Experimental Study on Shear Performance of RC Beams Strengthened with NSM CFRP Prestressed Concrete Prisms

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Abstract: This paper presents an experimental investigation of the shear performance of RC beams strengthened with near surface mounted (NSM) carbon fibre reinforced polymer (CFRP) prestressed concrete prisms (PCPs). The shear behaviour of strengthened beams can be affected by several design variables. In this research, the effect of the following parameters were considered: the prestress level, inclination and spacing of the CFRP-PCPs, and material type of the prism. The control beam had conventional shear steel reinforcement only while the other seven beams were shear strengthened with CFRP-PCPs by varying design parameters mentioned above. All the beams were tested under monotonic loading until they reached the failure load. The experimental results showed that the NSM CFRP-PCPs strengthening technique improves the shear performance of the beams effectively. The strengthened beams that applied the CFRP-PCPs at an inclination of 45° were more efficient in improving the shear capacity compared to vertical CFRP-PCPs. The shear capacity and deformation were enhanced with the increase of prestressing levels of CFRP rods and

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the decrease of CFRP-PCPs spacing. The failure modes of the strengthened beams were influenced mainly by the spacing and the inclination of the CFRP-PCPs. Moreover, the material type of the prism had little influence on the effectiveness of shear strengthening. The analytical model presented was developed to estimate the shear contribution of NSM CFRP-PCPs and the model was found to predict the shear capacity of the tested beams well.

Keywords: Shear Strengthening; near surface mounted NSM; carbon fibre reinforced polymer CFRP; prestressed concrete prisms PCPs

1. Introduction

Shear failure in reinforced concrete (RC) structures is generally brittle in nature and needs to be avoided via appropriate design. Ageing reinforced concrete structures usually exhibit shear cracks due to factors such as natural disasters, regular and unforeseen loads, loads not considered in the original design, and inadequate concrete strength due to production or ageing[1-2]. Therefore, an efficient and economic shear strengthening technique is required to solve the shear-deficiency problem in the existing damaged and ageing RC structures.

In recent decades, carbon fibre reinforced polymer (CFRP) composite has been widely used for shear and flexural strengthening of RC structures due to its various advantages, for instance, high strength to weight ratio, high fatigue strength, non-corroding properties, and high chemical resistance [3-5]. Therefore, a strengthening technique that applies the CFRP bars and laminates can effectively solve the problems that come with conventional strengthening techniques such as steel jacketing and concrete enlargement [6, 7]. The CFRP strengthening technique includes the external bonding (EB) technique with CFRP laminates, near surface mounted (NSM) method with CFRP laminates or rods, and external confinement (EC) using CFRP sheets [8-11]. Many researchers have suggested that the strengthening technique with CFRP rods or laminates can improve the flexural and shear behaviour of the deficient beams effectively [9, 12-14].

The NSM strengthening technique involves embedding the CFRP rods or laminates into grooves that are pre-cut on the concrete surface and bonding them to the concrete with an epoxy adhesive. It is the most effective CFRP strengthening method due to the following advantages: (a) it provides a larger bond area and higher confinement by the surrounding concrete, (b) it requires minimal installation time, (c) the concrete cover can protect the CFRP rods from vandalism, mechanical damage and fire [15, 16]. El-Hacha and Rizkalla reported that the NSM strengthening technique using CFRP bars and strips significantly improved the stiffness and provided a higher flexural capacity of RC beams compared with the EB strengthening technique [17]. Rahal and Rumaih tested the shear capacity of four RC T-beams strengthened with NSM CFRP bars and conventional steel reinforcing bars for a comparative study. The study showed that the NSM strengthening increased the ultimate shear capacity and cracking shear load by 37%-92% and 23%-85%, respectively. The NSM CFRP bars also reduced the width of the diagonal cracks and improved the ductility of the test regions [16]. Kuntal et al. 2017 studied the behaviour of the NSM technique to strengthen the shear capacity of prestressed concrete beams and revealed that NSM CFRP laminates oriented at 45° is more efficient in improving the shear capacity of beams in both configurations: with or without vertical stirrups [7]. Another very recent study from this research team aims to investigate the efficiency of different bonding agents on the NSM shear CFRP laminate strengthened high strength prestressed concrete beams. The beams were strengthened with NSM CFRP laminates which are oriented at 45-degree configuration and the beams are assessed by the three-point bending test. Experimental results revealed a similar performance between the high strength cement grout and geopolymer mortar but both are less efficient than the epoxy resin [18]. Some studies have also shown that the NSM CFRP shear strengthening technique can reduce the debonding failure which is commonly observed in the EB strengthening technique and it can enhance the shear resistance of RC beams significantly when compared with other strengthening techniques [13, 16, 19, 20].

The NSM strengthening using prestressed CFRP technique has the advantages of both the NSM and prestressing strengthening techniques. Many investigations have been conducted on the innovative strengthening technique using NSM prestressed CFRP rods or laminates to strengthen the RC beams [21-24]. These RC beams showed an increase of up to 79% in the ultimate flexure load compared to the control beam, as reported by Badawi and Soudki [21]. The prestressed strengthened beams exhibited a higher cracking load and lower deflection compared with the corresponding non-prestressed beams. Jung et al. [25] studied prestressed strengthening and concluded that the crack and yield loads increased with a higher level of prestress. However, the disadvantages of the NSM prestressed strengthening technique should be noted; the prestressing system cannot be removed until the filler is cured, and the anchorage system is costly. The strengthening technique that applies the carbon fibre reinforced polymer prestressed concrete prisms (CFRP-PCPs) is a good solution to these problems and is expected to improve the shear performance significantly.

CFRP-PCPs are bars of a small cross-section made of high-strength concrete that is concentrically pre-tensioned by a single CFRP bar [26]. The detailed fabrication method of CFRP-PCPs is detailed in Section 2. Previous research confirmed that beams reinforced with PCPs demonstrates superior performance before the cracking of the prisms and has a smaller deflection than control beams at ultimate limit state. The cracking load and flexural behaviour, under the service load condition, of these beams (strengthened with the NSM CFRP-PCPs) was improved with the increase of the prestress level of the prisms [26-29]. However, to date, there have been only a few studies on the shear strengthening of beams with NSM CFRP [7, 18, 30, 31] but no study with NSM CFRP-PCPs. Hence, the current study aims to investigate the shear performance of beams strengthened by NSM CFRP-PCPs under service and ultimate loads. A total of eight simply supported beams with the same cross-section were tested under monotonic four-point loading to rupture. The shear behaviour and failure mode were observed and analysed. The impacts of different design parameters on the test results were analysed and are presented in this paper. An analytical model to identify the contribution of the CFRP-PCPs in shear resistance is proposed and the results agree well with the experimental study.

2. Experimental programme

Before discussing the details of the experimental programme, it is necessary to discuss how the ultra-high performance concrete (UHPC) prism are strengthened by the prestressed CFRP and how this composite enhanced the shear capacity of the beam. If the UHPC prism is not prestressed, and the CFRP rod simply just embedded inside, the overall bending strength of this composite will be the normal composite strength without any enhancement as the threshold factor is still the relatively low tensile strength of the UHPC. Furthermore, when they are placed in the shear-tension direction in a shear span, under the shear-tension force, the cracks in the UHPC prisms will occur at its maximum tensile strength and that marks the start of the failure. The situation will be completely different if the CFRP rod is prestressed and maintained prestressed until the UHPC prism cast around it fully cured (CFRP-PCPs). Firstly, the bending strength will be increased as the prestressed CFRP rod, once released, will shrink back and pull the UHPC prism together with it, thus creates a pre-compressed status inside the UHPC prism. This compressive stress will counteract with the tensile stress generated by the bending moment and enhanced the overall bending strength of the composite prism. Secondly, when this CFRP-PCPs are placed along the shear-tension direction, the destructing tension force will need to overcome the prestressed compression force first inside the UHPC prism before it takes any tensile force, and that will not only increase the shear capacity but also will reduce the cracks in the prisms and the concrete glued around them. Finally, strengthened by prestressing, the CFRP-PCPs are also acting as stronger dowels and enhance the overall shear capacity of the beam.

