath the distribution beam support and the test came to an end.

It can be observed that J0 fails in a ductile pattern, but the specimens strengthened with CFRP sheet fail in brittle pattern with loud noise. Because CFRP is a brittle material, and its addition increases the brittleness of the specimen.

3.2 Failure characteristics

It can be seen from the whole process and final failure modes that the destruction process of specimens is as follows: i) firstly, a part of CFRP sheets in the tensile zone separated from the surface of the concrete; ii) then, the longitudinal CFRP sheets between the CFRP hoops fractured, the specimen damaged finally. At the end of the test, when the breaking CFRP sheet was removed, the concrete powder emerged, which was caused by the surface of the concrete being pulled away by the structural glue used to bond the CFRP. Although the lower and upper parts of the steel hoop of J1 and J1' at midspan yielded successively, and only the upper part of the steel hoop yielded in J2, the damage occurred at the supports of the distribution beam instead of the position of the steel hoop.

For the unreinforced J0, the failure was mainly triggered by the destruction of main part of pole. It's worth noting that the steel hoop for connection did not appear buckling phenomenon and even the steel in joint had not yielded yet.

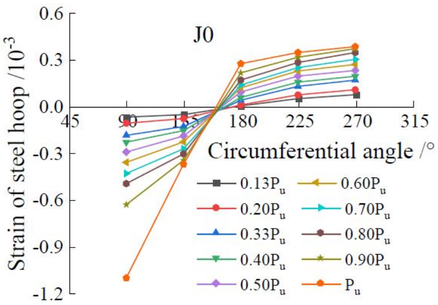
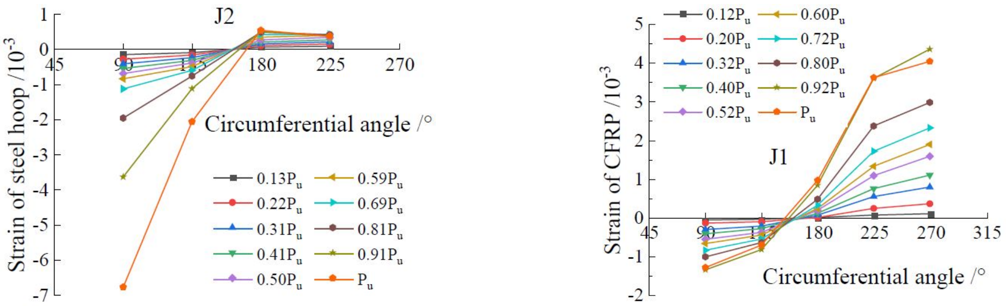


Fig. 4 Strain distribution of steel hoop in J0



(a) Circumferential strain of steel hoop in J2 (b) Circumferential strain of CFRP in J1

Fig. 5 Strain distribution of steel hoop and CFRP for parts of specimens

Table 1 Linear fitting degree R^2 of strain under each level of loading

Specimen	Level of loading									
	$0.1P_u$	$0.2P_u$	$0.3P_u$	$0.4P_u$	$0.5P_u$	$0.6P_u$	$0.7P_u$	$0.8P_u$	$0.9P_u$	P_u
J0	0.975	0.974	0.957	0.945	0.927	0.918	0.914	0.909	0.888	0.825
J1	0.97	0.943	0.957	0.961	0.961	0.961	0.96	0.954	0.951	0.946
J2	0.93	0.933	0.932	0.925	0.925	0.91	0.897	0.876	0.838	0.829

3.3 Cross-sectional strain analysis

Fig. 4 shows the strain distribution characteristics of the steel hoop for J0, and Fig. 5 draws the strain distribution of the steel hoop and the corresponding section 4-4 of CFRP sheet for the typical strengthened specimens. And the linear correlation coefficients of strains under each level of loading are shown in Table 1.

