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Test and Analysis on Anti-seismic Property of FRP-reinforced Non-ductile Reinforced Concrete Frame Structures

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The research is conducted for experimental analysis on anti-seismic property of FRP-reinforced non-ductile reinforced concrete frame structures. Research and analysis on the anti-seismic property of the structural model are conducted with the shaking table experiment and the simulation analysis. Based on the research findings, the experimental results of the FRP-reinforced non-ductile reinforced concrete frame structure in this research are in accordance with design requirements, contributing to reducing structural frame failures and displacements. Further researches by professionals are still needed for the anti-seismic property of the FRP-reinforced non-ductile reinforced concrete frame structure model in the future.

1. Introduction

The research on anti-seismic property of FRP-reinforced non-ductile reinforced concrete frame structure is conducted in this article, to analyze the reinforcement effect of fiber-reinforced composite materials. Firstly, it is needed to conduct model design for non-ductile reinforced concrete frame structures and the reinforcement measurement processing scheme analysis. Secondly, anti-seismic property is analyzed based on experiments, to judge the feasibility of the method in the research.

2. Literature review

The reinforcement almost does not produce additional load on the structure and does not affect the structure appearance and use space. Therefore, compared with the traditional reinforcement methods, the FRP reinforcement method is a more economical and more potential strengthening method in most cases. At present, the practical engineering application and scientific research mainly use FRP to reinforce concrete members as the main goal. Each country has promulgated and implemented the technical regulations of the FRP reinforced concrete structure, which is mainly aimed at the components, but the concrete changes of the seismic performance of the whole structure after the reinforcement of the key components are still lacking enough cognition, and the deficiency in this respect is insufficient. It may lead to the high evaluation of the seismic performance of the reinforced structure, because the improvement of the bearing capacity and the ductility of the structure cannot guarantee the improvement of the overall seismic performance of the structure, and there may be the risk of the structural failure mode change resulting from the transfer of the structural weak layer to the unreinforced area. Therefore, to study the overall seismic performance of the Non-ductile Reinforced concrete frame structure before and after the FRP reinforcement, and to realize the seismic reinforcement design method for the whole structure, that is to solve the problem of determining the reinforcement position and the amount of reinforcement when the reinforced concrete frame structure is reinforced by FRP, and the push FRP used in the field of seismic reinforcement concrete structure is the key to sustainable development.

The most important component of the reinforced concrete frame structure is the column, so many researches have been carried out on the seismic behaviour of reinforced concrete columns strengthened by FRP, but the research objects are mostly aimed at the piers with relatively small axial pressure, and the research on the seismic reinforcement of the high axial compression ratio columns in the frame structure is not enough. Cecconi and others test Non-ductile Reinforced concrete columns before and after CFRP reinforcement by

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pseudo static test. The effect of FRP constraint ratio and additional CFRP anchors on the ultimate displacement angle of reinforced columns is investigated. It is found that the ultimate displacement angle of the reinforced column increases with the increase of the FRP constraint ratio, while the additional CFRP anchor and the outsourced CFRP cloth are simultaneously strengthened. A simplified formula for calculating the displacement angle of a Non-ductile Reinforced Concrete Rectangular Column based on FRP seismic strengthening is proposed (Cecconi et al., 2017). Stockdale and Milani carried out the pseudo static test of reinforced concrete square columns with low concrete strength and weak confinement before and after FRP reinforcement. It is found that the unreinforced columns have been damaged earlier because of the smaller buckling or the length of the longitudinal reinforcement, and the seismic performance of the columns strengthened by FRP is obviously improved, especially the ductility of the columns. However, the reinforcement effect is not good for the specimens with insufficient length of longitudinal reinforcement (Stockdale and Milani, 2017). Moughty and Casas conducted the pseudo static test of the FRP reinforcement column the parameters of the axial compression ratio, the section shape of the reinforced column and the existence of the FRP reinforcement, and the improvement efficiency of the seismic performance of the column is evaluated based on the damage performance index FRP. Finally, the seismic behaviour of the FRP strengthening column considering the section shape and evaluation index the properties of the FRP material is put forward at the end of the Moughty and Casas (Moughty and Casas, 2017). Based on the section analysis of the limit state of reinforced concrete, Gholampour and Ozbakkaloglu proposed the calculation method of bending and shear bearing capacity of CFRP reinforced concrete column, but the method cannot be analysed in the whole process, which is mainly used to simplify the design (Gholampour and Ozbakkaloglu, 2017). The load displacement hysteretic curves of reinforced concrete columns reinforced by CFRP are calculated by Sung for the first time by strip method, but the stirrup constraint concrete hysteretic constitutive law is used to replace the CFRP confined concrete in the calculation of the reinforced concrete columns strengthened by the strip method (Sung, 2017). Based on the Open Sees software, Yang and Wang simulate the effect of the longitudinal bar slip on the seismic performance of the reinforced column by adding the zero-length element at the end of the nonlinear beam column element, and it is found that the effect is quite small (Yang and Wang, 2017). Drygala and others use the damage model to simulate the hysteretic behaviour of reinforced concrete columns and square columns strengthened by CFRP and can only calculate the rise section of the load displacement curve and cannot simulate the descending section (Drygala et al., 2017). Based on the study of the stress-strain relationship of CFRP confined concrete columns under repeated compression, a nonlinear finite element simulation analysis method for reinforced concrete columns constrained by CFRP is proposed by Shaw and Andrawes. This model divides the reinforced columns into CFRP shell elements and concrete solid elements and assumes that there is no bond slip between the CFRP and the coagulant soil units, and at the same time it ignores the effect of long term load on concrete (Shaw and Andrawes, 2017).