2.1 Parameters of specimens

A total of eight RC beams were designed and cast in this experimental investigation. The un-strengthened beam (CSB), reinforced with steel stirrups only, was tested as a control beam to compare its shear behaviour with that of other beams strengthened with NSM CFRP-PCPs. The prestress level, inclination and spacing of the CFRP-PCPs, and material type of the prism were considered and compared to investigate the effectiveness of shear strengthening with different design details. The design parameters of the tested beams are listed in Table 1.

All tested beams were 250 mm in depth and 170 mm in width. The beams had an overall length of 2200 mm and were reinforced with the same longitudinal and transverse steel reinforcement. The beams were reinforced in tension with two longitudinal steel bars of 22 mm diameter and the percentage of the tensile steel bars was 3.2 %. The purpose of adopting high tensile reinforcement ratio is to ensure a shear failure rather than a flexure failure. Three steel bars of 10 mm diameter were applied in the compression zone. The shear span ratio (i.e. the ratio of beam shear span to the effective depth of the beam) of beams was 2.9 so that the arch effect can be neglected and the shear failure of the beams can occur as expected. The 6 mm steel stirrups spaced at 200 mm were applied in the flexural span. The shear span used 6 mm steel stirrups spaced at 300 mm and a steel stirrups ratio of 0.11 % [32]. The detailed design and configuration of the specimens are shown in Fig. 1. The reason of using 6mm diameter transverse bars is to reduce the shear capacity provided by the shear stirrups to create a weakened shear span so that the mechanical performance of the CFRP-PCPs can be examined properly, and to ensure a purpose-designed shear failure rather than a flexure failure.

2.2 Material properties

The concrete, reinforcement steel, UHPC and epoxy mortar are all tested according to the National Standard of China specified in Table 2 [33-36]. The tested beams were cast with a concrete mix produced by the local mixing plant in Liuzhou City in China. The beams and concrete cubes were cured for 28 days before further sample preparation and cube tests. Due to the amount of total testing time, and the high workload for setting up and the actual tests for all the sample beams, it is impossible to complete all the test in one day. Hence, it makes no sense to test the concrete cubes on the same day with the beams. All the beams were tested, however, within one week after 28 days of maturation. The concrete strength will not increase significant enough to have an impact on the shear capacity of the beams. Considering the concrete mixing ratio, maturation of the concrete are not within the scope of this study, the cubes tested after 28 days of maturation, during the testing period of the beams is the sensible solution adopted in this study. Nine concrete cubes (150 mm × 150 mm × 150 mm) were also cast and cured under the same conditions to test the compressive strength of concrete [33]. After curing for 28 days, an average compressive strength of 30.76 MPa was obtained from the standard compressive tests. Additionally, a tensile strength of 3.02 MPa and modulus of elasticity of 3.27×10^4 MPa were also obtained. Three standard samples were prepared for each type of reinforcement. The test results are listed in Table 2. The prestressed CFRP rod of 7 mm diameter, the UHPC and the epoxy resin mortar were all provided by Liuzhou OVM Machinery Co. Ltd. Nine UHPC cubes (100 mm × 100 mm × 100 mm) were also cast and cured under the same conditions for the CFRP-PCPs prefabrication [35]. Nine epoxy mortar cubes (40 mm × 40 mm × 160 mm) were also cast and cured under the same conditions for CFRP-PCPs installation [36]. The mechanical properties of the CFRP rod were specified by the Product Quality Certificates provided by the manufacturer detailed in Table 2. The specific properties of the materials applied to the tested beams are summarized in Table 2.

2.3 The fabrication of CFRP prestressed concrete prisms (CFRP-PCPs)

The CFRP prestressed prisms were concentrically prestressed by a single CFRP rod and cast with the UHPC or epoxy resin mortar as shown in Fig. 2. The fabrication method of CFRP-PCPs was as follows:

a) The CFRP rods were placed concentrically on the stretching pedestal and pre-tensioned by the prestressing apparatus as shown in Fig. 2. During the prestressing procedure, the strain and the prestress force were monitored through strain gauges mounted on the rod and the load cell placed at each end of the beam. The prestressing force at jacking was 23.1 kN and 38.5 kN, respectively. The prestressing force was 30% and 50% of the ultimate strength of the CFRP rod. The data was collected by the data acquisition system (DAQ). The prestressed CFRP rods were maintained concentrically using the tightened anchor nuts and cast with the UHPC or epoxy resin mortar to form a whole prism.

b) After casting, the CFRP-PCPs were cured for 15 days in a wet condition. The anchor nuts could be removed when the CFRP-PCPs reached the design capacity. Finally, CFRP-PCPs with a cross-section of 25 mm × 25 mm were obtained and ready to be applied on both sides of the beam for shear strengthening of the tested beams.

2.4 NSM strengthening technique with CFRP-PCPs

The tested beams met the design strength requirement after curing for 28 days. As the concrete cover is 30 mm thick, thus it is sensible to set the maximum groove size to 30 mm × 30 mm. Then the grooves which had a cross section of 30 mm × 30 mm were pre-cut on both sides of the shear span of the strengthened beams. The grooves were cleaned to remove dust so that it was easy to form a strong bond between the prisms and the concrete. Following that, the epoxy resin was poured into the grooves up to halfway and the CFRP-PCPs were embedded into the grooves on the concrete surface. The epoxy resin had a tensile strength of 22.5 MPa and compressive strength of 90 MPa (provided by the manufacturer).

The CFRP reinforcement ratio are 0.17%, 0.11%, 0.16%, 0.24% respectively. The strengthened beams were cured for at least 7 days under the standard condition to ensure the design bonding strength was attained.

2.5 Test setup and loading procedure

The tested beams were simply supported and subjected four-point monotonic loading using the servo-hydraulic controlled MTS actuator. All the tested beams were loaded with force-controlled at a rate of 3 kN/min. The applied load was converted to two-point loads through a steel spreader beam. After the appearance of the first crack in the beam being tested, the beams were tested in displacement-control mode at a rate of 1.2 mm/min, the increase in load was paused intermittently at intervals of 10 kN to observe and maintain constant for five minutes to allow the documentation of the position, length and opening of the shear cracks. There is no noticeable increase in deformation observed during the break. The strain gauges were located at the middle of both tensile and compressive bars and the shear reinforcement in shear span to monitor strain of the steel bars and CFRP rods during the test. A total of five strain gauges were also mounted on all tested beams evenly distributed on the surface of the middle span to measure the strain of the concrete during the loading process. To measure the deflection of the beams, five linear variable differential transducers (LVDT) were located at the load points, the supports and the mid-span of beams. The strain data was collected and recorded from the beginning of loading to failure using the data acquisition(DAQ) system. The test setup and instrumentation details are shown in Fig. 3.

3 Experimental results and discussion

3.1 The failure modes

Figure 4 shows the failure modes for the control beam and of beams strengthened by the NSM CFRP-PCPs technique. During the loading process, first-cracking occurred in the mid-span (strengthened beams) or the sections close to the loading points (control beam). As the applied load was increased, the vertical cracks appeared in shear span and developed to the critical diagonal crack propagating toward the loading points. The development of shear cracks was effectively hindered by NSM CFRP-PCPs and steel stirrups. However, after the first crack in NSM CFRP-PCPs, the crack broadened rapidly and developed toward the compressive zone of the beam-top.

In the test of the ultimate limit state, the primary failure modes of specimens all demonstrated or initiated by the shear-tension (diagonal-tension) failure. Three accompanied secondary failure types accelerated the rupture of the beams after the structures being weakened by the shear-tension cracks. The first secondary failure type was shear-compression failure characterized by the concrete crushing under the loading points because the shear crack went across the prisms and decreased the shear-compression zone of the beams. This failure mode mainly occurred in the control beam and the beams strengthened with CFRP-PCPs spaced at 300 mm. The second type was debonding of the CFRP-PCPs across the shear cracks and the concrete crushing under the loading points characterized the second mode. In general, the initial shear cracks developed between two prisms in the beams strengthened with vertical CFRP-PCPs spaced at 200 mm. The shear cracks propagated and went across the CFRP-PCPs as the load increased, which led to the detachment of concrete along with the debonding of CFRP-PCPs. The third type was bending failure, which was observed in the beam FCSB2-b, strengthened with 45° NSM prestressed CFRP-PCPs spaced at 200 mm. The propagation of shear cracks was suppressed effectively by the CFRP-PCPs and the mode of failure was flexure dominant. The beam had several flexure cracks and was less brittle than the other beams. As shown in Fig. 4, the application of NSM CFRP-PCPs arrested the propagation of shear cracks effectively so that the ultimate capacity of strengthened beams was improved significantly compared with that of control beam. The strengthened beams exhibited more ductile behaviour due to effective control of shear cracks by NSM CFRP-PCPs as well as steel stirrups.

