Research Article

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Progressive collapse performance of shear strengthened RC frames by nano CFRP

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Abstract: To investigate the effect of nano carbon-fiber-reinforced polymer (nano CFRP) on retrofitting reinforced concrete (RC) structures, this article presented the progressive collapse performance of two half-scale four-bay RC frames including one control specimen S1 and specimen S2 with nano CFRP retrofitting beam ends. The progressive collapse performance was discussed by comparing crack patterns, failure mode, load–displacement curves, horizontal displacement, strains, and rotational and energy dissipation capacity. The test results demonstrated that nano CFRP did not change the ductile beam-hinge failure mode, but improved the load-bearing capacity and ductility efficiently, and postpone shear failure of the joint core region obviously. Due to the insufficient development of plastic hinges limited by nano CFRP, S2 failed to enter catenary action, which revealed its own defects in anti-progressive collapse. It was shown that the retrofitting strategy using nano CFRP can substantially improve the mechanical characteristics and implement the strong column-weak beam failure mode.

Keywords: nano CFRP, reinforced concrete frame, progressive collapse, strengthening, compressive arch action, catenary action

1 Introduction

In the past decades, several buildings have experienced partial or total progressive collapse due to blasts, terror attacks, and other extreme circumstances, resulting in extensive economic and human losses [1–3]. These accidents reflected that some RC frames exhibited insufficient load-bearing capacity and nonductile failure (e.g., shear failure) when some components were removed suddenly, although designed based on the current seismic codes [4,5]. Therefore, it is urgent to provide efficient and economic retrofitting techniques to improve the anti-progressive collapse capacity of RC frames.

Among various retrofitting techniques, the application of externally bonded carbon-fiber-reinforced polymer (CFRP) composites to retrofit RC frames has attracted much attention from both researchers and engineers due to some advantages including high strength, lightweight, convenience to be installed, etc. [6,7]. Zhang and Hsu [8] studied the shear strengthening of the RC beams using CFRP laminates and demonstrated the feasibility of the CFRP system. Wang and Li [9] investigated the effect of the initial load and load history on the ultimate strength of RC beams strengthened by CFRP laminates. The theoretical model that can predict the ultimate strength of retrofitted beams under sustaining loads was proposed. Garcia *et al.* [10] confirmed that CFRP retrofits can improve the seismic performance of a damaged two-story RC structure through experimental tests and numerical analysis. Mofidi and Chaallal. [11] investigated the performance of full-scale RC T-beams strengthened in shear with CFRP strips and sheets. It was proved that the contribution of transverse steel to the shear resistance of the frame cannot be affected by retrofitted CFRP. Esmaeel *et al.* [12] applied CFRP to retrofit damaged RC beam-column joints and revealed the complete jacketing retrofitting strategy can enhance the seismic performance. Ma *et al.* [13] tested the seismic performance of four full-scale interior subassemblies using CFRP retrofitting. The test results pointed out that the failure mode of retrofitted subassemblies varied with the ductile beam-hinge type, and the ductility

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and energy dissipation capacity can be enhanced. Adebayo and Kim [14] revealed the significant improvement in the mechanical performance of the CFRP-retrofitted recycled concrete and recommended CFRP for sustainable prefabricated concrete structures.

Despite several studies on the mechanical behavior of CFRP-retrofitted RC structures, limited studies reported the effects of advanced nano CFRP material on retrofitting RC structures. After adding nanomaterials into the CFRP adhesive matrix, the nano CFRP can be formed and it displays much better performances than that of traditional CFRP in many characteristics (*e.g.*, corrosion resistance, tensile strength, ultimate strain, and toughness), which makes it more appropriate to be used in many concrete infrastructures [15–17]. Irshidat *et al.* [18] investigated the effect of carbon nanotubes (CNTs) in improving the strengthening efficiency of CFRP-retrofitted RC beams experimentally. The test results showed that CNTs can improve the adhesion at the carbon fiber/epoxy interface leading to enhancement of resistance. Irshidat and AL-Saleh [19] found that CNTs can enhance the flexural strength of heat-damaged RC beams repaired with CFRP composites. In addition to the limited studies of the nano CFRP material in the retrofitting field, it should be noted that almost all of the past studies directed to nano CFRP strengthening of the structural component, neglecting the effects of the nano CFRP retrofitting technique on the performance of progressive collapse of the structure overall.