3. Methods

3.1 Model design and manufacturing

The model structure experiment is conducted so as to deduce the performance and reaction of the prototype structure based on the model experiment results; therefore, it is needed to determine the prototype structure to be researched before model design and manufacturing.

Figure 1: The frame model after CFRP reinforcement

The article researches aiming at the reinforced concrete frame structure widely applied in our country and the FRP-reinforced anti-seismic property. The prototype structure is a 4-layer reinforced concrete frame, with the height of the first layer of 3.6m and that of the rest layers of 3.0m; the column cross-section size is 400mm×400mm, and the beam cross-section is 250mm×500mm, and the thickness of the floor is 120mm. The seismic fortification intensity is 8 degree, and the basic designed seismic acceleration is 0.2g. The classification is Type Three, and the designed seismic group is Group 1, and the field type is Type II. (Figure 1 shows the frame model after CFRP reinforcement.)

The concrete material pouring is adopted directly considering of the large size of the model. The maximum size of the concrete aggregate is shrunk as per the size ratio of similitude; the concrete strength grade is the same as that of the prototype as C30; the HRB400 grade is adopted as the stress longitudinal bar of the beam column; HPB235 grade is adopted as the stirrup in the beam column and the stress reinforcement in the board. Table 1 shows the property indexes of concrete and steel bars.

Table 1: Properties of concrete and steel bar measured materials

material	Diameter/thickness (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Elastic modulus (MPa)
Longitudinal reinforcement	12	535	615	200000
stirrup	6	412	816	2000000
CFRP	0.167	-	4340	240000
concrete	-	20.3	-	-

3.2 Reinforcement scheme

Firstly, the shake table test of unreinforced reinforced concrete frame model is conducted, so as to acquire the failure mode and the failure position of the unreinforced model. According to the comparison model test results, the weak position corresponding to the second model is reinforced with the wrapping CFRP. According to the test results of the unreinforced model, the failure positions concentrate on the plastic hinge area on the beam column end on the lower two layers as well as the joint core areas; therefore, the reinforcement positions for the reinforcement model are the plastic hinge area on the beam column end on the lower two layers as well as the joint core areas. It is necessary to chamfer to the beam column before reinforcement, with the chamfering radius of about 20mm (0.1 times of that of the length of side of the column cross-section). The column members are reinforced in a way of horizontal wrapping of 2 layers of CFRP; the reinforcement region is within 250mm around the column end (1.25 times of the length of side of the column cross-section); the beam is reinforced for anti-bending and anti-shear capacity in a way of longitudinal pasting of 1 layer of CFRP and horizontal wrapping of 1 layer of U-shape hoop. The reinforcement region is within 450mm around the beam end (1.8 times of the height of the beam cross-section). (Figure 2 shows the CFRP reinforcement scheme.)