3.2 Load-displacement relationship

The load-displacement curves of the control beam and strengthened beams are presented in Fig. 5. The displacement is measured by the displacement sensor No 1,2 and 4. The value of the displacement is calculated in this way: LVDT 4 – (LVDT 1+LVDT 2)/2. The locations of the displacement sensors are shown in Fig. 1(a). The load-displacement curves of all specimens were similar until the shear cracks initially appeared in the control beam. The displacement of the control beam increased relatively rapidly due to the decrease of stiffness. The shear-strengthened RC beams with NSM CFRP-PCPs continued to carry increased loads. When the first cracking occurred in the strengthened beams, the decrease of stiffness was smaller than in the control beam because the propagation of shear cracks was hindered by the CFRP-PCPs. This indicated that the CFRP-PCPs bridge the shear crack thus providing load resistance, which has also been reported in the previous experimental research [22]. All the experimental results of the tested beams are summarized in Table 3. The first shear crack of the control beam occurred at 40 kN while the strengthened beams had a higher load level, ranging from 75 kN to 130 kN. This shows that the NSM CFRP-PCPs configurations can delay the occurrence of shear cracks effectively and convert the sudden brittle shear failure to shear-compression mode. Thus, the beams strengthened using the NSM CFRP-PCPs technique have higher stiffness and load-carrying capacity than the control beam with only conventional steel reinforcement.

The ultimate load of the control beam was 156.13 kN. The shear strength of all the strengthened beams was higher than that of the control beam. For example, the ultimate load of beam FCSB2-b was 317.65 kN, showing a remarkable increase of 103.45% over the control beam. It can be seen from the comparison of beams FCSB1-a, FCSB1-c and FCSB1-e that the load corresponding to the formation of the first shear crack increased as the pre-stress level increased. Therefore, the strengthened beam using CFRP-PCPs with higher pre-stress levels can arrest the formation and development of cracks more effectively. This leads to the higher stiffness and load corresponding to the first shear crack than that with lower prestress level. However, the CFRP-PCPs in 50% prestressed-strengthened beam (FCSB1-e) failed by debonding when the applied load approached the ultimate shear capacity. Therefore, the ultimate load of beam FCSB1-e was slightly lower than that of 30% prestressed-strengthened beam FCSB1-c due to the debonding of CFRP-PCPs. For the prestressed-strengthened beams with higher prestress levels, the strength of CFRP-PCPs was not fully utilised resulting in only a slight increase in the ultimate load.

The results show that the shear strength of the strengthened beams increases with the decrease in spacing of NSM CFRP-PCPs, as the other parameters were kept the same. In comparison to beam FCSB1-b, the ultimate load of beam FCSB2-a was increased by 26.33%. The beam FCSB2-b, strengthened with the CFRP-PCPs of 45° inclination, had a 19.88% increase in the ultimate load compared with the beam FCSB1-c, strengthened with vertical CFRP-PCPs. Additionally, the shear strengthening technique with 45° NSM CFRP-PCPs in beam FCSB2-b effectively averted brittle shear failure and the failure was due to ductile flexure. The shear failure crack tended to be almost vertical to the inclined 45° CFRP-PCPs. Also, the total bonding length of the NSM inclined CFRP-PCPs was higher than that of vertical CFRP-PCPs. These factors contributed to the better shear performance of beams with NSM CFRP-PCPs applied at 45° inclination, which exhibited the highest increase in strength and ductility. The material type of the prisms displayed a slight effect on the shear behaviour of the strengthened beams through the comparison of load-displacement curves between FCSB1-c and FCSB1-d.

3.3 Load-strain relationship in tensile steel bars

The load versus tensile steel strain relationship for the tested beams is shown in Fig. 6. From the initial loading to the formation of the first crack in the beam, the strain in tensile steel increased slowly and had a linear tendency. The crack appeared in the strengthened beams when the load ranged from 35 kN to 40 kN. After the concrete cracked, the external load was transferred to the tensile steel bars and the CFRP-PCPs. Therefore, the strain in the tensile steel bars increased rapidly with only a slight increase of applied load. The control beam, CSB, failed in shear-compressive mode at a small tensile steel strain of 1286 με while the tensile steel bars did not yield. For the strengthened beams, the ultimate strain of the tensile steel bars almost reached the yield strain and was higher than that of control beam. This can be explained by the shear strengthening with the NSM CFRP-PCPs, contributing to the crack resistance and the ultimate load. Additionally, the shear strengthening technique converted the brittle pure shear failure to the less brittle failure mode. It is worthy to be noted that the data collected from some strain gauges installed on the compression and lateral steel bars were intermittent and incomplete. This may be due to the poor installation and too small size of lateral steel bars were used. So that this part of data was not adopted. Some strain gauges installed in the CFRP-PCPs were used to monitoring the prestressing process during the prefabrication stage. They were not connected to the DAQ system. The details of the location of the strain gauges and the displacement sensors are specified in section 2.5. No strain gauges installed on the concrete.

4 Analytical formulation

The shear strengthening using NSM CFRP-PCPs is a highly effective technique to improve the shear resistance of the shear-deficient RC beams. The mechanical properties of the NSM CFRP-PCPs during the loading process can be considered analogous with that of shear steel stirrups. This analytical formulation only considered the shear capacity of beams that failed in shear with the rupture of CFRP-PCPs. The beams that failed due to debonding between the CFRP-PCPs and concrete was not included in the formulation due to its complicated mechanism. This problem should be researched in future work. According to modern design codes, the shear capacity of the strengthened beams was assumed to be due to the contribution of shear strength provided by different shear resisting components. The nominal shear capacity (V_n) of an RC member strengthened in shear with NSM CFRP composites can be obtained using

$$V_n = V_c + V_s + V_f \tag{1}$$

where V_c, V_s, and V_f are the nominal strengths provided by the concrete, steel stirrups and CFRP-PCPs, respectively.

The contribution of the NSM CFRP-PCPs for the nominal shear capacity of strengthened beams was calculated by the proposed analytical formulation. The analytical model was based on two cases:

Case 1: the contribution of vertical NSM CFRP-PCPs to the shear capacity.

The applied load was transferred to the CFRP rod after the rupture of CFRP-PCPs. The strain of CFRP rods in CFRP-PCPs, from the initial loading to the fracture of CFRP-PCPs, is consistent with that of UHPC prisms, i.e. the strain of CFRP rods when the shear failure occurred in the strengthened beams was assumed to be that of prisms in fracture. The contribution of NSM CFRP-PCPs shear strengthening is expressed as:

$$V_f = \left(\varepsilon_f \cdot E_f \cdot A_f + T_p\right) \cdot \frac{h_0}{s_p} \tag{2}$$

$$T_p = \sigma_{pe} A_f + f_p A_p \tag{3}$$

$$\varepsilon_f = (\frac{T_p}{A_f} - \sigma_{pe})/E_f \tag{4}$$

where ε_f is the strain of CFRP rods corresponding to the rupture of UHPC prisms, E_f is the elastic modulus of CFRP rods, A_f is the total area of vertical CFRP rods in the same cross-section of beam, T_p is the maximum tension load before the first crack for CFRP-PCPs, h_0 is the effective height of the tested beam, S_p is the spacing of CFRP-PCPs, σ_{pe} is the effective prestressing stress of CFRP rods, f_p is the ultimate strength of UHPC in CFRP-PCPs, A_p is the total area of prisms in the same cross-section of the beam.

Case 2: the contribution of inclined (45°) NSM CFRP-PCPs to the shear capacity.

During the loading process, the shear cracks that formed were nearly vertical to the inclined CFRP-PCPs in the strengthened beams. The analytical formulation for strengthened beams in Case 2 was proposed based on the calculated model of the beams reinforced with shear bend-up bars in China code [32]. The shear resistance of the NSM CFRP-PCPs at an inclination of $45°$ can be calculated as:

$$V_f = 0.8 \times \left(f_f A_f' + T_p\right) \cdot \sin\alpha \tag{5}$$

$$T_p = \sigma_{pe} A_f' + f_p A_p' \tag{6}$$

$$f_f = \frac{T_p}{A_f} - \sigma_{pe} \tag{7}$$

where f_f is the stress in CFRP rods when the beams failed in shear failure, $A_f^{'}$ is the total area of $45°$ CFRP rods in CFRP-PCPs crossed by the shear cracks, $A_p^{'}$ is the area of the prisms in CFRP-PCPs crossed by the shear cracks.

