Flexural behaviour of RC beams strengthened with prestressed steel wire ropes polymer mortar composite

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ABSTRACT
This paper presents an experimental study on the flexural behavior of reinforced concrete (RC) beams strengthened with prestressed steel wire ropes-polymer mortar composite (PSWR-PM). The effects of reinforcement ratio of steel wire ropes, strengthening procedures, and unbonded length of the steel wire rope-to-polymer mortar interface were studied with large-scale bending tests on RC beams strengthened by PSWR-PM. The test results showed that the proposed strengthening technique could substantially increase the load carrying capacity of the RC beams. Both strengthening procedures were effective in enhancing the flexural behaviour of the RC beams, and strengthening procedure II could even restrain the debonding failure. The unbonded length of the steel wire rope-to-polymer mortar interface had a minor impact on the ultimate capacity when the anchoring length of the steel wire rope in the polymer mortar was sufficient. Moreover, a theoretical model was proposed to predict the flexural strength of RC beams strengthened with the PSWR-PM technique and the predictions showed good agreement with the experimental results.

1. Introduction
A growing interest in the restoration and enhancement of reinforced concrete (RC) structures has led to the development of innovative materials and strengthening techniques. The prestressed steel wire rope-polymer mortar strengthening (P-SWR) technique is a technique because of good economy, durability and fire resistance and minimal increase in the size of the repaired members (Wu et al., 2010; Liao et al., 2017; Dai et al., 2018; Zeng, Chen, and Li, 2020). In this strengthening technique, the steel wire ropes are firstly anchored and prestressed to the tension side of RC members, and then covered by polymer mortar to achieve required durability and fire resistance. Steel wire rope or mesh has been widely adopted as a strengthening material due to its lower cost and desirable properties of bonding and high flexibility (Miao et al., 2020). The polymer mortar application changes the load transfer mechanism and reduces the forces of the steel wire ropes on the anchors. Moreover, the polymer mortar also provides desirable protection for the prestressed steel wire ropes against the adverse environmental condition. The dispersed anchors used for steel wire ropes have become another advantage of convenient prestressing and anchoring for steel wire ropes (Yang et al., 2012).

Previous investigations have been conducted to study the flexural behaviour of RC beams strengthened with prestressed steel wire ropes. Kim et al. (2007) and Yang, Byun, and Ashour (2012) adopted wire rope units to enhance the shear strength of RC beams. It was observed that the shear strength of the strengthened beams was significantly increased, and the higher the initial prestressing force in wire rope units, the higher the shear strength obtained. The tests by Wu et al. (2010), (2013), Qestha, Shafigh, and Jumaat (2015) and Huang, Guo, and Yao et al. (2019) indicated that the prestressed steel wire rope strengthening method could effectively improve the flexural behaviour of the strengthened beams. Zhang and Sun (2018) adopted polyurethane cement (PUC) with good fluidity to replace polymer mortar (PM). Although the RC beams strengthened with PSWR-PUC showed an increased strength, undesired debonding failure occurred for the strengthened beams, Li, Wu, and Bian (2018) and Zeng, Chen, and Li et al. (2020) performed an experiment and comparative evaluation on the flexural behaviour of RC beams strengthened with different strengthening methods. The results indicated that the flexural behaviour of the combinedly strengthened RC beam fell between those of RC beams strengthened with the corresponding individual ones.

However, one of the most critical disadvantage that compromised the appeal of the P-SWR technique is that premature debonding dominates the failure of strengthened RC beams, especially for ones strengthened with several layers of steel wire mesh. There are mainly...
three debonding failure modes, namely, plate end interfacial debonding, intermediate crack debonding and cover separation, as shown in Figure 1. The interfacial bonding strength is the critical factor that affects debonding failure modes, and the poor material strength and poor compaction of the polymer mortar will decrease the interfacial bonding strength. Especially for a strengthened RC bridge girder that uses a large-amount layers of steel wire meshes, the inexperienced strengthening procedure could not guarantee the proper compaction of the polymer mortar, and may even cause debonding failure. Consequently, it is of crucial necessity to restrain debonding failure, and no research has been reported on the influence of the strengthening procedure on the interfacial bond strength. Meanwhile, other disadvantages of the P-SWR technique will not be neglected since (i) a high anchor force in a large amount of steel wire meshes discourages secure anchoring, and even causes serious deformation to the anchor component (Huang, Guo, and Yao et al. 2019); and (iii) Plastering mortar led to vibration to the prestressed steel wire rope, which caused debonding on the originally well-bonded interface, and this reduced the interfacial bond strength.

This paper presented a novel technique for strengthening RC beams with prestressed steel wire ropes and polymer mortar (referred as “PSWR-PM” for brevity, as shown in Figure 2). The PSWR-PM strengthening method comprises the prestressed steel wire ropes, steel angle anchors, and polymer mortar. Steel wire ropes with larger diameter were adopted to replace steel wire mesh, to meet the high demand for reinforcement and secure anchoring. The steel angle anchors with three holes for fastening on the eye bolts are installed at the tensile surface of the strengthened RC beam. The steel angle anchors could either be placed at both ends of the beam only or dispersed along the length of the RC beam. The eye bolts connected with the steel wire rope pass through the holes in the steel angle anchors and are fastened to the steel angle anchors with nuts. The steel wire ropes can be prestressed by tightening the nuts, which is convenient to be done at the construction site. And then the prestressed steel wire ropes were covered with polymer mortar to ensure the compatibility of the steel wire ropes and the unstrengthen RC beams. Furthermore, efforts have been made to optimize the strengthening procedures suppressing debonding failure.

