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Seismic Performance of Two Story RC Frames with infill walls

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Abstract—This paper presents the results of the experimental and analytical investigations conducted on four 0.8 scale 2-story one bay ductile reinforced concrete (RC) frames with infill nonstructural walls subjected to cyclically increasing loads. The material properties and the member sizes of beams and columns in the four RC frame specimens are identical, but with different types of infill nonstructural wall. These four frames are the pure frame, frame with short column, frame with short beam and frame with wing walls. The four RC frame specimens were designed and constructed according to the general prototype building structures in Taiwan. Test results indicate that the ductility behavior of the frames with infill wall is similar to those of the pure frame. The ultimate base shear strength of the frames with infill walls is higher than those of the pure frame.

***Index Terms*—cyclic loading, infill RC wall, reinforced concrete.**

I. INTRODUCTION

The stiffness characteristics and shear strength of shear walls was investigated during the 1990s; Elnashai *et al*. [1]; Farrar and Baker [2]; Cheng *et al*. [3]; Eberhard and Sozen [4]. In the past 30 years, attention was given to the seismic behavior of reinforced concrete shear walls. Strength, ductility characteristic and energy dissipation capacity of shear walls under earthquake loading have also been studied; Pilakoutas and Elnashai [5], Mo and Lee [6]. However, the study on the interaction and affection of the infill reinforced concrete wall to the frame structure is very limited.

During the 1999 Chi-Chi Taiwan Earthquake, the magnitude of 7.3, many bridge and building structures in Taiwan were suffered huge damages. After some research, it was discovered that in the low rise buildings are many reinforced concrete nonstructural walls. Due to the lateral stiffness and strength of the frames are much different from those of the wall elements, the seismic behavior of these low rise buildings is sometimes controlled by the walls inside the frames. We can find this trend in the experimental and analytical reports in which the performance of the non-ductile frames with infill walls is much improved than those in just non-ductile frames on both lateral stiffness and strength. When it comes to ductile frames, which were already met the general prototype building structure codes in Taiwan, we want to know if the contribution of infill walls is the same as it is in the non-ductile frames, and the behavior of the ductile frames after simply repairing and retrofitting after earthquake. As for which kind of the infill walls inside the ductile frames, for usefulness we use openings like a door or a window. Then after test, we use cement mortar and epoxy to repair the specimens, and use steel jackets to retrofit the weak regions on the specimens, like short columns and short beams. For these purposes this project used four 0.8 scale 2-story one bay ductile reinforced concrete frames with infill nonstructural walls to be constructed and tested in the laboratory of the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. This paper presents the key experimental and analytical results.

II. TEST STRUCTURE AND TEST PROGRAM

The RC member sizes of the test frames were selected using ductile frame structures with the scale of 0.8, compared with normal prototype structure buildings. These four specimens have identical concrete and reinforcing materials, construction conditions, except that the other three specimens have different type of infill nonstructural walls: the frame element (specimen FE), the frame element with short column (specimen SCFE), the frame element with short beam (specimen SBFE) and the frame element with wing wall (specimen WWFE), as shown in Fig. 1. Then these four specimens were repaired and retrofitted at fail regions after first test, as shown in Fig. 2. The floor-to-floor height is 2350mm, and the column centerline spacing is 4000mm. In addition, the dimensions of the columns, beams and the thickness of the nonstructural walls are 400x400mm, 400x300mm and 100mm,

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respectively. The assumed nominal strength of the concrete is 210 kg/cm2. The reinforced bars of CNS SD 42 (yield strength of 4200 kg/cm2) for a diameter larger than 10mm and CNS SD28 (yield strength of 2800 kg/cm2) for a diameter equal to 10mm were specified. However, the actual material strength of the specimens is given in Table1. After test, as shown in Fig. 2, these four specimens were repaired. The loose concrete has been removed and replaced by cement motor, and cracks were filled with epoxy. In addition, the specimen FE was retrofitted using wing walls, 100mm thick and 700mm wide, and the specimen SCFE and SBFE were retrofitted using steel jacket at short column regions and short beam regions, respectively. All steel jackets were A36 grade, and the thickness was 6mm. The steel jackets were welded together before anchored by filling epoxy inside the steel jackets and bolted by 12mm diameter chemically adhered anchor bolts at spacing of around 300 to 400 mm. To differentiate the repaired specimens from original ones, the name of these repaired specimens were putting a letter "R" in back of the name of the original specimens, like specimen FE became specimen FER. Finally, in order to ensure a rigid foundation, a 740 mm high footing was constructed and mounted on the strong floor by eight pre-stressing rods with 900 kN pretension in each one.