The analytical formulation is applicable to the beams FCSB1-b and FCSB2-b that failed in shear failure with the rupture of CFRP-PCPs. The analytical values of shear strength provided by the NSM CFRP-PCPs obtained from the proposed equations were compared with the test results, as shown in Table 4. For the beam FCSB1-b, strengthened with vertical CFRP-PCPs spaced at 200 mm, the ratio of experimental value to calculated value was 0.91. Additionally, the ratio of 0.81 was obtained for the beam FCSB2-b, strengthened with $45°$ CFRP-PCPs. It can be concluded that the proposed analytical formulation is reliable in the prediction of the contribution of NSM CFRP-PCPs in the shear capacity of strengthened beams. However, this research only proposed the analytical formulation of beams that failed due to the rupture of CFRP-PCPs and the number of beams with such a failure mode were limited in this study. Hence, further work related to the shear-strengthened beams using the proposed strengthening technique should be conducted to verify the accuracy of the analytical formulation.

5 Conclusions

An experimental study was conducted to investigate the shear performance of RC beams strengthened by NSM CFRP-PCPs. The contributions of the NSM CFRP-PCPs strengthening technique to the shear capacity was evaluated using an analytical formulation and compared with the experimental results From the experimental and analytical results presented in this research, the following major conclusions can be obtained:

1. The shear capacity of RC beams strengthened significantly by using the NSM CFRP-PCPs technique. A 65.23% to 103.45% increase was observed compared to the non-strengthened control beam. The results indicate that the NSM CFRP-PCPs strengthening technique is an effective method for improving the stiffness and the shear capacity of shear-deficient beams.
2. The NSM CFRP-PCPs can delay and prevent the propagation of shear cracks, which enhances the shear capacity and ductility, and the ultimate deflection was also increased substantially compared with the control beam.
3. The experimental results show that the shear capacity and the ultimate deflection were increased when the spacing of the CFRP-PCPs decreased and the level of prestressing increased. The CFRP-PCPs installed at 45° inclination have also demonstrated a better shear performance than those with vertical setup. The type of material for prisms has an insignificant impact on the shear capacity of the RC beams.
4. The analytical model proposed is only applicable when the beams are failed due to the rupture of CFRP-PCPs. The analytical results agreed well with the test results. The formulation was approved to be reliable and accurate on predicting the shear capacity that strengthened by the NSM CFRP-PCPs.

5. A further parametric study would be very useful to explore the limitation of the analytical model and formulation for predicting the shear capacity of the RC beam strengthened by the NSM CFRP-PCPs. The analytical model proposed in this study will provide an important reference for future studies.

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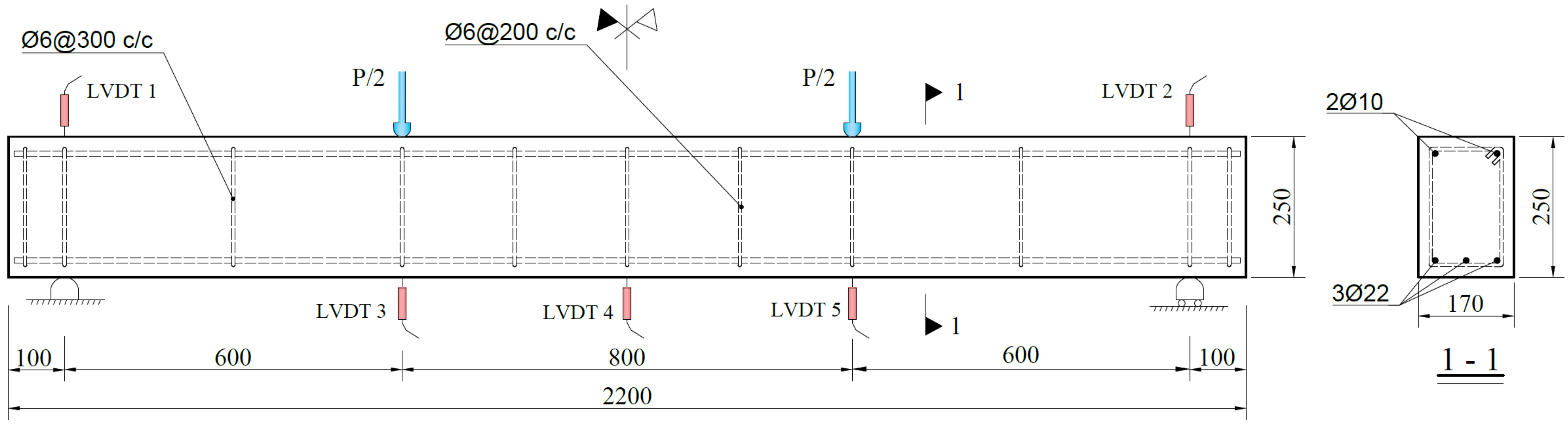
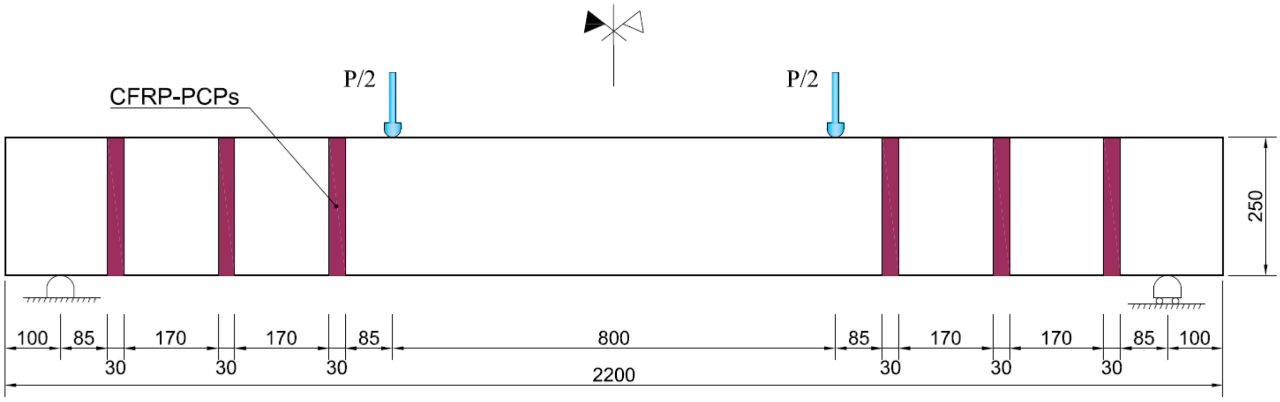
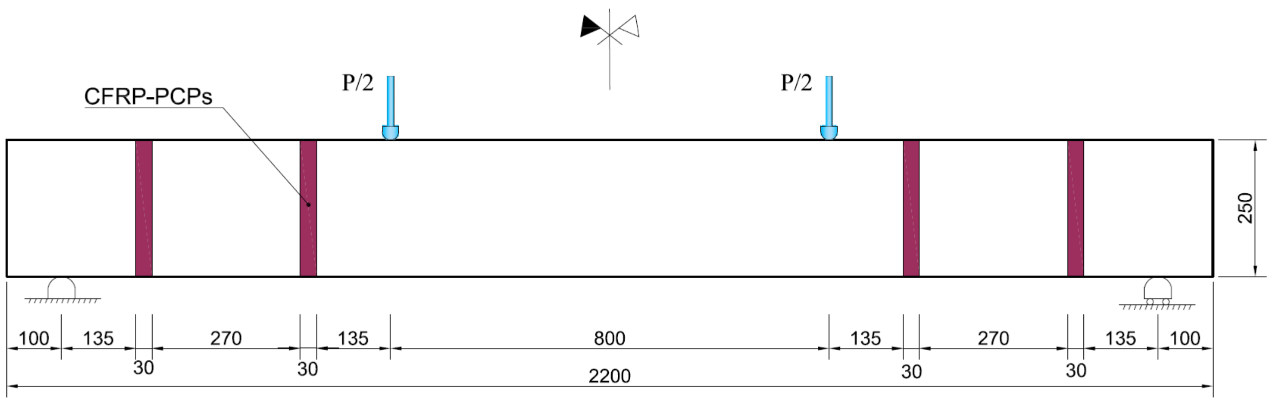