In this paper, the relationship between the applied torque and tensile force exerted in the bolt was addressed experimentally fistly. Afterwards, strengthening effects of PSWR-PM on the flexural behavior of RC beams were studied, where impacts of the reinforcement ratio of the steel wire ropes (0% or 0.94%), two different strengthening procedures (Method I and II), and the various unbonded lengths of the steel wire rope-to-mortar interface were investigated. The flexural behaviour of the strengthened specimens was evaluated in terms of the failure modes, load–deflection curves, ductility and deformability, and strain distribution along the beam length. Finally, a theoretical model was proposed to predict the

![Figure 1](image1.png)  
**Figure 1.** Debonding failure modes of RC beams strengthened with P-SWR.

![Figure 2](image2.png)  
**Figure 2.** Strengthening details of the PSWR-PM strengthening system (unit: mm).
flexural strengths of RC beams strengthened with the PSWR-PM system.

2. Experimental program

2.1. Material properties

The steel wire rope with an 8 mm nominal diameter consisted of six unidirectional twisted steel strands and one steel core. The nominal cross-sectional areas of the steel wire rope and each wire were 25.5 mm² and 0.22 mm², respectively.

Tensile tests were conducted on the mechanical properties of steel wire ropes (in Figure 3), and the average tensile strength and ultimate strain were 1654 MPa and 1.06%, respectively. Commercial polymer mortar and adhesive were used. The normal bond strength and shear bond strength between adhesive and concrete were 4.4 MPa and 3.52 MPa. The average cubic compressive strength of the concrete and the polymer mortar after curing for 28 days was 57.8 MPa and 55.6 MPa, respectively. The other materials employed in this study are summarized in Table 1.

2.2. Torque coefficient for prestressed steel wire rope

Since the steel wire rope is prestressed by tightening the nuts, the relationship between the applied torque (T) and tensile force (Q) exerted in the bolt is crucial for calculating the prestressed tensile force of the steel wire ropes. The T–Q relationship can be addressed as (Kim et al. 2007):

\[ T = k \cdot Q \cdot d \] (1)

where \( d \) is the bolt diameter; and \( k \) is a coefficient, a torque coefficient.

If the coefficients \( d \) and \( k \) are assumed to be constant for the same bolts and nuts, Eq. (1) can be rewritten in Eq. (2):

\[ Q = \eta \cdot T \] (2)

where \( \eta \) is a coefficient with a constant value.

To obtain the values of \( \eta \) and \( k \), as well as to monitor the prestress loss of the steel wire ropes during and after the prestressing process, nine specimens were tested, as shown in Figure 4. All the specimens were

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Figure 3. Sample of the steel wire rope: (a) global view and cross section; (b) test sample; (c) material test; and (d) stress–strain curve.

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Table 1. Summary of the mechanical properties of the materials.

| Material          | Diameter (mm) | Yielding strength (MPa) | Ultimate strength (MPa) | Yield strain \((\times 10^{-5})\) | Elastic modules (MPa) |
|-------------------|---------------|-------------------------|-------------------------|---------------------------------|-----------------------|
| HPB300            | 10            | 439                     | 584                     | 2195                            | 219,000               |
| HRB400            | 14            | 445                     | 630                     | 2385                            | 219,000               |
| Steel wire rope   | Nominal diameter (mm) | Effective cross-sectional area (mm²) | Ultimate force (kN) | Elastic modulus (MPa) |
|                   | 8             | 25.5                    | 42.2                    | 155,000                         |
composed of an 8 mm-diameter steel wire rope, a set of eye bolts and nuts, and a load cell with a dead-end anchor. Each specimen was tested three times, and a new set of eye bolts with nuts was used each time. The testing process was as follows: first, one eye bolt linking one end of the steel wire rope was connected to the end anchor via the nut (in Figure 4b). Another eye bolt linking the other end of the steel wire rope passed through the hollow load cell and was fixed to the dead-end anchor (in Figure 4c). The steel wire rope was then prestressed by tightening the nut with a torque wrench (in Figure 4b). After applying the tension on the steel wire rope, the prestress in the steel wire rope was continuously recorded in 72 h (in Figure 4d).

2.3. Specimen details

All six RC beams were designed following the strong shear-weak bending principle, with a cross-sectional area of 200 mm (width) × 400 mm (depth) and a span length of 3300 mm, as shown in Figure 5. The longitudinal steel reinforcement comprised five 14 mm-diameter steel bars, three in the tension zone, and two in the compression zone. The stirrups with a diameter of 10 mm were spaced at a centre-to-centre interval of 100 mm. The clear cover to the stirrups was 15 mm on both sides. The beams were cast uniformly and cured for 28 days in the laboratory.

A total of six RC beams, including five strengthened beams and one unstrengthened beam, were fabricated and tested. All of the beams except CB were strengthened with three steel wire ropes with a prestress level of 0.22 (the prestress levels represent the prestress tension force divided by the capacity of the steel wire rope, corresponding to 9.49 kN) for each steel wire rope. The thickness of the PSWR-PM strengthening layer was designed to be 50 mm, and the internal axial force point of the steel wire ropes was 25 mm from the tension edge of the beam, as shown in Figure 5. Steel anchor was manufactured by welding two steel angles, and details of steel angle anchors were shown.
in Figure 2. The steel angle anchors were defined geometrically according to the following points: (i) steel anchors were not yielding when the steel wire ropes were prestressed, (ii) the length and width were less than the beam width and thickness of polymer mortar, respectively. In addition, the presence of steel anchors and bolts should avoid local damages in the concrete. In fact, protections have been taken to avoid local damages, i.e., (i) the steel angle anchors were installed near the support, (ii) bolts with a length of 200 mm were installed in the concrete by epoxy adhesives, (iii) the installed bolts were far away the tensile reinforcement to avoid damaging the reinforcement.

The test parameters included the reinforcement ratio of the steel wire ropes, the strengthening procedures, and the unbonded interfacial length between the steel wire rope and polymer mortar, as listed in Table 2.