As can be seen from Table 1, the linear fitting degree of J0 is greater than 0.90 and the strain of the steel hoop approximately follows the plane section assumption before the load reaches $0.8P_u$ (P_u stands for ultimate load). However, when the load approaches the failure load, the strain in the tensile zone of the bottom steel hoop shows a certain degree of nonlinear property and no longer conforms to the plane section assumption. There may be two reasons. In the first place, the main crack between the distribution beam support and the steel hoop develops gradually, resulting in the tensile stress on the tensile side of the pole is not fully transmitted to the joint hoop. Secondly, the steel hoop has been corroded in the service, and the rust between the inner wall of the concrete pole and the steel hoop weakens the force of friction, the steel hoop and concrete in the tensile zone slip relatively when the load approaches the failure load and dissipated some energy, thus reducing the load transferred to the joint hoop. The linear fitting degree of J2 is greater than 0.90 before the load reaches $0.8P_u$, the strain of the steel hoop approximately conforms to the plane section assumption. The steel hoop of J2 is wrapped with double-layer longitudinal CFRP, which is equivalent to increasing the reinforcement ratio in the tensile zone. The CFRP plays a large tensile role, which relatively weakens the tensile properties of the steel hoop wrapped inside. Therefore, the strain in the tensile zone of J2 appears nonlinear in the later stage of loading. Fig. 5 (b) shows the strain of CFRP in J1. The CFRP and concrete pole can work together during the whole loading. Therefore, the strain of J1 along the section height is more in line with the plane section assumption. It can be seen from Table 1 that the linear fitting degree of J1 is greater than 0.90 during the whole loading, but in the later stage of loading, the strain in the tensile area at the bottom of the specimen tends to decrease. This may be due to the slow development of concrete cracks at the edge of the tensile zone restrained by CFRP, which restricts each other, resulting in the reduction of the strain of CFRP.

3.4 Load-midspan deflection response

Fig. 6 offers the load-midspan deflection curves of three strengthened specimens and one specimen without strengthening. As drawn in the figure, although the three specimens adopt different reinforcement schemes, before the loads reach 58.8 kN, the curves basically coincide with each other and change linearly. In this stage, as the same load is applied, the midspan deflection of strengthened specimens is reduced by 80.93%~91.9% compared with that of control specimen. When the load is greater than 58.8 kN, the slope of each curve starts to decrease, and the curves of the three strengthened specimens gradually separate, showing their own nonlinear properties. It indicates that when the load is small, the constraint effect of CFRP sheet on the specimen is small, and the influences of CFRP amount on the stiffness is not obvious. However, some CFRP sheets debond or fracture with the load increasing, so the difference of the improvement effect of the CFRP amount on the stiffness increases gradually.

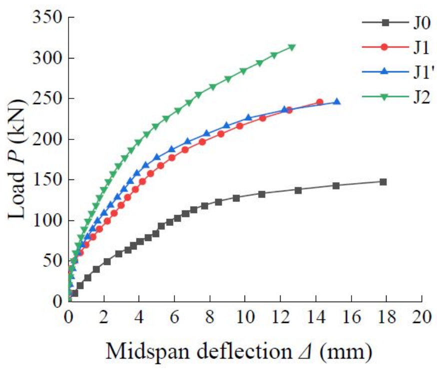


Fig. 6 Load-midspan deflection curves

When the load is between 68.6 kN and 147 kN, the deflection of the specimens strengthened with single layer and double-layer of CFRP sheets decreases by 64.94%~78.29% and 79.53%~87.16% respectively compared with that of the specimen not strengthened. As the specimens are damaged, the deflection of the specimens strengthened with one and two layers of CFRP sheets decreases by 17.05% and 28.71% respectively compared with that of the specimen not strengthened.