Figure 2: CFRP reinforcement scheme

3.3 Measurement scheme

The shaking table test conducted in this article mainly measures accelerated speed and displacement of various layers of the main measurement model, vertical and horizontal dynamic strain of concrete of ground floor column end of unreinforced frame and joint sections, vertical and horizontal dynamic stains of CFRP on corresponding positions of the reinforced frame and the unreinforced frame, and dynamic strains of vertical bars on various layers of column ends. Therefore three types of input methods for seismic oscillation during test, i.e., single direction and simultaneous input along with two horizontal directions (X and Y). Considering of the asymmetry on stress along with the first-cross direction (Y direction) of the model, single-direction accelerometer and displacement meter are arranged on the two directions of various layers. Three accelerometers are arranged on each layer (1 on the X direction and 2 on the Y direction). Five displacement meters are arranged on each layer, including 3 LVDT ones and 2 stay wire ones. According to the calculated natural period of vibration of the prototype structure, the site conditions (with characteristic site period of Tg=0.35s) and shaking table displacement control parameters, three natural seismic oscillation records including El Centro, Kobe and Northridge are finally selected. In this test, the seismic oscillation records are inputted along with two horizontal directions (X and Y) in a single-way and dual-way in this article. Therefore, the seismic oscillation records on two orthogonal directions are adopted by Kobe and Northridge. The seismic oscillation record duration during the test is compressed as per the time ratio of similitude. The time interval of the selected three seismic oscillation records is 0.02s, and the time interval after compression is 0.01s. The seismic oscillation peak acceleration during the test is gradually increased. The three original seismic oscillation peak accelerations are conducted with normalization processing.

4. Results and discussion

4.1 Dynamic characteristics of the mode.

Spectral analysis can be conducted on measured accelerated speed data of the white noise excitation before and after the seismic oscillation actions of different levels, to acquire the natural vibration frequency and period of the model structure along with X direction and Y direction. Table 2 shows the specific values of the frequency and the period of the first two orders along with the two directions.

Table 2: Model self-oscillation frequency and cycle

model	The earthquake levels	X direction (strong axis)				Y direction (weak axis)			
		The first order		The second order		The first order		The second order	
		Frequency (Hz)	Cycle (s)	Frequency (Hz)	Cycle (s)	Frequency (Hz)	Cycle (s)	Frequency (Hz)	Cycle (s)
CFRP reinforcement mode	Before the earthquake	3.412	0.293	10.944	0.091	3.113	0.321	4.596	0.218
	After 0.7 g one-way	1.355	0.738	5.328	0.188	1.154	0.867	2.063	0.485
	After 0.7 g two-way	1.154	0.867	4.596	0.218	1.154	0.867	2.063	0.485
	After 1.0 g one-way	1.147	0.871	4.590	0.218	1.147	0.871	2.063	0.485
Compare the model	Before the earthquake	3.428	0.292	10.954	0.091	3.121	0.320	4.770	0.210
	After 0.3 g one-way	1.410	0.709	12.335	0.081	0.909	1.100	1.410	0.709
	After 0.6 g two-way	1.154	0.867	10.809	0.093	0.757	1.321	1.154	0.867

According to the table, CFRP reinforcement model has similar initial natural vibration frequency and period with the unreinforced model, indicating that the wrapping CFRP reinforcement method leads to almost no influence on the primary rigidity of the structure. However, under different levels of seismic effects, there are large differences on changing rules of natural vibration frequency and period. With respect to CFRP reinforcement model, during the working condition 1 to working condition 30; i.e., after the single-direction accelerated speed effect level from 0.15g to 0.7g (equivalent to 8 degree of frequent occurrence earthquake to 8 degree of rare occurrence earthquake), the natural vibration frequency on the two directions is significantly reduced, in which the first order frequency on the two directions is reduced to 39.7% and 37.1% of the primary frequency, respectively, indicating certain degree of failure to the structure, and the model is in the elastic-plastic state. After the model experiences the dual-direction seismic oscillation effect with the targeted accelerated speed peak value of 0.7g (working condition 41), the frequency on X direction is slightly reduced, and no change is detected on the natural vibration frequency on Y direction. It indicates that the X direction gives rise to minor failure, which is further intensified due to the Y direction failure. When the model experiences the single direction seismic effect with the targeted accelerated speed range of 1.0g again (working condition 54), the natural vibration frequency of the model is slightly reduced, and the first order frequency is reduced by 0.6%, indicating slight new failure of the new model, with almost stable natural vibration frequency.