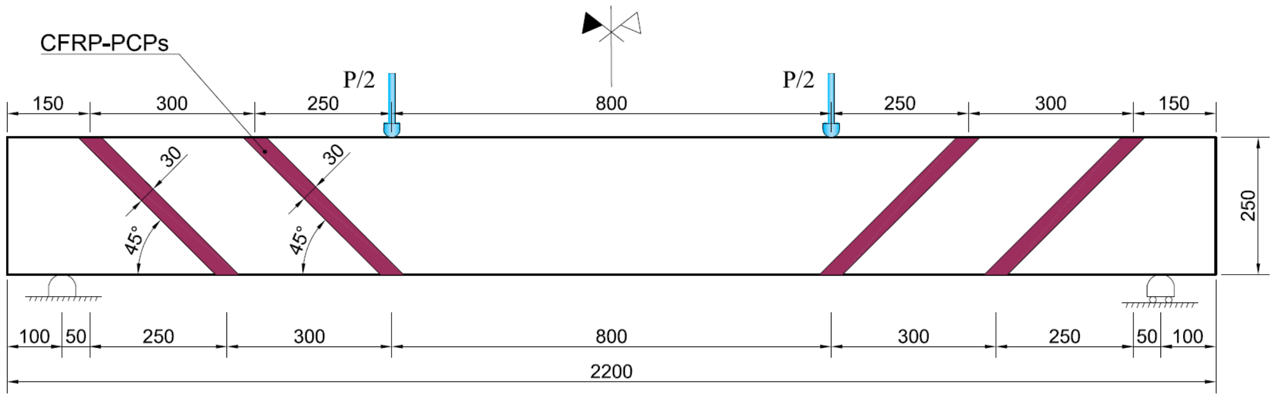
a) Specimen detail and locations of displacement sensors



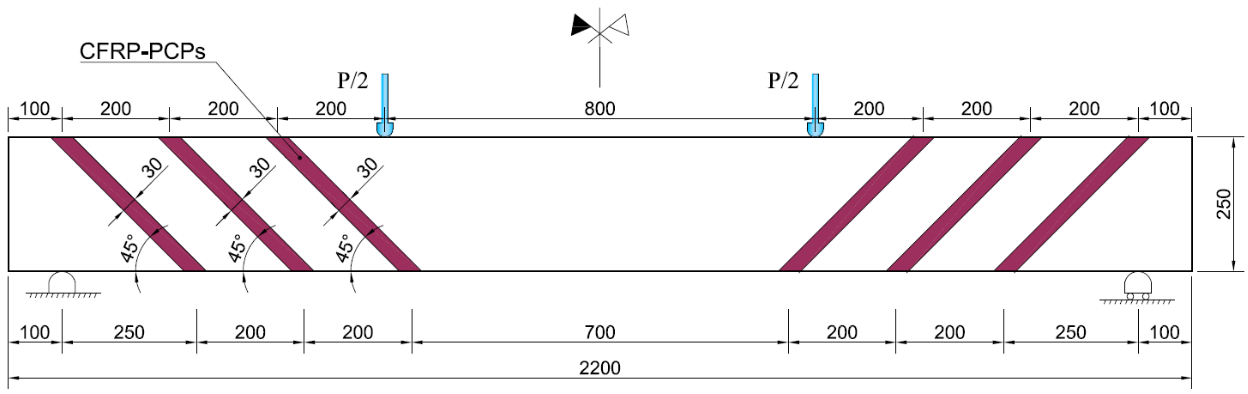
b) Beams FCSB1-a、FCSB1-c、FCSB1-d、FCSB1-e.



c) Beam FCSB1-b



d) Beam FCSB2-a



e) Beam FCSB2-b

Fig.1a-e. Specimen design and shear strengthening configurations using NSM CFRP-PCPs (mm)

a) Installation of CFRP rods.

b) Prestressing the CFRP rods.

c) Pouring the concrete.

d) The finished CFRP-PCPs.

e) Strengthening procedure.

f) Strengthened beams.

Fig. 2. The fabrication of CFRP-PCPs and NSM strengthening process.

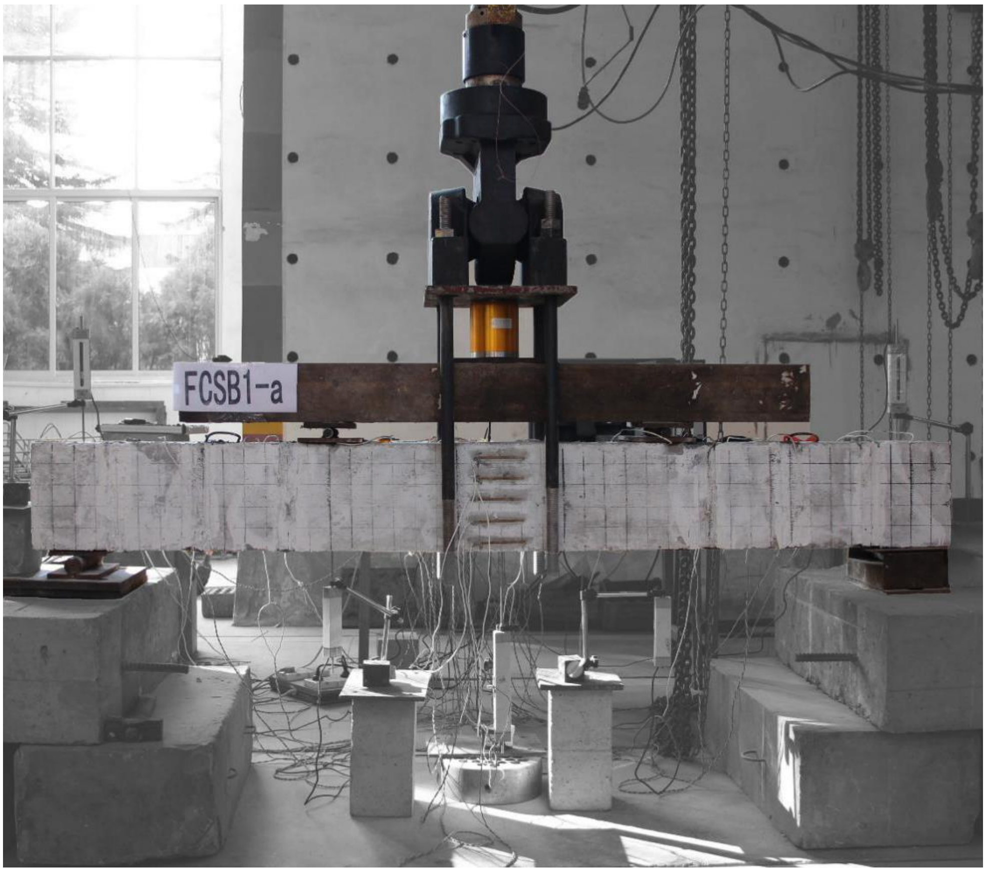
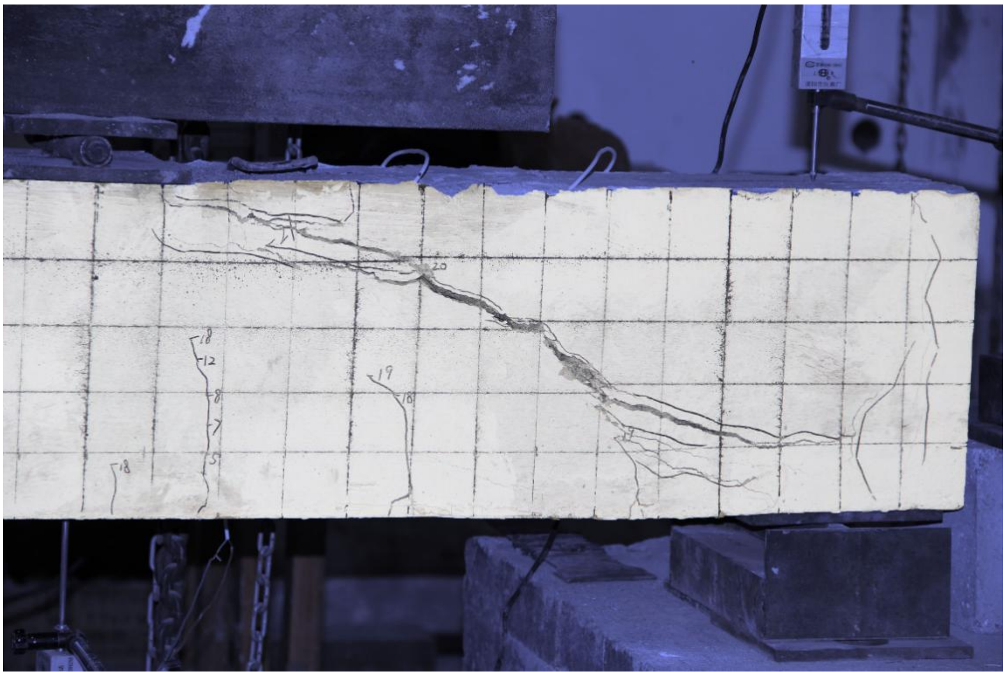
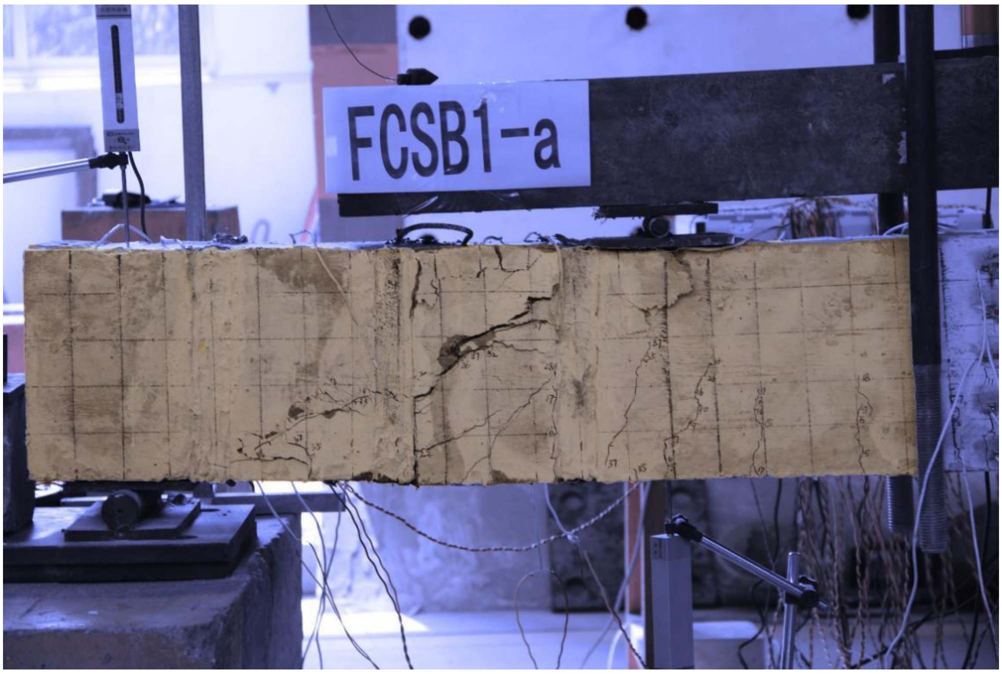