An unstrengthen RC beam named CB was tested as a control specimen, while the remaining beams were all strengthened with the proposed PSWR-PM systems. The specimens except CB were all strengthened using three of 8 mm-diameter steel wire ropes embedded in the polymer mortar (i.e., with a reinforcement ratio of 0.094%).

The strengthening procedures, which could influence the interfacial bonding strength between the steel wire rope and polymer mortar, were studied by considering two procedures (I and II). In the strengthening procedure I, the steel wire ropes were first prestressed and anchored to the beam, and then the polymer mortar was plastered to the beam and covered the steel wire ropes, as shown in Figure 6a. In strengthening procedure II, a layer of polymer mortar was applied to the concrete surface before the steel wire ropes were anchored and prestressed. The other layer of polymer mortar was then coated onto the prestressed steel wire rope, as shown in Figure 6a. Specimen SB-1 was strengthened using strengthening procedure I with three steel wire ropes, while the remaining strengthened beams were all strengthened using procedure II.

The available studies (Gustavson 2004; Li and Song 2020) highlighted that the bond strength between the concrete and the prestressed steel wire ropes was the key factor for affecting the structural response of the strengthened members. However, the inexperienced strengthening procedures could result in a poor interfacial bonding strength. A smooth film was set on the surface of each steel wire rope on both sides of the loading point, simulating the unbonded interface between the steel wire rope and polymer mortar. Its effect on the flexural behaviour of the strengthened beam was studied. The various lengths of the smooth film were set to be 550 mm, 220 mm, and 80 mm, as shown in Figure 6b. It should be noted that for the strengthened beam with the longest 550-mm smooth film in shear span, the remaining bonding length is 550 mm and longer than the effective length, according to the Chinese code (GB50010–2010, 2010).

2.4. Strengthening procedures

For specimen SB-1 adopting strengthening procedure I, the following steps were involved, as shown in Figure 7. Firstly, two steel angle anchors were installed at both ends of the beam. Secondly, the bottom surface of the beam was sandblasted with a rough depth of approximately 5 mm and thoroughly cleaned (Shang, Yu, and Zhang 2010). Thirdly, the well-mixed bonding adhesive (adhesive component A, adhesive component B, and silicon ash powder with a mass ratio of 2:1:1) was plastered on the rough surface to ensure the bonding strength between the concrete substrate and the cast-in-place polymer mortar layer, as shown in Figure 7a. Then, three strands of steel wire ropes were fixed to the steel anchor and prestressed to the designated prestress level (0.22 of the nominal tensile capacity, corresponding to 9.49 kN for each strand of the steel wire ropes) by tightening the nuts. Finally, the polymer mortar was plastered on the concrete substrate and covered with the steel wire ropes, as shown in Figure 7c.

For strengthening procedures II, the first three steps were the same as those of procedure I, as shown in Figure 7a. After the bonding agent was almost sticky (approximately 30 min as suggested by the manufacturer), a polymer mortar with a thickness of 25 mm was plastered on the concrete surface (in Figure 7d). Then, the steel wire ropes with smooth films were anchored and prestressed on the surface of the beam, followed by plastering of other polymer mortar with a 25 mm

Table 2. Main test parameters.

|           | Ratio of steel wire rope (%) | Interface conditions | Unbonding length, $L_{ub}$/mm | Strengthening procedures | Remarks          |
|-----------|------------------------------|----------------------|--------------------------------|--------------------------|------------------|
| CB        | –                            | –                    | –                              | –                        | Control beam     |
| SB-1      | 0.094                        | Perfect              | –                              | I                        | Strengthened beam|
| SB-2      | 0.094                        | Perfect              | –                              | II                       | Strengthened beam|
| SB-3      | 0.094                        | Defect               | 80                             | II                       | Strengthened beam|
| SB-4      | 0.094                        | Defect               | 200                            | II                       | Strengthened beam|
| SB-5      | 0.094                        | Defect               | 550                            | II                       | Strengthened beam|

The beams strengthened using three steel wire ropes with a reinforcement ratio of 0.094%; b: The interface conditions refer to the interface between the steel wire rope and PM layer, including perfect and defect.
2.5. Loading layer and polymer was embedded and to the mens.

- thickness, as shown in Figure 7e. The second layer of polymer mortar should be plastered before the setting and hardening of the first polymer-mortar layer. Each layer had a thickness of 25 mm, as shown in Figure 6a. Finally, the strengthened beams were cured in the laboratory environment.

2.5. Loading program and instrumentation

All the specimens were tested in four-point bending, as illustrated in Figure 8. Each specimen had a clear span of 3300 mm between the two supports and a constant moment region of 600 mm. The test was carried out under a force-displacement hybrid control method. The load was first conducted at a force rate of 5 kN/min up to the theoretical yield capacity of the specimens (70 kN for reference specimen CB and 120 kN for the strengthened specimens) and then changed to a displacement rate of 1 mm/min until the specimens occurred failure, which was when the load capacity decreased to 85% of the peak value or when the beams experienced excessive plastic deflection.

All the specimens were instrumented with sensors to measure the force, displacement, material strain, and crack width and spacing at key locations. The externally applied load was measured by a load cell embedded in the actuator. The vertical displacement was measured by linear variable differential transformers (LVDTs) and Laser displacement monitor (LDM), as illustrated in Figure 9. Three strain gauges (C1-C3 in Figure 9a) were applied on the top surface of the beam, while others were used to monitor the strain responses in the longitudinal reinforcing bars, as shown in Figure 9a. Since the surface of the steel wire rope was twisted, the average strain of the steel wire rope within 100 mm was measured by an alternative extensometer (Liu, Huang, and Guo et al. 2018) to strain gauge, as shown in Figure 9b. The average strain of the steel wire rope was the elongation divided by the 100-mm original length.