Considering the existing gap in knowledge, this study focused on investigating the performance of progressive collapse for the RC frame strengthened by nano CFRP. The two half-scale four-bay RC frame models including the control specimen and retrofitting specimen were cast and tested under the scene of eliminating the middle column. The mechanical behavior of the structure was subsequently observed and measured during the progressive collapse. Finally, the retrofitting effects were identified and discussed by comparing crack patterns, failure mode, load–displacement curves, horizontal displacement, strains, and rotational and energy dissipation capacity.

2 Experimental study

2.1 Specimen design

A three-story, four-bay prototype RC frame exhibiting unequal beam spans was designed according to the Chinese concrete design code GB 50010-2010 [20]. The

first floor represented a height of 3 m, whereas the higher floors represented a height of 4 m. Based on the similarity theory in which similarity indicates that the ratio of the physical quantities corresponding to the model and prototype remains constant, both the RC frame models were extracted from the prototype frame in Figure 1 and were subsequently scaled down to half. One frame, referred to as S1, was only with steel reinforcement, whereas the other frame, referred to as S2, consisted of steel reinforcement and U-shaped nano CFRP strengthening. The ground center column was removed to simulate the failure of the center ground column due to accidental load; however, the corresponding frame joint was observed to remain intact. The side-span beam was designed herein to simulate the horizontal constraints of the beams from the surrounding bay.

Figure 2 shows the geometric details of the two identical test specimens. The side- and middle-span beams exhibited lengths of 1.0 and 1.5 m, respectively. The side-span beam was set up to simulate the constraint with respect to the lateral movement of the internal columns. A 200 mm-long short column was necessary above the beams to place the loading apparatus above the specimens. Thus, the height of the columns was 1,700 mm. These dimensions were along the centerlines of the respective frame components. This is an important factor because the beam depths and column widths mean that the final clear spans will be slightly less than those shown in Figure 2. All the beams had cross-sections with a width of 110 mm and a

Figure 1: Relationship of the test specimen with a prototype.

Figure 2: Concrete dimensions and steel reinforcements of S1 and S2.

height of 200 mm, whereas all the columns had a 200 mm square cross-section. To simulate the boundary conditions, the model frames were developed on a 500 mm-high foundation beam that was fixed using a bolt to the strong reaction floor through a preformed hole, as shown in Figure 2. Ten 100 mm-diameter holes were performed during casting.

For the beams, the transverse reinforcement spacing for the whole span was 160 mm. All the beams had stirrup diameters of 6 mm. About 135° end hooks were set for the stirrups to satisfy the detailed requirements according to the Chinese concrete design code GB 50010-2010 [20]. A sufficient number of stirrups were used in the columns to prevent shear failure preceding the flexural failure in S1 and S2. The transverse reinforcement of the columns was achieved using 8 mm stirrups with 135° end hooks for a spacing of 160 mm. However, the stirrup spacing was reduced to 80 mm at 0.2 m high regions from the corresponding joint interfaces to avoid potential shear damage of the testing zone and to reinforce the column support and loading regions. Furthermore, the S1 and S2 specimens exhibited a longitudinal reinforcement ratio of 0.46 and 0.81% in the beam bottom and the beam top, respectively, along with a longitudinal reinforcement ratio of 1.57% in each column. These reinforcement percentages were obtained according to the gross areas of the concrete sections. For the beams, two 8 mm-diameter bars and one 10 mm-diameter longitudinal bar were used as the top bars, whereas one 10 mm-diameter bar was cut off at 0.25 times of the beam span from the beam end. The longitudinal steel reinforcement of the columns was determined to avoid flexural yield even though some cracking is expected to have occurred. For all the columns, the diameter of the longitudinal bars was 8 mm. In addition, the HPR300 and HRB400 steel bars were

considered to be the stirrups and longitudinal reinforcement, respectively. Note that the HPB300 steel bar type denotes a hot-rolled plain steel bar with a characteristic yield strength of 300 MPa; HRB400 denotes a hot-rolled ribbed steel bar with a characteristic yield strength of 400 MPa. Furthermore, the specified clear cover depth was 10 mm for both models.