With respect to the unreinforced comparison model, after experiencing the 0.4g of Y direction and dual-direction of (0.25g on the X direction and 0.3g on the Y direction) standard seismic effect (working condition 15), the natural vibration frequencies of the model on the two directions are greatly reduced, and the first order frequencies on the X direction and the Y direction are reduced from 3.428Hz and 3.121Hz to 1.410Hz and 0.909Hz, respectively, which are only 41.1% and 29.1% of the primary frequencies. It indicates that there are serious failures on the structure, and the model is in the elastic-plastic stage. When the model experiences the accelerated speed standard seismic effect of 0.6g, the natural vibration frequencies on the two directions further reduce, and the natural vibration period is further increased. At the same time, considering that model Y has serious failure on bearing of seismic effect, the first order frequency on the Y direction after earthquake is reduced to 0.757Hz, which is only 24.3% of the initial frequency. The fundamental frequency on the X direction is reduced to 1.154Hz, which is 33.7% of the initial frequency. It indicates that the seismic effect gives rise to further damage to the structure, and the model is further damaged.

4.2 Maximum displacement of buildings

The relative displacement time-history of various structural layers can be acquired by subtracting the displacement time-history of corresponding table-board from measured displacement time-history. Table 3 shows the relative displacement amplitude of 1 and 2 layers under the action of Y in some working conditions.

Table 3: The relative displacement amplitude of 1 and 2 layers under the action of Y in some working conditions

Working condition	1 layer			2 layer		
	YL(mm)	YM(mm)	YR(mm)	YL(mm)	YM(mm)	YR(mm)
10	7.74	4.65	4.08	11.50	8.39	6.60
11	8.68	4.61	4.72	10.43	9.13	8.39
12	10.26	8.36	8.65	17.96	16.11	14.15
13	11.37	8.66	7.96	17.94	15.28	15.43
24	8.68	9.34	10.12	15.04	16.75	17.79
25	7.00	9.15	10.88	14.82	16.67	19.68
26	14.11	13.04	14.67	24.02	25.25	26.30

4.3 Strain effect

The CFRP strains of longitudinal bars of columns, the column end of the bottom reinforcement area and the joint area are measured in this article. The changes of reinforcing bar strain can be adopted to adjust the stress state of the structure. However, the reinforcing bar strain time-history acquired in the test is not favorable enough considering of the complicated measurement equipment and working conditions of the strain gage. Based on reinforcing bar strain measurement results, the maximum strain values of longitudinal bars of various layers are 1284με, 1405με and 8453με, respectively under the seismic levels of 0.3g, 0.7g and 1.0g, indicating that the reinforcing bar is in the yielding stage under relatively large seismic effect. Strain gages are pasted in the joint core are along with the horizontal direction, the longitudinal direction and the 45° direction, so as to acquire the horizontal, longitudinal and the shearing strains, as shown in Table 4.

Table 4: Maximum CFRP strain under different earthquake levels

target	Column end plastic hinge region			Node area	
	The longitudinal *εl*(*με*)	The transverse *εl*(*με*)	The transverse *εl*(*με*)	The transverse *εl*(*με*)	Shear (*με*)
0.15g	437	45	34	10	132
0.30g	728	67	72	17	215
0.40g	808	114	473	28	260
0.50g	895	315	1114	458	896
0.70g	1007	321	916	24	599
1.00g	1077	551	1453	722	1440

5. Conclusions

Anti-seismic property research and analysis are conducted with the FRP-reinforced non-ductile reinforced concrete frame structure model. Comparison is made between the experimental model and the unreinforced structure, to analyze changes on the anti-seismic property based on experimental analysis. According to research findings, CFRP reinforced model and unreinforced model have the same initial dynamic property; i.e., the initial rigidity of the FRP-reinforced non-ductile reinforced concrete frame structure is less influenced. After the FRP reinforcement, the reinforced concrete frame structure model has a relatively smaller rigidity degeneration, contributing to reducing of failures of structural frame. Under the seismic effect, the structural displacement of the FRP-reinforced non-ductile reinforced concrete frame structure model is relieved; however, the model in this article cannot meet seismic demands of large earthquakes. No in-depth research on the FRP-reinforced non-ductile reinforced concrete frame structure model has been conducted in this article due to limited length, such as influences of reinforcing bar sizes, space frame joints and filled walls.

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