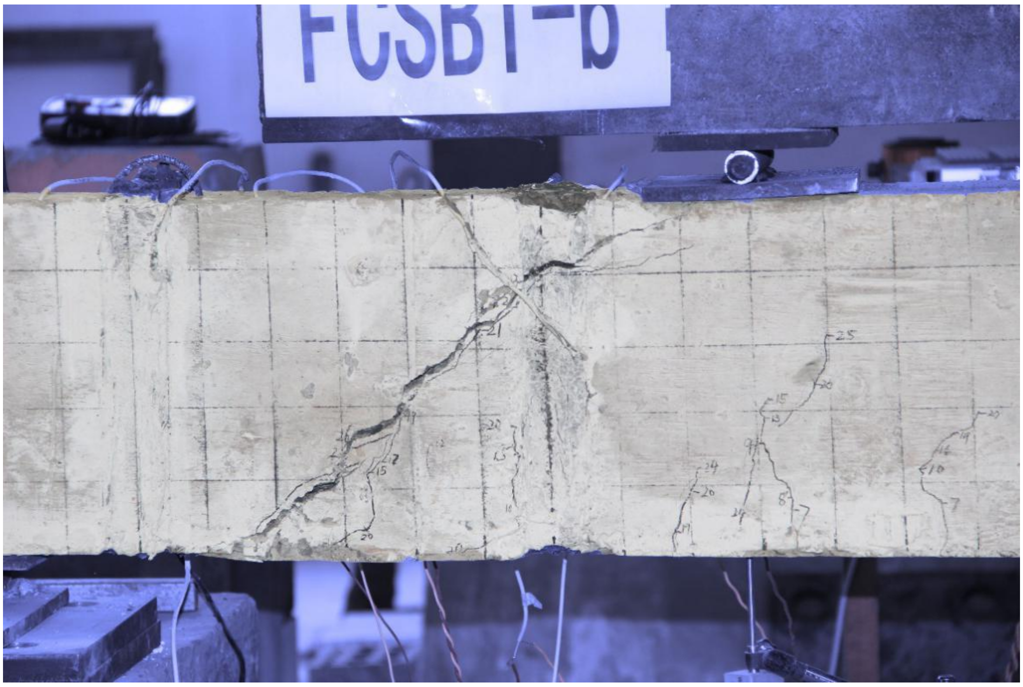
Fig. 3. The test setup and instrumentation