3. Experimental results and discussion

3.1. Torque coefficient

The experimentally obtained T-Q relationship from Section 2.2 is shown in Figure 10. Figure 10 shows that Q increases linearly with the increase of T. k and / had values of 0.2192 and 0.3802, respectively, with a correlation coefficient ($R^2$) of 0.9955.

The response of the tensile force (Q) of the steel wire rope and the monitoring time (t) is shown in Figure 11. After applying tension on the steel wire rope, the relationship between the effective tension on the steel wire rope and the test time showed a three-stage development trend in 72 h. The tensile force
Figure 7. Strengthening procedures of I and II: (a) roughing and cleaning the concrete substrate; (b) eye bolt-steel wire rope connection; (c) plastering polymer mortar layer after prestressing the steel wire ropes for procedure I; (d) plastering the first polymer mortar layer; (e) installation and pretension of the steel wire ropes; and (f) plastering the second polymer mortar layer.

decreased rapidly in the first 0.1 h, and the loss reached 50% of the total loss in 72 h. Then, the tensile force continued to decrease gently and became stable after 7 h. For example, a tightening torque of 25 N-mm was applied to the eye-bolt, which resulted in a corresponding tensile force of 9.49 kN, and the effective tensile force was 9.25 kN after 72 h, with a total loss of 2.5%. Those results will be used for Specimen details in section 2.3 and Strengthening procedures in section 2.4.

3.2. Failure modes

The failure modes for each specimen are shown in Figure 12. The control specimen CB exhibited a typical flexural failure mode, which included longitudinal steel reinforcement yielding followed by concrete crushing in the compressive zone, as shown in Figure 12a. Three vertical flexural cracks with a maximum width of 0.06 mm initiated within the constant moment region at a load level of 25.8 kN (the corresponding moment of 17.4 kNm). As the load increased, new cracks were generated and developed until the longitudinal steel reinforcement yielded at a load level of 85.6 kN (57.8 kNm), wherein the mid-span deflection reached 6.16 mm. When the applied load reached 104.3 kN (70.4 kNm) with a deflection of 28.3 mm, the control beam CB reached its ultimate state in which the compressive edge of the concrete crushing occurred.

Strengthened specimen SB-1 showed a typical flexural-dominated failure mode, in which the longitudinal steel reinforcement yielded and the concrete crushed in the compressive zone. Meanwhile, interfacial debonding failure between the polymer mortar layer and the concrete substrate was observed, as shown in Figure 12b. The vertical flexural cracks were initiated at a load of 31.1 kN (21.0 kNm), and multiple fine cracks diffused in the polymer mortar layer. As the load reached 119.1 kN (80.4 kNm) with a mid-span deflection of 12.42 mm, the longitudinal steel reinforcement yielded. When the deflection increased to 37 mm, an interfacial crack occurred along with the interface between the polymer mortar layer and the concrete substrate. As the load increased to 171.0 kN (115.4 kNm) with a deflection of 38.07 mm, failure was triggered by a sudden progression of the interface crack, resulting in the steel
Figure 8. Test setup (unit: mm): (a) tested specimen details; (b) actual setup.

Figure 9. Details of the measurement schemes (unit: mm): (a) measurement point locations; (b) instrumentation layout.
wire rope-polymer mortar reinforced composite layer peeling away from the concrete substrate. The debonding failure mode of specimen SB-1 was characterized by interface debonding initiating the end of the reinforced composite layer, and the partial separation penetrating into the concrete along with longitudinal steel reinforcement. This debonding failure mode was named as plate end debonding (PED) failure, as previous researches (Wu et al. 2010; Huang, Guo, and Yao et al. 2019). It is reasonable that the high interfacial shear and normal stresses near the plate end led to the plate end debonding failure (Smith and Teng 2001). Meanwhile, the wire ropes were densely arranged and anchored at steel anchor of composite end, as shown in Figure 7c. When plastering the polymer mortar at the composite end, it was difficult to guarantee the compaction between the polymer mortar and the concrete substrate, especially for the procedure I. It will weaken the bonding strength of the interface at the end of steel wire ropes-polymer mortar composite.

Strengthened beam SB-2 failed in a typical flexural failure mode, which was identified by the fracture of the steel wire ropes after the longitudinal steel reinforcement yielded, as depicted in Figure 12c. The initial flexural cracks occurred at a load of 35.1 kN (23.7 kNm) and then developed in height and number. When the load increased to 119.3 kN (80.5 kNm), the longitudinal steel reinforcement yielded. As the applied load increased, the beam failed with the rupture of the steel wire ropes near one loading point, and no debonding failure occurred. The failure mode of specimen SB-2 was an alternative failure mode to the debonding failure mode of specimen SB-1, which indicated that strengthening with procedure II could improve the compaction between the polymer mortar and the concrete substrate and restrain the plate end debonding failure.

Compared with specimen SB-2, similar flexural failure modes and crack patterns were observed for specimens SB-3, SB-4, and SB-5 with various unbonded lengths, as shown in Figure 12(d-f)). In particular, specimen SB-4 was reloaded due to an error of the loading equipment, which had a certain impact on its flexural behaviour. Since similar flexural failure modes and crack patterns led to the failed influential analysis of the unbonded length of the steel wire rope-to-polymer mortar interface on the failure modes, other flexural behaviour affected by the unbonded length should be further discussed in the following section.

### 3.3. Load-deflection response

The mid-span moment ($M$)–deflection ($\Delta$) curves of all the specimens are shown in Figure 13. Table 3 summarizes the key points in the flexural response, including the cracking moment ($M_{cr}$), the yielding moment ($M_{y}$) associated with the yielding of the longitudinal steel reinforcement (confirmed by a strain gauge), the ultimate moment ($M_{u}$), the failure moment ($M_{f}$) and the corresponding deflections of $\Delta_{cr}$, $\Delta_{y}$, $\Delta_{u}$, and $\Delta_{f}$. The failure moment ($M_{f}$) was defined as 85% of the ultimate capacity of the specimen.