2.2 Material properties

Concrete was supplied by a local ready-mix company. Both specimens were produced from the same concrete batch. It was subjected to a 4 week curing period in the test facility with respect to loading failures. Both the specimens had the same concrete grade and rebar specification. Three cubic concrete samples having a dimension of 150 mm $\times$ 150 mm $\times$ 150 mm and six prism concrete samples having a dimension of 150 mm $\times$ 150 mm $\times$ 300 mm were considered for the material testing conducted in accordance with the Chinese standard GB/T 50081-2002 [21]. The 28 day compressive strength of the cubic concrete samples was averaged, which yielded 31.9 MPa. On average, the concrete elastic modulus and the Poisson ratio of the prism concrete samples were 34,500 N/mm² and 0.24, respectively. Three samples with a length of 500 mm for each different specification of steel bars were tested according to the Chinese standard GB/T 228.1-2010 [22]. Table 1 summarizes the displacement control scheme of the MTS actuator in experiment. After divided into three stages, the loading increased stepwise until the complete failure of the frame was achieved. Furthermore, Tables 2 and 3 summarize the mechanical properties of the nano CFRP and epoxy resins, respectively, as indicated by the manufacturer.

Table 1: Displacement control of the MTS actuator

	First stage	Second stage	Final stage
Vertical displacement of the failure column (mm)	0–25	25–85	85–failure
Loading rate (mm/min)	1	4	10
Each step (mm)	5	10	15

2.3 Strengthening schemes

S2 was strengthened using the layered U-shaped nano CFRP sheets. Considering the length of the potential plastic hinge, the strengthening length of the beam end was 350 mm; considering that the stirrups have certain spacing, the strengthening scheme with intermittent spacing is adopted; therefore, this area was strengthened using two identical 150 mm-wide U-shaped nano CFRP sheets having a center-to-center spacing of 200 mm. Considering that the stirrups have certain spacing and to simulate this characteristic as far as possible, a non-strengthened area of 50 mm clear spacing was reserved. Figure 3 illustrates the S2 model. The nano CFRP strengthening process can be given as follows.

- 1) The area of the specimen in which the continuous nano CFRP sheet was to be glued was sandblasted to eliminate any surface cement paste and round off the beam edges. Here, the bond area was ground to eliminate any possible irregularities and achieve a desirable smooth bond surface; furthermore, the residues were eliminated using compressed air.
- 2) The concrete surface was saturated with epoxy, a two-component adhesive of a resin, and a hardener primarily engineered for structural applications and directly supplied by the manufacturer.

3) Once the resin became tacky, the nano CFRP sheet was epoxy-bonded to the beam surface. Each wrap was continuous at the bottom face of the beam and terminated close to the top of the beam on both faces.

4) Finally, the nano CFRP sheets were saturated with epoxy.

2.4 Test setup and instrumentation

For the tests, a disproportionate collapse situation was simulated by eliminating the center support column of each frame. Figure 4 shows a schematic representation and a general view of the test setup. The material testing system (MTS) actuator placed at the top of the failure column was used to load the frames *via* displacement control. The mechanical properties of steel bars are given in Table 4. Meanwhile, a load cell with a 1,000 kN measuring range and installed under the actuator was used to measure the applied vertical load. In addition, Figure 4 displays two collars that were designed to prevent unwanted failures resulting from any out-of-plane deformation of the testing frame. In particular, the collars were manufactured using two steel plates tightly bolted together in opposite directions out of the beam plane and were subsequently bolted to the lateral steel braces. About 5 mm gaps were intentionally left between the collars and the beam surfaces in the initial loading stage to avoid unwanted frictional forces from acting on the columns in the early stages of the tests.

Four horizontal linear variable differential transformers (LVDTs) were placed at each joint of the specimens to measure the horizontal movements of the columns for

Table 2: Properties of nano CFRP

Material	Weight (g/mm ²)	Thickness (mm)	Tensile strength (MPa)	Elasticity modulus (GPa)	Failure strain (%)
Nano CFRP	300	0.167	4,340	240	1.7

Table 3: Properties of epoxy

Material	Compressive strength (MPa)	Flexural strength (MPa)	Tensile strength (MPa)	Elasticity modulus (MPa)	Failure strain (%)	Bond strength (MPa)
Epoxy	81	97	68	2,870	2.9	4

determining the axial stiffness values of the horizontal restraints for the test specimen. Figure 5 shows the LVDT layout. Accordingly, the deformation information of the longitudinal reinforcements is the key to analyze the collapse mechanism of the frame; thus, strain gauges, whose layout is presented in Figure 5, were used to measure the strains of the beam rebars. The reinforcement strains at the potential plastic hinge region were measured. Furthermore, all sensors including strain gauges, load cells, and LVDTs were connected with a strain collection system that was activated for data storage and operated at a frequency of 5 s each time. The beam surface was brushed with lime and divided using a dense mesh to monitor the crack development at all the locations throughout the testing period. After each loading increment, a few seconds were allotted for the

instrumentation to record stable data and to observe new cracks on the specimens.