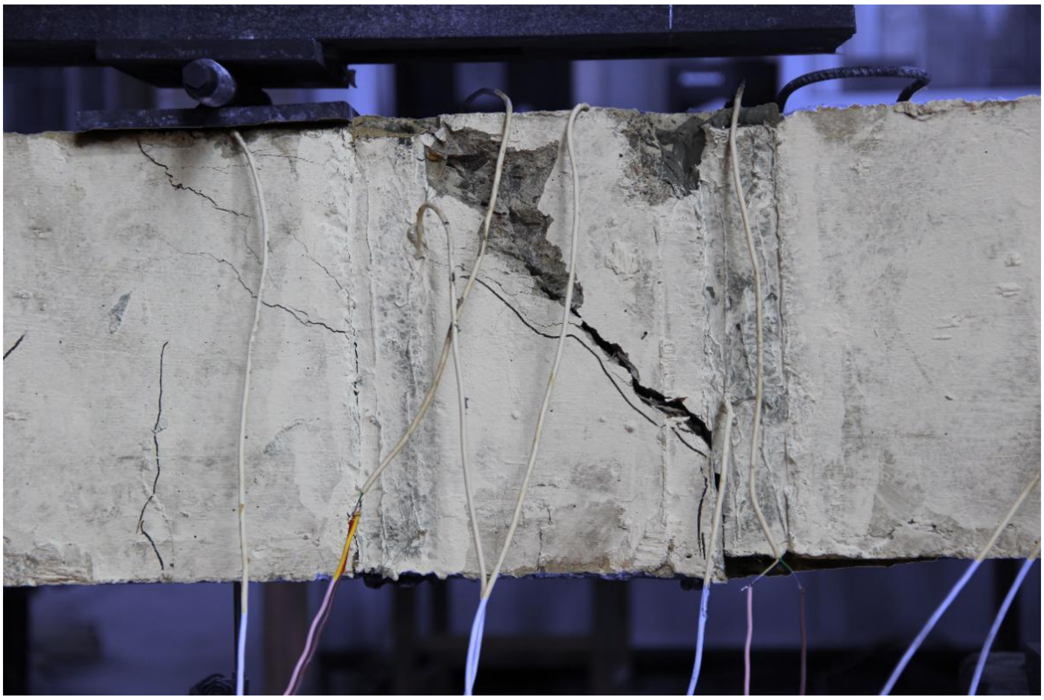
a) Control beam



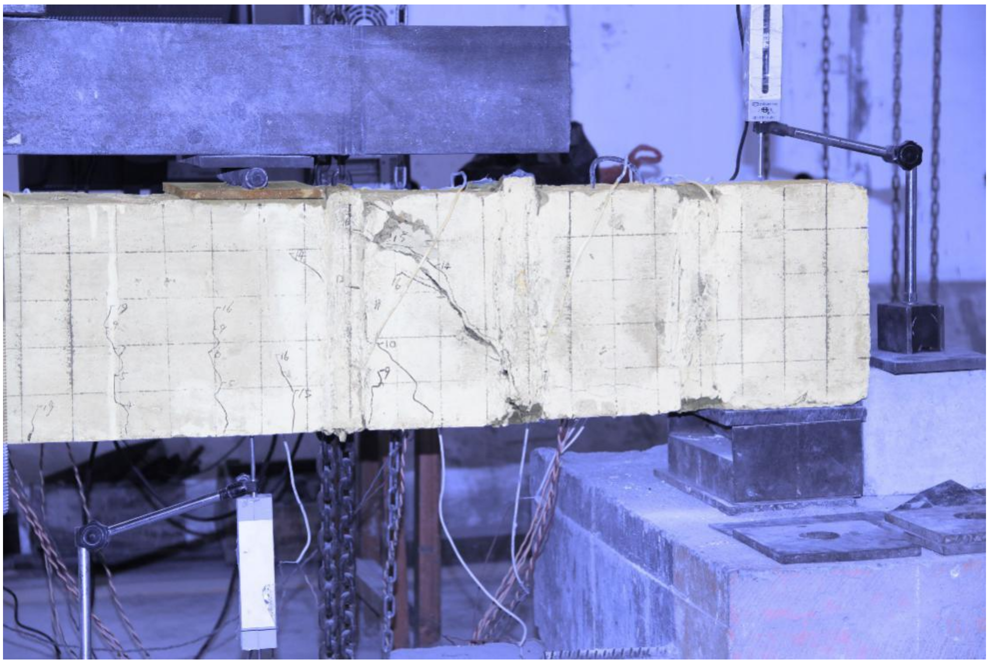
b) Beam FCSB1-a



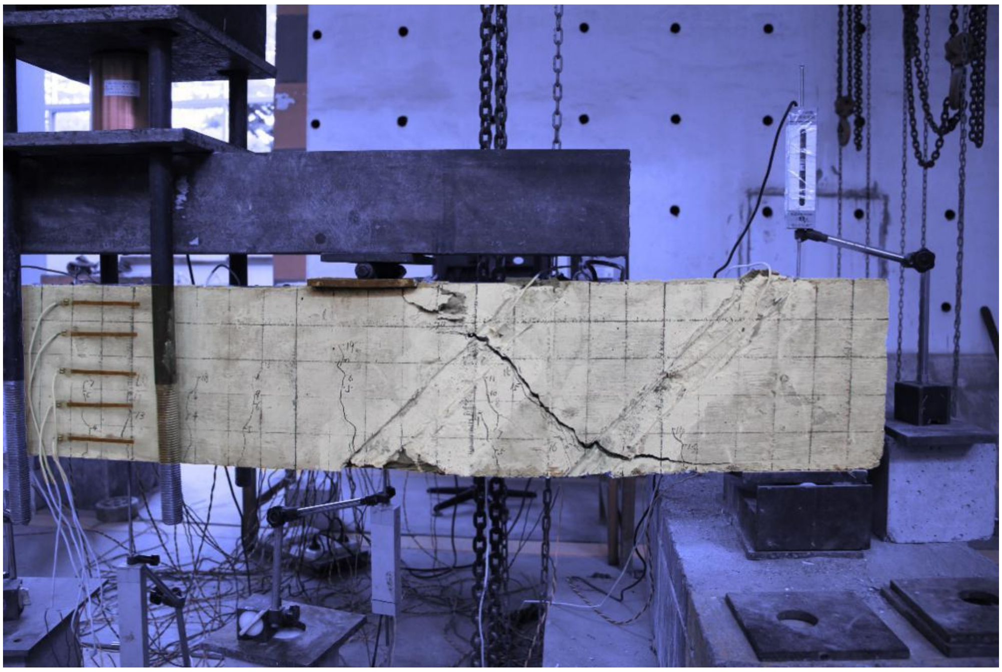
c) Beam FCSB1-b



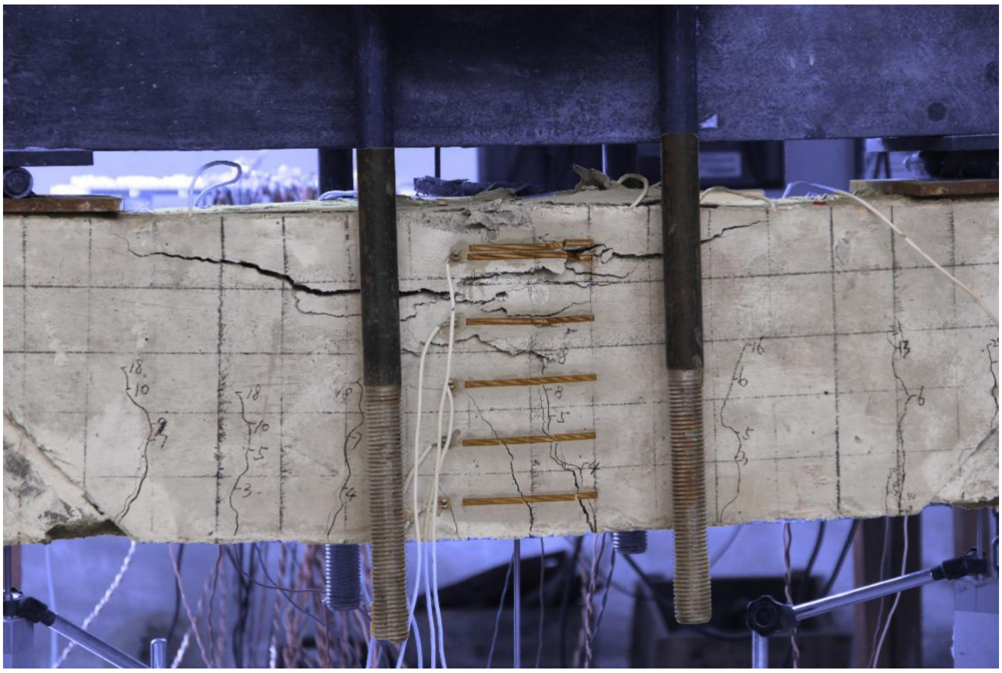
d) Beam FCSB1-c



e) Beam FCSB1-d



f) Beam FCSB2-a



g) Beam FCSB2-b

Fig. 4a-g. Failure pattern of control beam and shear strengthened beams

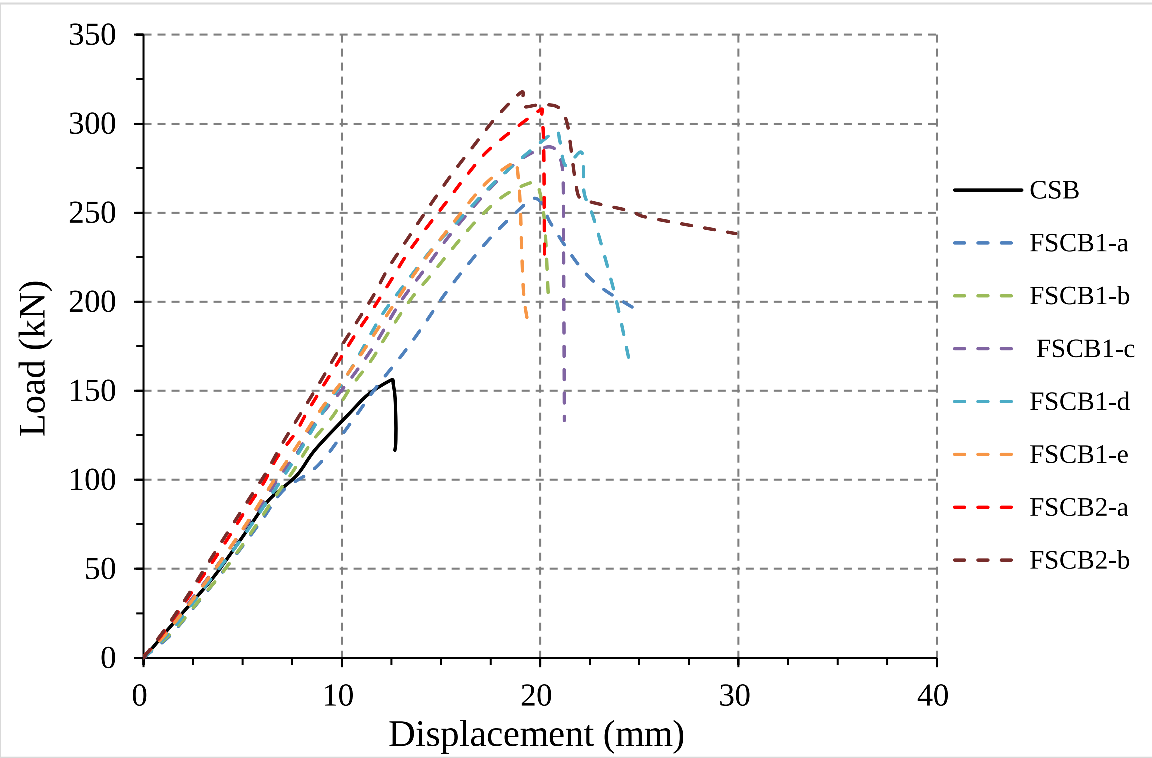


Fig. 5. The load versus displacement relationship of the tested beams

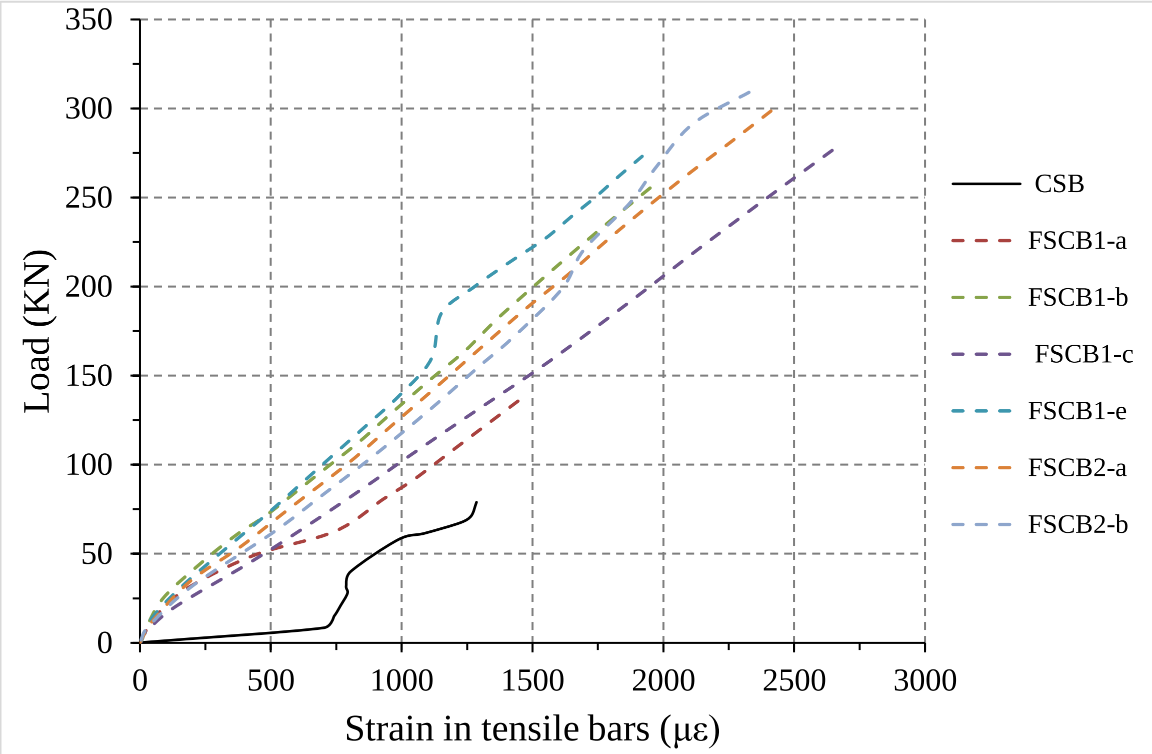
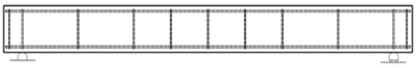
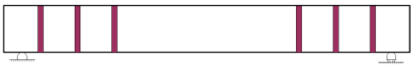
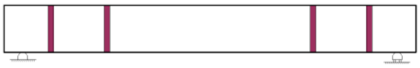
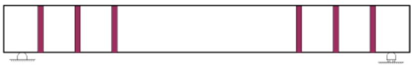
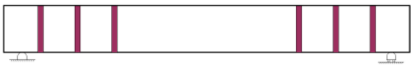
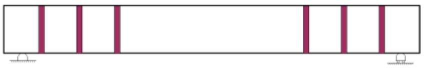
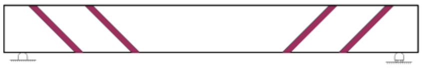
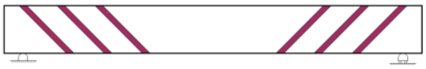


Fig. 6. The load versus strain relationship in tensile steel bars

Table 1: Fabrication details and parameters of the tested beams

Specimens	Spacing (mm)	Inclination of CFRP-PCPs[a]	Prestress level[b]	Material type of prisms[c]	Shear strengthened
CSB 	200	-	-	-	NO
FCSB1-a 	200	90°	0	UHPC	
FCSB1-b 	300	90°	30%	UHPC	
FCSB1-c 	200	90°	30%	UHPC	
FCSB1-d 	200	90°	30%	Epoxy resin mortar	YES
FCSB1-e 	200	90°	50%	UHPC	
FCSB2-a 	300	45°	30%	UHPC	
FCSB2-b 	200	45°	30%	UHPC	

[a] The inclined angle of the strengthening CFRP-PCPs with respect to the longitudinal axis of the tested beam.
[b] The prestress level ranged from 30% to 50% of the ultimate strength of the CFRP rods.
[c] The prisms were cast with different materials including ultra-high performance concrete (UHPC) or epoxy resin mortar.