The $M$–$\Delta$ curves of the strengthened specimens consisted of three branches, as shown in Figure 12. The curve was linear from the beginning up to the cracking load in the first stage and then almost linear with a lower slope until the longitudinal steel reinforcement yielded. Finally, the shape of the curve was mostly linear with slow growth until failure, after which the moment response of the strengthened beams was reduced to that of specimen CB, as depicted in Figure 13a.

For the control beam CB, the $M$–$\Delta$ curve was the typical curve of a balanced-reinforced beam, with an ultimate moment of 70.4 kNm and a corresponding mid-span deflection of 23.5 mm. Specimens SB-1 and SB-2 had a close cracking load of 30 kN to 35 kN.
(approximately 25 kNm), which was slightly higher than that of the control beam CB. This was due to the additional stiffness contribution of the 50 mm thick strengthening layer. The yielding of the longitudinal steel reinforcement was delayed in the post-cracking stage, due to the steel wire ropes providing a portion of the moment capacity when compared to the control beam CB, as shown in Figure 13a. In addition, there were increases in the peak moment of 64% and 66% for specimens SB-1 and SB-2, respectively, compared
strengthening of specimens SB-1 and SB-2 also validated that, strengthening procedure II effectively suppressed the catastrophic debonding failure and improved the material utilization, as shown in Figures 13a and 16.

The $M$-$\Delta$ curves and test results of the specimens with various unbonded lengths are shown in Figure 13b and Table 3. The figure and table show that the specimens had close cracking and ultimate loads and the same failure mode of the rupture of the steel wire ropes. However, the yielding loads decreased with increasing unbonded length, and the corresponding deflections increased as the unbonded interfacial lengths increased. Reasonably, the smooth film would weaken the bonding strength of the steel wire rope-polymer mortar interface and leading to the increased slip of the steel wire rope in the polymer mortar, and then a larger deflection for specimens.

### 3.4. Flexural stiffness

The flexural stiffness is an essential concern when evaluating the serviceability of strengthened RC members. The service load is characterized as a load range from the cracking load to the load where the midspan deflection equals to the span/480 (Obaydullah et al. 2016). The flexural stiffness is defined as the ratio of the service load to the corresponding deflection, as shown in Eq. (3). The deflection corresponding to the span length divided by 480 is calculated to be 6.46 mm, and the stiffness results are listed in Table 4.

$$k_{\text{stiff}} = \frac{\Delta P}{\Delta \delta} = \frac{P_{\text{cr}}}{\Delta \delta_{\text{cr}}}$$

where $\Delta P$ is the service load; $P_{\text{cr}}$ is the cracking load; $\delta_{\text{cr}}$, $\Delta \delta_{\text{cr}}$ and $\delta_{\text{cr}}$ are the corresponding deflection, respectively.

It can be seen from Table 4 that the flexural stiffness of the strengthened specimens generally exhibited an increase compared to that for specimen CB. These results illustrated that the prestressed steel wire rope-polymer mortar strengthening system was efficient in enhancing the ultimate and yielding moments of the strengthened beams.

Compared with specimen SB-1 adopting the strengthening procedure of I, the moments of specimen SB-2 were similar, but the corresponding deflections of specimen SB-1 were higher than those of specimen SB-2. It is reasonable that strengthening procedure I led to the poor bond strength of the steel wire rope-polymer mortar layer interface as well as the polymer-mortar-concrete substrate interface. As the applied load increased, slip occurred in the two interfaces mentioned above, resulting in an average strain in the unbonded length substituted to local concentrated strain. If slip continued to develop, a new strengthening system would occur, similar to the prestressing system for externally post-tensioning CFRP or steel. This changed strengthening system led to a larger deflection in the case with a sufficient anchoring length of the steel wire ropes. Meanwhile, the different failure modes of specimen SB-1 and specimen SB-2 also validated that, strengthening procedure II effectively suppressed the catastrophic debonding failure and improved the material utilization, as shown in Figures 13a and 16.

### Table 3. Summary of the test results.

| Beam | $M_u$ (kNm) | $M_y$ (kNm) | $M_f$ (kNm) | $\Delta_y$ (kNm) | $\Delta_i$ (kNm) | $\Delta_f$ (kNm) | $\Delta_u$ (kNm) | $\Delta_{u,2}$ (kNm) | Failure mode |
|------|------------|------------|------------|-----------------|-----------------|-----------------|-----------------|-----------------|--------------|
| CB   | 17.4       | 57.8       | 70.4       | 59.8            | 1.86            | 9.04            | 23.50           | 43.90           | 1.00         | SY→CC       |
| SB-1 | 21.0       | 80.4       | 115.4      | 98.1            | 1.40            | 9.04            | 28.98           | 59.56           | 1.66         | SY→EID      |
| SB-2 | 23.7       | 80.5       | 117.1      | 99.5            | 0.84            | 9.04            | 28.98           | 59.56           | 1.66         | SY→EID      |
| SB-3 | 26.6       | 75.5       | 116.8      | 99.3            | 1.07            | 9.41            | 36.25           | 66.29           | 1.66         | SY→EID      |
| SB-4 | 27.0       | 77.6       | 124.6      | 105.9           | 1.10            | 7.74            | 30.07           | 61.11           | 1.77         | SY→EID      |
| SB-5 | 26.6       | 66.6       | 120.0      | 101.7           | 1.27            | 10.01           | 40.52           | 62.31           | 1.77         | SY→EID      |

$M_u$ – cracking moment; $\Delta_u$ – cracking deflection; $M_y$ – yield moment; $\Delta_y$ – yield deflection; $M_f$ – ultimate moment; $\Delta_f$ – yield deflection, $M_i$ – failure moment, and $\delta_i$ – failure deflection; 2) SY – steel bar yielding, CC – concrete crushing, TP – tensile rupture of SWRs, and EID – plate end interfacial debonding.
increase over control specimen CB. For the strengthened specimens, the additional polymer mortar and the steel wire rope could suppress the initiation and development of cracks. Furthermore, the prestress of the strengthening materials was an effective method to further increase the flexural stiffness of the strengthened beam. On the other hand, the stiffness of specimen SB-1 was similar to that of specimen CB but lower than that of specimen SB-2. It is reasonable that strengthening procedure could improve the composite action between the strengthening layers and the concrete substrate.