3.5 Ultimate bearing capacity

From the failure mode of J0, it can be seen that the failure of J0 is mainly caused by the main part of pole, rather than the steel hoop connector, and its bearing capacity is controlled by the main part of pole. Therefore, the flexural capacity can be calculated according to the following formulas of from China power industry standard (DL/T5154-2012).

$$P = \frac{2M}{l} \tag{1}$$

$$M = f_c A(r_1 + r_2)\frac{\sin\pi\,\alpha}{2\pi} + f_s A_s r_s \frac{\sin\pi\,\alpha + \sin\pi\,\alpha_t}{\pi} \tag{2}$$

$$\alpha = \frac{f_s A_s}{\alpha_1 f_c A + 2.5 f_s A_s} \tag{3}$$

$$\alpha_t = 1 - 1.5\alpha \tag{4}$$

Where P is the concentrated force; M is the design value of flexural moment; r_1, and r_2 are the internal and outer radius of the pole; r_s is the radius of the circle where the longitudinal ordinary steel bar is located, mm; f_c and f_s are the design values of prismatic compressive strength of concrete and tensile strength of steel bar; When the strength of concrete is less than 50 MPa, $\alpha_1 = 1$; α is the ratio of concrete cross-sectional area in compression zone to total concrete area; α_t is the ratio of tensile longitudinal reinforcement cross-sectional area to total longitudinal reinforcement area, when $\alpha > 2/3$, $\alpha_t = 0$; A and A_s are concrete area and longitudinal reinforcement area.

To objectively evaluate the damage degree and reinforcement effect, the bearing capacity of the pole before and after damage was calculated based on the above formulas, and they were named P_a' and P_b' respectively. After calculation, P_a' and P_b' are 221.5 kN and 154.52 kN respectively. It should be noted that P_b' is calculated based on the measured performance indices of the longitudinal reinforcement and concrete in the damaged specimens. The error between the calculated value and test value of bearing capacity of J0 is only 5.12%, indicating that the above formulas can reflect the bearing capacity of damaged poles. Fig. 7 is the comparison between the test value of ultimate bearing capacity of each specimen and the calculated bearing capacity of undamaged pole (P_a'). In the figure, the ultimate bearing capacity of strengthening with single-layer CFRP sheet is the average bearing capacity of J1 and J1'.

As can be seen from the figure, the bearing capacity of J0 is 33.6% smaller than P_a', which means that the bearing capacity of this type of pole decreases seriously. after more than 50 years of service. This is mainly caused by carbonization of concrete and corrosion of steel bars. After measurement, it is found that the carbonation depth of concrete reaches 10 mm~20 mm, and the

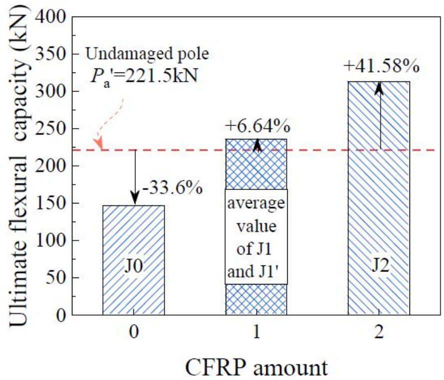


Fig. 7 Comparison of ultimate bearing capacity

corrosion rate of steel section is 7.1%. In addition, the ultimate bearing capacity of specimens reinforced with CFRP sheets is larger than that undamaged. The ultimate bearing capacity of specimen strengthened with single-layer CFRP sheet is similar to that undamaged. However, the ultimate bearing capacity of specimen strengthened with double-layer CFRP is 41.58% higher than that undamaged.

3.6 Stiffness

By acquiring the first derivative of the load-midspan deflection curve, the variations of stiffness at each stage of loading can be obtained. The differential function of Origin software is applied to gain the relevant stiffness of each point, and the degradation law curve of the stiffness of each specimen with loading is drawn in Fig. 8.

