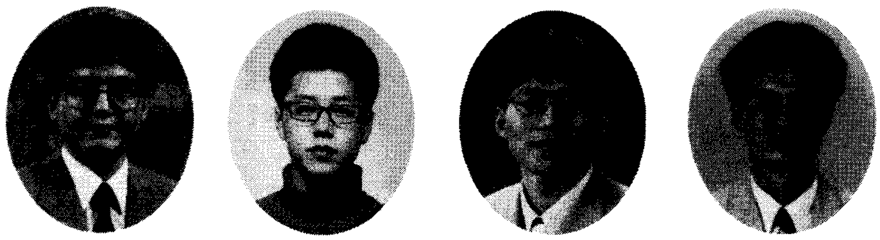
Pushover Tests of 1:5 Scale 3-Story Reinforced Concrete Frames



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ABSTRACT

The objective of the research stated herein is to observe the elastic and inelastic behaviors and ultimate capacity of 1:5 scale 3-story reinforced concrete frame. Pushover tests were performed to 1:5 scale 3-story reinforced concrete frames with and without infilled masonry. To simulate the earthquake effect, the lateral force distribution was maintained by an inverted triangle by using the whiffle tree.

From the test results, the relation ships between the total lateral load and the roof drift, the distribution of column shears, the relation between story shear and story drift, and the angular rotations at the critical portions of structures were obtained. The effects of infilled masonry were investigated with regards to the stiffness, strength, and ductility of structures. Final collapse modes of structures with and without infilled masonry were compared.

Keywords : pushover test, masonry wall, reinforced concrete frame, ductility, strength, stiffness

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. 1. INTRODUCTION

In reinforced concrete (RC) and steel structures, the masonry infills can be frequently found as interior and exterior partitions. However, they are normally considered as an architectural elements, and their influence on the structure is often ignored by engineers. It has been reported from a number of researches and the post-earthquake safety evaluation of existing buildings that the masonry infills may or may not be beneficial to the performance of the structure, and this interaction effect has been a subject of many debates.

The behavior of the masonry-infilled frame under lateral loads has been investigated by a number of researchers. Holmes(1961), Stafford Smith(1962, 1966, 1967), and Mainstone and Weeks(1970) performed experimental and analytical researches on the lateral stiffness and strength of steel frame infilled with mortar and concrete panels. Dawe and Seah(1989), Flanagan et al.(1992), and Mander et al.(1993) studied the behavior of masonry infilled steel frame under in-plane and out-of-plane loads. Dhanasekar and Page(1986) developed finite-element models for analyzing masonry-infilled steel frames. Wood (1978), Liau and Kwan(1985) developed plastic analysis methods to evaluate the in-plane limit loads of steel infilled frames.

The behavior of masonry-infilled RC frames is generally more complicated than that of steel infilled frames. Study on this subject has been conducted by a number of researchers including Fiorato et al.(1970), Klingner and Bertero(1976), Kahn and Hanson(1979), Bertero and Brokken (1983), Zarnic and Tomazevic(1990), and others. These studies have made it possible to identify innumerable failure mechanisms that can be caused by the frame-panel interaction.

In this study was selected a 3-story RC frame designed for gravity load which is in use as a police office building in Korea, and its 1/5 scale model was prepared. With this model, pushover tests were carried out under displacement control, and the influence of the masonry infills on the structure was investigated. Since the pushover analysis technique can provide much information on the global as well as local deformation demands and capacities that govern the seismic performance of the structures up to the collapse, this technique has become a useful and general tool in the recently developed performance-based design methodology. Therefore, though the reliability of this technique should be verified through experimental proof, the efforts in this regard are not sufficient particularly in the case of the structures not designed for the earthquake hazard. The study reported herein addresses the investigation of the behavior of this kind of reinforced concrete frame with and without infilled masonry walls through the experimental pushover test.

2. EXPERIMENT

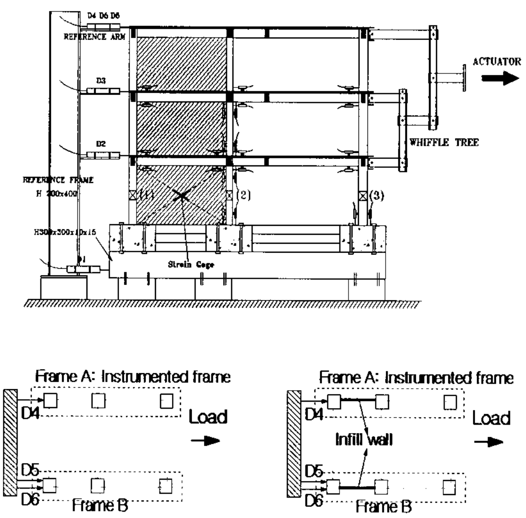
2.1 Test Setup and Experimental Program

The test model used in this study has a typical non-seismic details such as lap-splices at the bottom of the columns, large spacing of hoops, no hoops in the beam-column joints, and no use of 135° seismic hooks.

As shown in Table 1 and Figure 1, absolute displacements of each story were measured by using transducers installed on the reference frame. The load cell was set up in the mid-height of the first story column to measure the shear force at each column; 22 displacement transducers were used to measure local responses (local rotation) in the critical regions. The roof story drift was obtained by averaging the two measured value in both Frame A and Frame B. Two different data acquisition systems were used due to the limitations in the number of available channels in the data acquisition systems, but the measured data were synchronized with each other by comparing the displacements of transducers D5 and D6 in Figure 1. Figure 3 shows a comparison of the roof displacements at Frame A and Frame B, where no significant discrepancy was found. So the experimental results were interpreted, assuming that behavior of Frame A represents that of the whole model structure. The displacement at the second floor(D2) and third floor(D3) were those measured at the middle of both frames. The whiffle tree was constructed to pull the specimen, and the lateral force distribution was maintained in the shape of an inverted triangle[see Figure 2]. To prevent whiffle tree from transferring the moment, the joints in the whiffle tree were made as hinge connections. The detailed test setup is shown in Figure 1 and Photo 1.