3 Experimental results and discussion

3.1 Crack patterns and failure mode

Figure 6 denotes the failure mode of S1. The crack patterns of the beams at both sides of the middle joint were symmetrical, except at the local regions near the joint. Because of the lack of rotational restraint in the middle joint, the middle joint can be rotated to one side with a

Figure 3: The S2 model with nano CFRP: (a) schematic diagram and (b) photograph.

Figure 4: Typical test setup.

wide crack, resulting in a fracture at the bottom bars at or near the joint interface on that side. The cracks were considered severe and extensive at three regions, *i.e.*, near the joint interface, at the beam ends, and between the curtailment points of the top bars. At the beam end, the crack that appeared at the top of the beam and developed toward the belly along the beam height direction conformed to the mode of flexural crack. As the loading increased and cracks began to develop, plastic hinges would develop at the curtailment point of the top reinforcement in the beam because of the discontinuity in longitudinal reinforcement, as depicted in Figure 6(a). Consequently, the hinge significantly enhanced the deformation

capacity of S1. In the compressive arch action stage, the reinforcement in the beam bottom of the middle joint fractured, as shown in Figure 6(b). Final failure was induced by the fracture of the beam top reinforcement in the plastic hinge region, as clearly illustrated in Figure 6(c). Concrete crushing was observed; however, it was not obvious at the bottom of the beam end.

Figure 7 denotes the failure mode of S2. As the loading increased, the plastic hinge began to develop at the curtailment point of the beam top reinforcement, mainly because of discontinuity with respect to the longitudinal reinforcement. In the compressive arch action stage, the reinforcement in the beam bottom of the middle joint fractured, as shown in Figure 7(a). In contrast to the scene in S1, the top bar of the beam end of S2 yielded and fractured at the compressive arch action stage after the beam bottom reinforcement of the middle joint was fractured, as illustrated in Figure 7(b).

3.2 Load–displacement histories

The tests were terminated once the fracture of the top reinforcement was achieved at one of the beam ends. Prior to failure, the specimens experienced large deformations and rotations. The beam surfaces were not

Table 4: Properties of the steel bars (average values)

Material	Diameter (mm)	Bar type	Elastic modulus (GPa)	Yield strength (MPa)	Ultimate tensile strength (MPa)	Fracture strain (%)
Longitudinal reinforcement	8	Ribbed	201	462	557	12
	10	Ribbed	207	452	534	14
Stirrup	6	Plain	216	345	433	19
	8	Plain	212	357	429	18

Figure 5: Layout of LVDTs and steel strain gauges.

Figure 6: Crack pattern and failure mode of S1: (a) at the beam end near column D; (b) at the middle point; and (c) the failure mode of S1.

observed to contact the collars of braces. Consequently, at the end of the tests, the middle joint displacements became 350.1 mm (23.3% of the single-span length) for S1 and 206.5 mm (13.8% of the single-span length) for S2. Herein, the experimental results and observations of the two specimens were demonstrated based on the load–displacement histories, horizontal displacement variation, strain variation, crack patterns and failure mode, rotational capacity, and energy dissipation capacity. Furthermore, for model S1, the progressive collapse process was classified under three primary stages, *i.e.*, flexural action, compressive arch action, and catenary action; for S2, the process could

Figure 7: Crack pattern and failure mode of S2: (a) at the middle joint and (b) at the beam end near column A.

only be classified into two primary stages, *i.e.*, flexural action and compressive arch action.