Table 1. Strength of concrete

Unit: kgf/cm^2	FE	WWFE	SCFE	SBFE
Foundation	208.08	208.08	215.97	215.97
First Floor	147.52	147.52	198.22	198.22
Second Floor	194.56	194.56	268.96	268.96

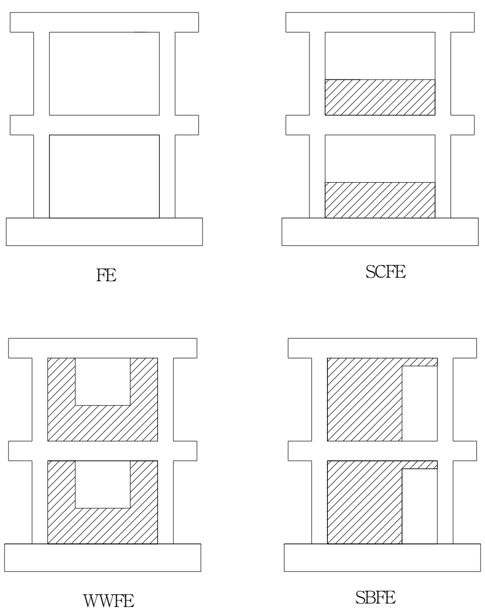

Fig 1. Sketch of the Specimens

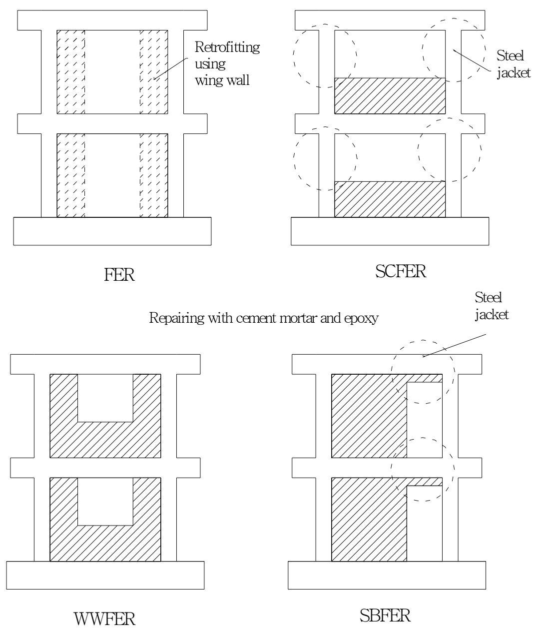

Fig 2. Sketch of the retrofitted specimens

The test employed three actuators to impose cyclically increasing lateral displacements to the specimens. As shown in Fig. 3, two actuators were positioned at the elevation of the beam centerline of the second floor, and the other one was located at that of the first floor. However, a hybrid control scheme was adopted where only one master actuator in the second floor was programmed to apply specific displacement history while another two actuators were slaved to strictly follow the forces measured in the master actuator. Thus, a specific cyclically increasing roof displacement history can be achieved while an inverted triangular force distribution pattern can be maintained throughout the test. The second to first floor lateral force ratio of 2 is to simulate the effect of the first vibration mode of the frame specimens. In addition, an axial load of $0.15f_c'Ag$ was applied on the top of each column in order to simulate the cumulative axial load in the columns of the actual structure.

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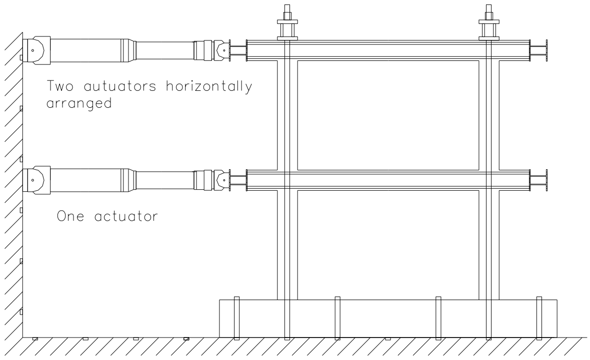

Fig 3: Configuration of experimental setup.