It can be observed that the initial elastic stiffness of the strengthened specimens is significantly increased compared with reference specimen. For example, the elastic stiffness of J1, J1’, and J2 is about 3 times, 3 times and 4 times bigger than that of J0, respectively. Meanwhile, the elastic-plastic stiffness of each strengthened specimen is larger than that of J0. The reasons may be as follow: Firstly, there is damage in the specimens because they have been kept in service far longer

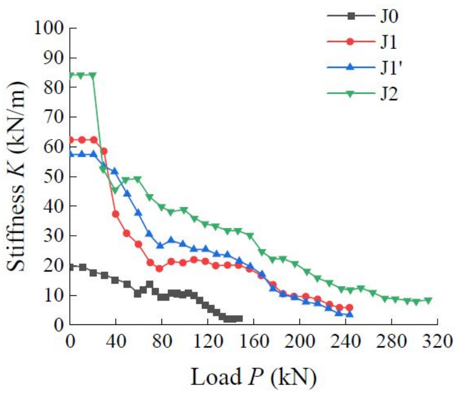


Fig. 8 Stiffness degeneration curves

than originally intended. Therefore, the cracks of J0 develop rapidly, showing the typical failure of insufficiently-reinforced beam, and its stiffness decreases. And secondly, CFRP has good tensile strength and can effectively restrain the crack development in the tensile zone when used to strengthen the damaged specimens. Moreover, the elastic modulus of CFRP is greater than that of reinforced concrete so the flexural stiffness of strengthened specimen increases to a certain extent. The initial elastic stiffness of the specimen with double-layer longitudinal CFRP sheets is much greater than those of only with single-layer longitudinal CFRP sheet, and their ratio is about 1.4. In addition, the initial stiffness of J1 and J1' is relatively close to each other. The elastic-plastic stiffness of the specimen strengthened by double-layer longitudinal CFRP sheets is slightly larger than those of the single-layer one. It is indicated that reasonable reinforcement scheme can improve the deformation resistance of the old concrete pole with connector and maintain the safe operation of the electrical circuit.

3.7 Cost-effective effect

It is found that the ultimate bearing capacity, initial stiffness and midspan ultimate deflection of the specimen strengthened with double-layer CFRP sheets are increased by 32.77%, 40.89% and -14.05%, respectively, compared with that strengthened with single-layer CFRP sheet, but the cost increased by 58.33% (The cost of reinforcement with single-layer and double-layer CFRP sheets is approximately $135.26 and $214.17). From a comprehensive comparison, it has better economic benefit to reinforce this kind of poles with single-layer CFRP sheet.

4. Conclusions

Through the flexural capacity test of the concrete pole with corroded steel hoop connector, the failure mechanism and mechanical properties of the existing reinforced concrete poles strengthened with CFRP sheet are revealed. The following conclusions can be drawn:

- The failure mode of the specimens strengthened with CFRP sheet is brittle failure, and the failure mainly is the debonding failure between concrete and CFRP sheet, with longitudinal CFRP sheet being pulled off as the symbol. Moreover, the cross-sectional strain approximately accords with the plane section assumption in the early stage of loading.
- Comparing the bearing capacity and stiffness of reinforced and unreinforced specimens, the ultimate bearing capacity and stiffness of the pole with a steel hoop connector can be greatly improved by using the reinforcement method of CFRP sheet.
- After more than 50 years of service, the ultimate bearing capacity of the pole with a steel hoop connector degrades 33.6%. Compared with the undamaged specimen, the bearing capacity of the specimens strengthened with single-layer and double-layer CFRP sheets increased by 6.64% and 41.58%, respectively.
- The test results of the one with double-layer CFRP sheets are higher than those with single-layer CFRP sheet in both initial elastic stiffness and elastic-plastic stiffness.
- The law of stiffness degradation can provide an effective reference for the reinforcing design of in-service concrete poles and provide a necessary foundation for further study of the long-term stiffness of such members.
- Overall comparison, the old concrete poles strengthened with single-layer CFRP sheet can better satisfy strength and stiffness requirements, and has a certain economy.

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