Figure 8 denotes section OA, which refers to the stage characterized *via* flexural action. Flexural action would develop until all the plastic hinges are present at the middle joint interfaces and the beam ends [23]. Section AB could be considered to be the compressive stage, where the compressive arch action contributes to the resistance. Section BC is the catenary stage, which denotes the transformation of the strain in the bars in the original compressive region of the beams from compression to tension. It is also noteworthy that around the starting point of the catenary stage (point B), the surrounding columns would change their movements from outward to inward and then back to their initial positions (with a horizontal displacement of 0 mm). This stage begins when the vertical displacement of the failure column is 222.3 mm for S1. However, the material test for S2 ends in the stage of compressive arch action. The bond failure of the nano CFRP sheets did not occur during testing *via* visual observations.

The overall structural behavior of each RC frame was characterized based on the resistance force *versus* vertical

Figure 8: Resistance force versus the vertical displacement of the middle column of the frames.

displacement history of the middle joint. Beam deflection was represented by the vertical displacement of the middle joint, which was obtained from the actuator in-built displacement transducer. Figure 8 shows the resistance force versus vertical displacement histories of S1 and S2.

In the case of S1, the middle joint region and beam ends were initially subjected to positive and negative bending moments, respectively. Hence, bottom reinforcement rapidly achieved yield at the middle joint. Afterward, the top reinforcement yields at both the beam end indicated that the plastic hinges have formed and that the flexural action has attained its capacity. In the case of a small middle joint vertical displacement, the beam ends attempted to push the two columns outward as the beam deflected downward. Regardless, such action was prevented by the horizontal restraints at the column side, which generated the axial compressive force in the beam. Due to the compressive force in the beam, load resistance was enhanced to become greater than the flexural capacity.

When the vertical displacement was 26.7 mm, the resistance force achieved its maximum value of 38.4 kN in the compressive stage and subsequently decreased to 12.7 kN until the end of the compressive stage (with a vertical displacement of 222.3 mm). Here, the sudden reduction in the applied load at the middle joint was caused by the fracture of the bottom rebars at the middle joint interfaces, and the corresponding displacement was 103.7 mm. Specifically, the crushing of concrete and fracture of rebars allowed the beam ends to rotate and pull in the columns. As the vertical displacement for the middle joint widened, the beam axial force gradually switched from compression to tension. Consequently, a catenary action could be observed when the beam was in pure

tension at a middle joint vertical displacement of 222.3 mm. Due to the development of catenary action in the damaged structure, the resistance force further increased in the catenary stage until the end of the test. The catenary action completely utilized the reserve strength of steel reinforcement. Furthermore, the ultimate capacity of catenary action was determined solely based on the top reinforcement because of the fracture of the bottom reinforcement near the joint interfaces during the development of catenary action. The test ended when the center column attained a vertical displacement of 350.1 mm as fracture developed at the top rebar at the beam end. The catenary action recorded a maximum resistance force of 30.5 kN, which was 0.79 times that of the compressive arch action.

In the case of S2, the vertical displacement of 24.0 mm was assigned to the end of the flexural action stage because of which the vertical displacement corresponded to the formation of plastic hinges in the beams. Furthermore, when the vertical displacement was 82.7 mm, the resistance force achieved its maximum value of 45.5 kN and then gradually decreased to 42.9 kN until a vertical displacement of 109.4 mm was reached. Beyond this point, the resistance force decreased several times due to the rebar fractures. The test ended in the stage of compressive arch action for a vertical displacement of 206.5 mm.

After the comparison between S1 and S2, the scheme of strengthening at the beam end using the U-shaped nano CFRP may result in higher rotational stiffness, limited deformation, and present a smaller rotation at failure. It is not conducive to the development of a catenary resistance mechanism. However, this strengthening scheme improves the compression failure ability at the beam end, resulting in an increase of the maximum resistance force at the compressive stage by 18%.

3.3 Horizontal displacement variation

Figure 9 denotes the horizontal displacements of the joints at the adjacent and external columns (Figure 5, locations of LVDTs) versus the vertical displacements of the center column of S1 and S2. In the case of S1, the adjacent and external joints initially moved outward (*i.e.*, horizontally away from the center column) and achieved maximum values of approximately 4.4 mm at a vertical displacement of 42.7 mm when the vertical displacement increased in the flexural action and compressive stages. Subsequently, these joints moved to their initial positions (with a horizontal displacement of 0 mm) at a

Figure 9: Horizontal displacements of the joints versus the vertical displacement of the middle column for S1 and S2.

vertical displacement of approximately 222.3 mm. Further, the joints continued to move inward until the end of the test in the catenary stage, with a horizontal displacement of approximately 20.8 mm inward.