For the strengthened specimens with various unbonded lengths of the steel wire rope-to-polymer mortar interface, the stiffness decreased as the unbonded lengths increased, and specimen SB-5 had the lowest stiffness and highest deflection. This is attributed to the fact that the smooth film weakened the interfacial bond strength between the steel wire ropes and polymer mortar, bringing about the slip of the steel wire rope occurring in the polymer mortar and then the increased deflection of the beams. For specimens SB-4 with two M–Δ curves, an alternate nominal flexural stiffness was calculated from the first M–Δ curve obtained from the first loading process.

3.5. Yield point definition and ductility

The yield point is a basic and critical factor for assessing the structural behaviour and that of the RC beam is usually defined as the load corresponding to the yielding of the longitudinal steel reinforcement. However, it is questionable whether the yielding of the strengthened members is defined by the yielding of only one material, which might not represent the yielding of the whole member (Feng, Qiang, and Ye 2017). Furthermore, some materials with almost linear elastic behaviour up to failure, such as steel wire rope or fiber-reinforced polymers, are often used to strengthen RC beams. In contrast, it is unreasonable to only confirm the yield point of these strengthened RC beams by the yielding of tensile steel reinforcement. Five methods, including the graphic method, Park method (k = 0.75), equivalent energy method, farthest point method (Feng, Qiang, and Ye 2017), and traditional method by tensile steel reinforcement yielding, are adopted to calculate the yield point, as shown in Figure 14 and Table 5.

The ductility is a crucial measurement of safety, as catastrophic failures on the RC structure can result in excessive damage and fatalities. Therefore, a ductility index and a deformability index are generally used to evaluate the ductility deformation capacity of strengthened structural member. The ductility index and the deformability index are given as follows:

\[ \begin{align*}
\mu_{\Delta u} &= \frac{\Delta u}{\Delta y} \\
\mu_{\Delta f} &= \frac{\Delta f}{\Delta y}
\end{align*} \]

where \( \mu_{\Delta u} \) and \( \mu_{\Delta f} \) are the ductility index at the ultimate load and the deformability index at the failure load, respectively; \( \Delta u, \Delta y, \) and \( \Delta f \) are the displacements corresponding to the ultimate load, failure, and yield load, respectively.

The results of the ductility and deformability indexes for the tested specimens are listed in Table 5. The yield points calculated by the graphic method and the farthest point method were close to those determined by the traditional method. The yield point determined by the farthest point method was to be the turning point of the M–Δ curve. Therefore, the farthest point method can be a suitable choice. The ductility indexes and deformability indexes of specimens SB-1 and SB-2 were lower than that of CB due to the lower ultimate strain of the steel wire ropes than that of the steel reinforcement and the increased stiffness of the strengthened beams. The ductility and deformability indexes increased with increasing unbonded length.

3.6. Moment-strain response

The strain distribution along the specimen depth is shown in Figure 15, where the origin of the y-axis represents the bottom of the beam. It is obvious that the strains are distributed approximately linearly along the beam depth, and the strain distribution meet the assumption of “the plane section remains plane after bending”. This is attributed to the maintained compatibility between strengthening layer and the RC beam up to failure. However, the strain of steel wire ropes increases quickly after the longitudinal steel bars yield, and incompatibility is found between different materials, as depicted in Figure 15.

Figure 16a exhibits the moment-strain responses of the longitudinal steel bars and steel wire ropes at the mid-span cross-section for specimen SB-2. The theoretical yielding and ultimate strains of the steel bars and steel wire ropes are calculated based on the experimental results, as listed in Table 2.

For the longitudinal steel bars and steel wire ropes, concrete cracking resulted in a decreased slope of the curves, and then the strain increased linearly with the
increase in the applied load until failure. Since debonding failure occurred in specimen SB-1, the measured strain in the steel wire rope reached 8.78 × 10^{-3}, which was only 82% of the theoretical ultimate tensile strain (approximately 10.67 × 10^{-3}). For specimens SB-2 and SB-5, the measured ultimate strain of the steel wire ropes reached 13.4 × 10^{-3}, which was larger than the theoretical ultimate strain. The measured ultimate strains in the steel wire ropes of specimens SB-3 and SB-4 were smaller than the theoretical strain, due to the rupture point of the steel wire ropes being far from the measured location. It should be mentioned that the strain data may not be reliable when the steel bar at the strain gauge position yielded (Sallam et al. 2010a; Sallam 2010b), which may be the reason for the disorganized strain data after the yield of steel bar, as shown in Figure 16b.

### 3.7 Strain distribution along with the beam length

Since the tensile steel reinforcement is a cross-section portion of the strengthened beam, its longitudinal strain distributions could offer an insightful examination into the interaction mechanism among the steel
wire ropes, polymer mortar layer, and RC beam substrate, as indicated by previous studies (Yang et al. 2018; Guo et al. 2020). Figure 17 shows the strain distributions of the longitudinal steel bars in specimens with various interfacial unbounded lengths, and the x-axis origin represents the mid-span location.