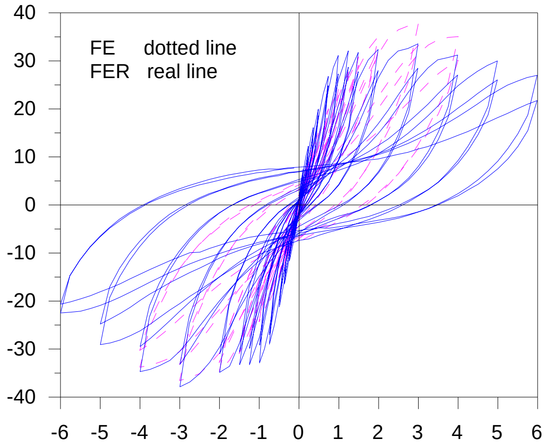

Fig 4. Test responses of specimens FE and FER

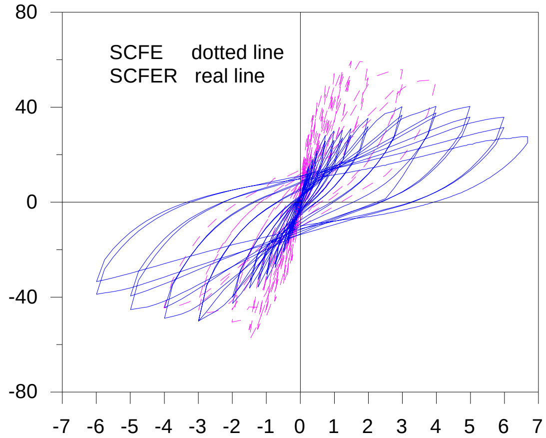

Fig 5. Test responses of specimens SCFE and SCFER

III. TEST RESULTS

A. Specimen FE and FER

Fig. 4 shows the cyclic roof drift ratio versus base shear force relationships for specimen FE and FER. During the testing of specimen FE, concrete cracks developed near the beam-ends and the joint on second floor when a roof drift reached 0.01 radian. When the roof drift reached 0.02 radian, many concrete cracks developed. The crack width has extended to 5mm on the plastic hinging of the beam ends while the unconfined concrete have large cracks and have come off a little bit on the top of the beam of the second floor when the roof drift reached 0.03 radian. At drifts of 4%, the maximum crack width was 10mm, and the unconfined concrete on the beam of second floor have failed (Photo.1). At this time, it was found that the peak lateral strength seemed to have developed and the lateral strength of the specimen have started to drop at 4% drift, so testing procedure was stopped. The maximum base shear was 40 tons at 3% drift. After repaired and retrofitted, the specimen FER was tested again. Because having added wing walls into this frame, the lateral stiffness and strength are better than specimen FE at early stages of the test. When the roof drift reached 1.5%, cracking started to appear at the bottom of the wing wall near the footing, and also, appear at the top of the beam of the second floor. Compared with cracks happened on the top of the beam on specimen FE, cracking on specimen FER did not extend along the whole beam, but along the shorter beam after wing wall was constructed. The crack pattern on specimen FER is the same as specimen FE, except that it is shorter on specimen FER because of wing walls. Moreover, the wing wall failed only at downside near footing, suggesting that it may not develop complete shear strength (Photo 2). The maximum base shear was measured to be 38 tons at -3% drift, similar to specimen FE.

B. Specimen SCFE

The hysteretic relationship is shown in Fig. 5. At 0.5% drift, cracking started to appear at the top plastic hinging of

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the column of first floor, and the connection of the infill wall and the column separated a little bit. When the roof drift reached 1%, the crack width of the column plastic hinging was 0.7 mm, the shear crack width on the wall was 2.5 mm and the gap between column and wall was 5 mm. After 1.5% drift, there were many cracks starting to appear more on short column regions. At 3% drift, the major cracking happened on the wall at the connection of the column and the wall. The test stopped at 4% drift when the strength of the specimen started to drop. The base shear strength of 60 tons appeared at 1.5% drift. The test proceeded again after repairing the cracks using the cement mortar and epoxy, and putting on steel jackets on short column regions. The lateral stiffness and strength of specimen SCFER at early stages of the test were shown to have much decline than those of specimen SCFE. Cracking initially happened on walls at the connection of the column and the wall, just like specimen SCFE but more severely, and then continued extending at the bottom of the column due to the effect of steel jackets on short column regions. At 5% drift when the wall has failed, it acted just like a pure frame (Photo 3), and the testing stopped at 7% drift. The maximum base shear was 50 tons at 3% drift.