The response for S1 was similar to that for S2. With an increase in the vertical displacement, the adjacent and external joints initially moved outward and achieved their maximum value of approximately 3.9 mm at a vertical displacement of 95.9 mm. Next, the joints exhibited a tendency to move to their initial positions. At a vertical displacement of approximately 215.5 mm, the test ended in the compressive stage with a horizontal displacement of approximately 1.3 mm outward.

Concerning the mechanism, the compressive arch action was activated and contributed to the resistance force in the compressive stage. Obviously, the length of the line connecting the two diagonal corners of the beam exceeded the horizontal length of the beam in the centroidal axis. When the center column was pushed downward, the originally inclined diagonal line (pressure line) moved gradually into a horizontal position, as shown in Figure 10(a). Consequently, the adjacent and external joints would be pushed outward, triggering the compressive arch action. This explains the gradual increase in the resistance force as the vertical displacements increased during the flexural action stage. Furthermore, the beams remain in compression, with

Figure 10: Schematic of the beneficial and adverse compressive arch actions to the resistance force: (a) beneficial action before vertical displacements of 26.7 and 82.7 mm for S1 and S2, respectively and (b) adverse action owing to the vertical displacements of 222.3 and 206.5 mm for S1 and S2, respectively.

the vertical displacements of 26.7 and 82.7 mm for S1 and S2, respectively.

The directions of the vertical components of the compressive forces in the beams began to shift from pointing upward to downward, imposing an additional load on the center column, which implied a transition from the beneficial compressive arch action to the adverse catenary action, as illustrated in Figure 10(b). These observations explain the gradual decrease in resistance force as the vertical displacements increased to 222.3 and 206.5 mm for S1 and S2, respectively. In this stage, the frame experiences a large deformation majorly, and the joints would be tensioned inward.

3.4 Strain variation

To assess the effects of the retrofitting strategy on rebars, local strains were monitored using foil-type strain gauges fixed on the rebars. Figure 11 shows the strains of the rebars in beam BC (Figure 5) varying with the vertical displacement of eliminating the middle column. It should be noted that the strain values given are the mean value of all results in the specific zone. In flexural action, the strains of S1 and S2 showed a similar trend. In detail, the strains of rebars at the top left end of the beam were positive, and increased almost linearly to the yield strain (up to $2,300 \mu\epsilon$) with vertical displacement increasing. Conversely, the strains of rebars at the top right end of the beam were negative and reduced gradually to about $2,000 \mu\epsilon$ with the increase of vertical displacement. These phenomena indicated the elastic behavior of rebars during the flexural action.

Figure 11: Strains of the rebars in beam BC for S1 and S2.

In compressive arch action, the strains at the top left end of rebars continued to increase and developed in the hardening stage. The strain of S2 can reach its peak (up to $8,500 \mu\epsilon$) with the vertical displacement increasing to 75.2 mm. However, the largest strain of S1 was almost $12,000 \mu\epsilon$ when the vertical displacement increased to 49.5 mm. These results reflected that the nano CFRP can anticipate efficiently in the loading process with stress redistribution, and consequently, reduce the stress of rebars and enlarge the deformability. Similar phenomena can be observed at the top right end of rebars. The strains of S2 were obviously less than S1 due to the anticipation of nano CFRP.

In catenary action, the strain at the top right end of rebars in the S1 case changed from a negative value to a positive value, indicating the tension action in the top region. Due to the fracture of rebars and nano CFRP in the section location, S2 lost the resistance capability in the final compressive arch action and failed to experience catenary action without strains results. This is consistent with the relationship between the resistance force and vertical displacement shown in Figure 8, which proves the lack of anti-progressive collapse capacity of RC frames after the retrofit.

3.5 Rotational and energy dissipation capacity

The rotational capacities of the beams were evaluated via chord rotations. The chord rotation of the beams was calculated as the ratio of the middle joint vertical displacement with respect to the length of the single-span beam (1,500 mm) as defined in UFC 4-023-03 (2013) [24]. Thus, the chord rotations of S1 and S2 at the final middle joint displacement were 0.23 and 0.14 rad, respectively. This indicates that the rotational capacity of the frame strengthened using nano CFRP at the beam end was lower than that of a bare frame. This can be due to the fact that the plastic hinges in joint regions in S2 develop insufficiently due to the limitation of nano CFRP. In that case, the crack appears majorly in the section, rather than the joint region.