For specimen SB-2, the strain distributions of the longitudinal steel bars within the flexural-shear span were generally linear except for some zigzags from the initial applied load to the yield load. Afterward, the strains in the steel reinforcement increased considerably near the loading point until reaching the ultimate state. Previous research (Sallam 2010b) has found that the locations of strain gauges used could lead to the discrepancy of strain data. Therefore, despite a sudden increase in strain, the distribution of strain along the length of the steel rebar is smooth, which indicated that the steel wire rope and polymer mortar has good compatibility.

For the other specimens with various unbounded interfacial lengths, the strain distribution and development were similar to that of specimen SB-2, which was approximately linear in shear spans and flat in the mid-span. However, the substantial differences, as the higher curve slope within the shear span and the suddenly increased strains in a larger range, implied that a higher proportion of the load was carried by the steel bars for the other specimens compared to that of specimen SB-2. It is reasonable that the smooth film resulted in a poor interfacial bond strength and slip within the unbounded length and consequently decreased the utilized strain of the steel wire ropes. Compared with the other specimens, specimen SB-5 exhibited the highest curve slope in the shear span and the steel bar yielding in the longest range under the same load level, which indicated that interface debonding between the steel wire ropes and polymer mortar occurred in the range of interfacial unbounded length (Yang et al. 2018).

4. Calculation of the flexural strength
A model was proposed to predict the ultimate flexural capacity of strengthened RC beams with flexural failure model. The assumptions made in the model were as follows: (1) the plane section remains plane after bending; (2) the tensile strength of the concrete is ignored, and the nonlinear compressive stress-strain response of the concrete are considered according to the Chinese code (GB50010-2010, 2010); (3) the stress-strain relationship of the steel bars is considered to
be linear elastic-plastic; (4) the stress–strain relationship of the steel wire rope is considered to be linear elastic; and (5) the tensile strength of the polymer mortar is neglected.

The calculations of the compressive and tensile forces are derived based on the equilibrium condition in the cross-section, which can be described as follows:

\[ C_c + F'_s = T_s + T_w \]  

(4)

\[
\begin{align*}
C_c &= \int_0^x \sigma_c(\varepsilon_c) bdy \\
F'_s &= \sigma_s A'_s = E_s \varepsilon'_s A'_s \\
T_s &= \sigma_s A_s = E_s \varepsilon_s A_s \\
T_w &= \sigma_w A_w = E_w \varepsilon_w A_w
\end{align*}
\]

(5)

where \( C_c \) is the internal axial force resulting from the concrete compressive zone; \( F'_s \), \( T_s \) and \( T_w \) are the resultant forces of the compressive steel bar, tensile steel rebar, and steel wire rope, respectively; \( \sigma_c(\varepsilon_c) \) is the compressive stress of the concrete corresponding to the strain of \( \varepsilon_c \); \( \sigma_s \) and \( \varepsilon_s \) are the stress and strain of the compressive steel bar, respectively; \( \sigma_w \) and \( \varepsilon_w \) are the stress and strain of the steel wire rope, respectively; \( A'_s \), \( A_s \) and \( A_w \) are the cross-sectional areas of the compressive steel bar, tensile steel rebar, and steel wire rope, respectively; \( E_s \) and \( E_w \) are the elastic modulus of the steel bar and steel wire rope; and \( b \) and \( x_c \) are the width and depth of the concrete compressive zone, respectively.

The stress section of the compressive concrete zone can be simplified as an equivalent rectangular shape since the value and the action point of the resultant force \( C_c \) in the compressive concrete section are equivalent, as shown in Figure 18. The resultant force \( C_c \) in the compressive concrete block can be derived as follows:

\[
\begin{align*}
\int C_c &= a_1 f_c b \beta \varepsilon_c = a_1 f_c b x_c \\
x_c &= \beta x_c
\end{align*}
\]

(6)

where \( f_c \) is the cylinder compressive strength of the concrete (i.e., \( f_c = 0.8 f_{cm} \) in which \( f_{cm} \) is the cubic-concrete compressive strength); \( a_1 \) and \( \beta \) denote the ratio of the equivalent rectangular stress to \( f_c \) and the ratio of the depth of the equivalent rectangular stress

Figure 17. Longitudinal strain distribution of tensile bars at different loading levels: (a) Specimen SB-2; (b) Specimen SB-3; (c) Specimen SB-4; (d) Specimen SB-5.
block to $x_{c}$. $x_{c}$ and $x_{w}$ are the total height of the compressive zone and the depth of the equivalent rectangular stress block, respectively. Meanwhile, the following variable definitions of $\alpha_{1}$ and $\beta_{1}$ are applied (Hou, Li, and Gao et al. 2020):

$$\alpha_{1} = \begin{cases} 
\frac{2(3\epsilon_{c} - \epsilon_{c0})}{3\epsilon_{c0}} & 0 \leq \epsilon_{c} \leq \epsilon_{0} \\
\frac{2(3\epsilon_{c0} - \epsilon_{0})}{3\epsilon_{c0}} & \epsilon_{0} < \epsilon_{c} \leq \epsilon_{cu} 
\end{cases} \quad (7)
$$

$$\beta_{1} = \begin{cases} 
\frac{4\epsilon_{c0} - \epsilon_{c}}{3\epsilon_{c0} - \epsilon_{0}} & 0 \leq \epsilon_{c} \leq \epsilon_{0} \\
\frac{6\epsilon_{c0} - 4\epsilon_{c0} + \epsilon_{c}}{6\epsilon_{c0} - 2\epsilon_{0}} & \epsilon_{0} < \epsilon_{c} \leq \epsilon_{cu} 
\end{cases}
$$

where $\epsilon_{c}$ is the strain value on the compressive edge of the concrete and $\epsilon_{0}$ and $\epsilon_{u}$ are taken as 0.002 and 0.0033, respectively, according to the Chinese code GB50010-2010(2010).