Photo 1. The major cracking on the top of the beam


Photo 2. The final failure on specimen FER

Photo 3. Beam end cracked after wall failed

Photo 4. The wall damaged severely at 1.5% drift

C. Specimen SBFE

Fig. 6 shows the relationship between the base shear and roof drift. At 0.25% drift, there were several parallel cracks appeared on the column, and some shear cracks, the width of 0.9mm, on wall. At 0.5% drift, cracking happened on short beam region over the opening, a door, and so did on joint near the opening side. Also, the crack width of 2mm was measured on wall. As the roof drift reached 1% drift, many cracks developed on the region over the opening and on the wall as well. The crack width on the wall was 6.6mm. In addition, some small concrete pieces had started to drop. When the roof drift reached 1.5%, the infill wall damaged severely; however, the frame just slightly injured, especially on the short beam region (Photo 4). The maximum base shear strength was 144 tons at 1% drift. Similarly, because the base shear had dropped, the test was stopped. After repairing, specimen SBFER proceeded to another test. Because of un-symmetry on the specimen, the unbalanced strength was even more obvious on specimen SBFER than on specimen SBFE, the specimen before repaired. The failed region was similar to specimen SBFE. After testing, the frame without the infill wall was still in a moderate shape. The base shear

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strength of 124 tons, slightly lower than specimen SBFE of 144 tons, both happened at 1% drift.

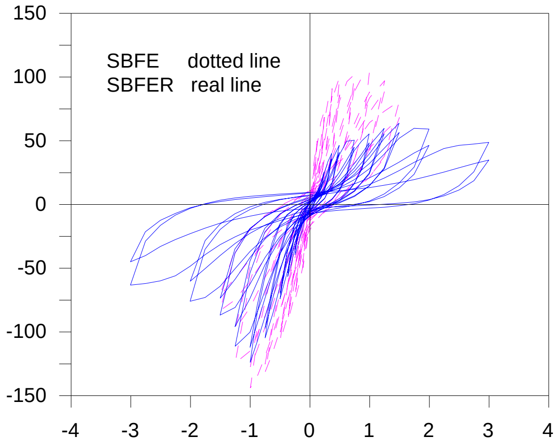

Fig 6. Test responses of specimens SBFE and SBFER

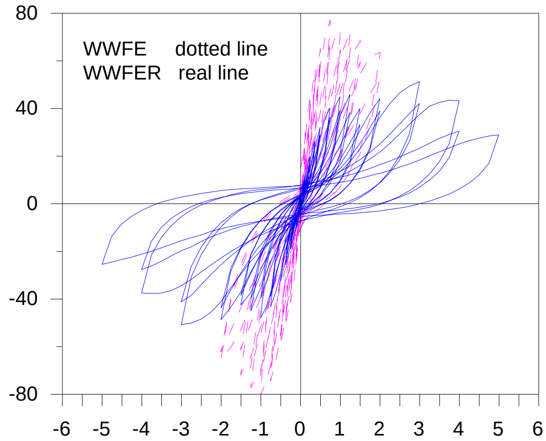

Fig 7. Test responses of specimens WWFE and WWFER

Photo 5. Cracking happened at the corner of the walls

Photo 6. Specimen WWFER acted like a frame after wall failed

D. Specimen WWFE

The base shear and roof drift relationship are shown in Fig. 7. At 0.5% drift, cracking started to appear at the bottom of column plastic hinging and near the corner of the opening of the wall. The crack width was 1 mm and 4 mm, respectively. When the roof drift reached 1%, cracking gradually became large at both corners of the openings of the wall. The reinforced bar inside the wall could be seen due to failed concrete on the wall at 1.25% drift. The wall deteriorated as the roof drift became larger. In addition, there were cracks developed along the top of the beam. The test stopped at 2% drift (Photo 5), and the maximum base shear strength was 80 tons measured at 1% drift. Like other specimens, specimen WWFER, the repaired specimen WWFE, was tested again after simple repair procedures had been done. At early stages of the test, the lateral stiffness and strength of specimen WWFER had much difference compared with original WWFE. The cracks happened mainly at the same positions where the cracks happened before on the walls. Also, according to its hysteretic loops, it acted just like a frame without infill walls (Photo 6). The strength gradually developed to its maximum level, 51 tons at 3 % drift. At last, the test stopped at 5% drift.

IV. CONCLUSIONS

Based on the experimental and analytical results, the following conclusions can be drawn.

1. Test results indicate that the specimen SBFE has the largest lateral stiffness and the ultimate strength as compared to other specimens. The specimen SCFE has a better ductility capacity as compared to the specimen FE, even though the SCFE specimen posses the short column effect. This point is quite different with the common engineering view.

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2. For those repaired frames, the failure starts at the places that have damaged in the first test and which propagate very rapidly and seriously. The effects cause lateral strength of the simply repaired frames is lower than that of the original frames.

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