Accordingly, the energy dissipation capacities of S1 and S2 were represented by the area enclosed by the resistance force–vertical displacement curve and x-axis, respectively, as depicted in the schematic shown in Figure 12. Here, the areas of S1 and S2 were 7985.1 and 5505.2 N m, respectively. Subsequently, the energy dissipation capacity of S2 could be easily obtained as 0.69 times that of S1, indicating

Figure 12: Schematic of the energy dissipation capacity of the specimen: (a) S1 and (b) S2.

that S2 failed to improve the dissipation capacity of progressive collapse even though it was strengthened using the

U-shaped nano CFRP, resulting in lower ductility than that obtained in case of S1. Therefore, the frame strengthened at the beam end using the U-shaped nano CFRP limits the rotational capacity, which implies that in the stage of flexural action and compressive arch action, S2 requires greater loading compared to S1 if the same vertical displacement must be achieved. Sequentially, it causes energy dissipation capacity to focus on the stage before the catenary and also adversely affects the development of the catenary.

3.6 Discussion

Prior to the final failure, three loading stages (flexural action, compressive arch action, and catenary action) were identified to describe the progressive collapse process of S1, whereas only two loading stages (flexural action and compressive arch action) were identified for S2. Figure 8 describes that S1 and S2 exhibited almost similar resistance performances during the flexural action stage. In the subsequent compressive arch stage, the maximum resistance forces of S2 can be improved by 18.5% after nano CFRP retrofitting, increasing from 38.4 kN (S1 case) to 45.5 kN (S2 case). Correspondingly, the vertical displacement of eliminating the middle column increased by 3.09 times, increasing from 26.7 to 82.7 mm, indicating the significant improvement of deformation capability, as listed in Table 5. For the last catenary stage, S1 developed the plastic hinge continuously with more extension of rebars and deformability as shown in Figure 6. Unfortunately, S2 failed to experience this stage due to the fact that the nano CFRP can hinder the crack spreading of concrete and hardening of rebars, and consequently, the fractures of rebars and nano CFRP occurred in the normal section without plastic hinge developing sufficiently, as shown in Figure 7.

It can be concluded that the retrofitting strategy using U-shaped nano CFRP strengthening the beam ends can improve its mechanical properties in terms of strength and ductility, and postpone shear failure of the joint core

Table 5: Comparison of the test results

Specimen ID	Maximum resistance force at the compressive stage (kN)	Vertical displacement at the first bar fracture (mm)	Initial vertical displacement at the catenary stage (mm)	Maximum vertical displacement (mm)
S1	38.4	103.7	222.3	350.1
S2	45.5	109.4	—	206.5
Δ (%)	18.5	5.5	—	-41.0

Note: $\Delta = (S2 - S1)/S1 \times 100\%$.

region. Due to the high performance and ease of installation, the nano CFRP material has much potential in retrofitting structures, particularly beam component and beam-column subassemblies. However, this strategy has its own defects in anti-progressive collapse, which should be considered in retrofitting design and implementation.

4 Conclusion

This article investigated the effect of nano CFRP on improving the strength and ductility of RC frames by means of retrofitting the beam component. Two four-bay RC frame specimens, among which one was strengthened using U-shaped nano CFRP at the beam end and the other was not strengthened, were tested to understand the mechanical behavior of progressive collapse. The retrofitting effects were studied and identified through crack patterns, failure mode, load–displacement curves, horizontal displacement, strains, and rotational and energy dissipation capacity. The major conclusions were the following:

- 1) The frame S2 strengthened using U-shaped nano CFRP at the beam end experienced the same ductile beam-hinge failure mode as S1. However, the mechanical characteristics of S2 have been improved significantly, namely 18.5% in load-bearing capacity and 3.09 times in ductility compared to S1, which demonstrated the excellent retrofitting effects of nano CFRP.
- 2) S2 exhibited a much smaller response in strains of rebars and maximum horizontal displacements for beam-column subassemblies than S1, which reflected the efficient anticipation of nano CFRP in load bearing and deformability.
- 3) During the progressive collapse process, S2 only experienced the flexural action and compressive arch action and failed to enter the catenary action. Therefore, this strategy retrofitting beam end has its own defects in anti-progressive collapse, which should be considered in retrofitting design.

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