Table 2: Mechanical properties of materials

Materials		Modulus of elasticity (GPa) [cov] [a]	Compressive strength (MPa) [cov]	Tensile strength (MPa) [cov]	Yield strength (MPa) [cov]
Concrete [34]		32.7 [14.7%]	30.76 [13.3%]	3.02 [14.4%]	—
Steel bars [35]	Φ 22	211 [5.7%]	—	665 [2.4%]	465 [2.2%]
	Φ10	221 [6.2%]	—	620 [2.6%]	445 [2.8%]
	Φ 6	207 [4.1%]	—	610 [6.5%]	424 [6.3%]
CFRP rod [b]		155		2400	2000 [c]
UHPC [36]		43.5 [11.5%]	154 [12.5%]	17.4 [12.2%]	—
Epoxy resin mortar [37]		85 [7.2%]	75 [7.5%]	20 [7.4%]	—

[a] [cov] is the coefficient of variation.
[b] Mechancial properties specified in the Product Quality Certifications provided by the manufacturer, OVM Machinery Co. Ltd (http://www.ovm.cn/).
[c] The nominal yield strength of CFRP rods, equivalent to 85 % of the ultimate strength, prewarning upper limit in practical design.

Table 3: Experimental results of the tested beams

Specimen	Ultimate load (kN)	Improvement[a] (%)	Failure mode
CSB	156.13	—	Shear-tension
FCSB1-a	257.97	65.23	
FCSB1-b	266.93	70.97	
FCSB1-c	286.61	83.57	
FCSB1-d	295.67	89.37	
FCSB1-e	278.69	78.5	
FCSB2-a	308.05	97.3	
FCSB2-b	317.65	103.45	

[a] The improvement in ultimate load of the shear-strengthened beams compared with the control beam.
[b] The beam FCSB1-e failed in shear-compression failure and debonding failure.

Table 4: Comparison between the experimental and analytical values of shear strength

Specimen	Experimental values (kN)		Analytical values (kN)		Ratio	
	V_f	V_n	V_f'	V_n'	V_f/V_f'	V_n/V_n'
CSB	-	156.13	-	136.81	-	1.14
FCSB1-b	110.80	266.93	121.28	258.09	0.91	1.03
FCSB2-a	151.92	308.05	187.07	323.88	0.81	0.95
FCSB2-b	-	317.65	-	315.38	-	1.01