The strains of various materials have a linear relationship based on the plane cross-sections assumption and are obtained as:

$$\begin{align*}
\epsilon_{y} &= \frac{x_{c} - x_{0}}{x_{c}} \epsilon_{c} = \frac{x_{c} - x_{0}}{x_{c}} \epsilon_{e} \\
\epsilon_{x} &= \frac{h_{0} - x_{c}}{x_{c}} \epsilon_{c} = \frac{h_{0} - x_{c}}{x_{c}} \epsilon_{e} \\
\epsilon_{w(\text{load})} &= \frac{h_{0} - x_{c}}{x_{c}} \epsilon_{c} = \frac{h_{0} - x_{c}}{x_{c}} \epsilon_{e}
\end{align*} \quad (8)$$

where $\alpha_{y}$, $h_{0}$, and $h_{w}$ are the physical dimensions of the strengthened RC beams, as shown in Figure 18.

The total strain $\epsilon_{w}$ in the steel wire ropes consists of the following strain values in the loading process:

$$\epsilon_{w} = \epsilon_{w(\text{press})} + \epsilon_{w(\text{load})} \quad \epsilon_{w} \leq \epsilon_{wu} \quad (9)$$

where $\epsilon_{wu}$ is the rupture strain of $10.67 \times 10^{-3}$ $\epsilon_{w(\text{press})}$ is the prestressing strain of $0.267 \times 10^{-3}$ due to the initial pretension on the steel wire ropes, and $\epsilon_{w(\text{load})}$ is the strain due to the applied loads.

It should be noted that the analysis of the flexural strength herein is only proper for the strengthened beam with the typical failure mode of tensile rupture of the steel wire ropes and tensile steel bar yielding in the constant-moment span. Based on the equilibrium condition in the cross-section, Eq. (1) can be given as the follows:

$$\alpha_{1}f_{y}A_{y} + E_{w}\epsilon_{w}A_{w} = f_{y}A_{y} + E_{w}\epsilon_{w}A_{w} \quad (10)$$

where $f_{y}$ is the yield stress of the tensile rebar. When the steel wire ropes rupture, the compressive strain of the steel bars gained from the strain gauge does not reach its yielding strain.

Therefore, the equivalent depth of $x_{c}$ can be gained based on Eq. (10) when the strain gauge in the test records the strain of the compressive concrete edge, $\epsilon_{c}$, at the ultimate load. Finally, the prediction for the flexural strength of the strengthened specimen, $M_{u}$, can be derived as:

$$M_{u} = \alpha_{1}f_{y}A_{y}(h_{f} - \frac{x_{c}}{2}) + \sigma_{c}(h_{f} - a_{y}) - f_{y}A_{y}(h_{f} - h_{0}) \quad (11)$$

The tested and calculated results of the tested beams are shown in Table 6. The ratio of the calculated values to the test results ranges from 1.0 to 1.1, indicating that the predicted values show good agreement with the experimental results.

5. Conclusions

The flexural response of RC beams strengthened with prestressed steel wire ropes and polymer mortar (PSWR-PM strengthening system) was investigated, and a model to predict the flexural strength of the strengthened beams was proposed. Within the scope of the parameters considered in this research, the main conclusions can be drawn as follows:

Table 6. Flexural strength comparison of the tests and predicted results.

| Specimen | $\epsilon_{w}$ (με) | $\alpha_{1}$ | $\beta_{1}$ (mm) | $M_{wp}$ (kN·m) | $M_{w}$ (kN·m) | $M_{wp}/M_{w}$ | Failure mode |
|-----------|---------------------|-------------|------------------|----------------|-------------|---------------|-------------|
| CB        | 2432                | 0.93       | 0.78             | 24.4           | 71.3        | 1.04          | SY→CC       |
| SB-1      | 1380                | 0.74       | 0.72             | 38.3           | 117.0       | 1.01          | SY→EID      |
| SB-2      | 1721                | 0.844      | 35.3             | 122.6          | 117.1       | 1.05          | SY→TP       |

$M_{wp}$: predicted moment, $M_{w}$: experimental moment, 1) SY: steel bar yielding, CC: concrete crushing, TP: tensile rupture of SWRs, EID: end interfacial debonding, ICD: intermediate crack debonding mode.
A relationship between the applied torque and tensile force exerted in the bolt was addressed experimentally. Tightening bolts could be applied conveniently and effectively to prestressing the steel wire ropes in the PSWR-PM strengthening system.

The prestressed steel wire-polymer mortar layer could work together with the RC beams, and subsequently increase the cracking, yielding, and ultimate moments as well as the flexural stiffness of the strengthened member.

The prestressed steel wire-polymer mortar strengthening adopting either procedures I or II could effectively enhance the yield and ultimate loads of the RC beams. The strengthening with procedure I was capable of suppressing the interface debonding failure of the strengthened beam and improving the utilization of the strengthening materials.

In the case of sufficient anchoring at both ends, the unbonded length of the steel wire ropes within the flexural-shear span has an unobvious influence on the ultimate capacity of strengthened beams. As the unbonded length increases, the stiffness increase decreases accordingly, while the key deflections were increasing due to the interfacial slip between the steel wire ropes and polymer mortar.

The proposed model shows good accuracy in predicting the ultimate capacity of RC beams, and the computed values agree well with both the experimental results from the current tests.

More research is still required to investigate the dynamic behaviour of RC beams strengthened with steel wire ropes and polymer mortar, such as the behaviour under fatigue loads and impact loads, by doing which the knowledge of the steel wire rope-polymer mortar strengthening system could be further enriched.

Disclosure statement

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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