Table 1 Type and number of sensors (unit: ea)

	Displacement transducer (for story drift)	Load -cell	Displacement transducer (for local rotations)	Strain gage
Table	1(D1)	6	14 ea (for beam) 10 ea (for column)	4
Second Floor	1(D2)			
Third Floor	1(D3)			
Roof	3(D4, D5, D6)			
Sub-total	6	6	24	4
Total: 40				



(a) Bare Frame(BF) (b) Infilled Frame(IF)

Fig.1 Experimental setup(unit: mm)

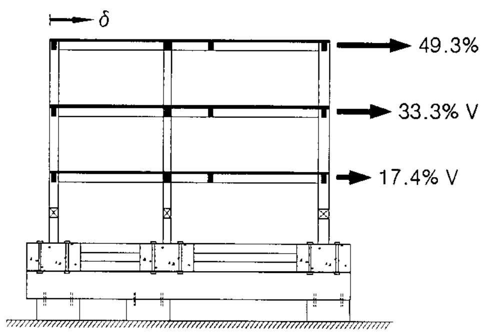
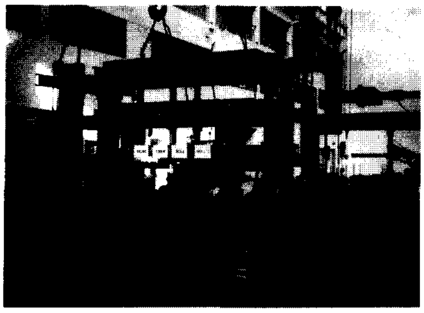
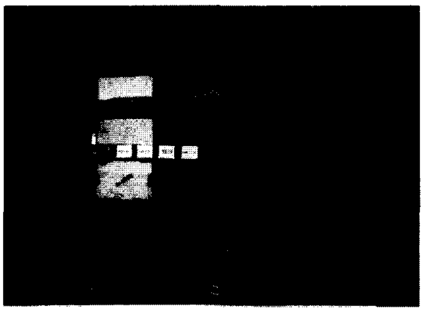


Fig.2 Lateral force distribution

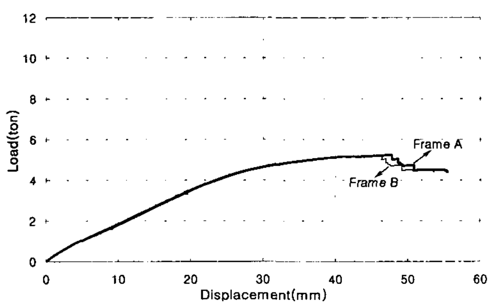


(a) Bare Frame(BF)

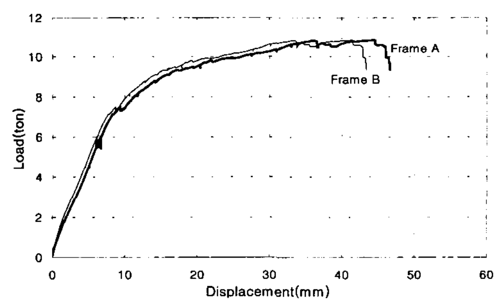


(b) Infilled Frame(IF)

Photo 1



(a) Bare Frame(BF)



(b) Infilled Frame(IF)

Fig.3 Relations of roof displacement versus Load

3. EXPERIMENTAL RESULTS AND INTERPRETATION

3.1 Lateral Load versus Roof Drift

In the bare frame(BF), as shown in Figure 4, the gradual yield phenomenon is noticed, but displacement ductility is not great; this implies that this model structure has the characteristic of the structure with the limited ductility due to non-seismic details. In BF, the collapse load of the structure is found to be 5.24 tf and the initial stiffness 0.19 tf/mm, and the lateral displacement capacity appears 47.2 mm while the yield displacement is 25.2 mm. The latter indicates the displacement ductility ratio of about 1.87. In the infilled frame(IF), the collapse load and the initial stiffness are 10.86 tf and 0.97 tf/mm, respectively. Considering that the yield displacement is about 9.3 mm and the lateral displacement capacity is 43.1 mm, the displacement ductility ratio of the structure appears to be about 4.63. Though the lateral displacement capacity of IF is a little smaller than that of BF, the stiffness and displacement ductility ratio appear to be larger than those of BF(Table 2).

Table 2 Strength, stiffness, drift, and displacement ductility ratio

	Bare Frame(1)	Infilled Frame(2)	(2)/(1)
Strength	5.24t	10.86t	2.07
Stiffness	0.19 t/mm	0.97 t/mm	5.11
Yield drift at roof	25.2mm (1.14%)*	9.3mm (0.42%)*	0.37
Drift capacity	47.2mm (2.13%)*	43.1mm (1.94%)*	0.91
Displacement ductility ratio	1.87	4.63	2.48

The number in ( )* means drift ratio(% of building height)

3.2 Base Shear

As shown in Figure 4, the yielding base shear of the BF is estimated to be about 4.8 tf, which is found to be about 12.6 times of the design base shear(0.38 tf)[14]; the maximum base shear is about 5.24 tf, which is 13.8 times and 1.1 times of the design and the yield base shear, respectively. Also, in the case of IF, yield base shear is about 9 tf, which is found to be about 19.1 times of the design base shear(0.47 tf); the maximum base shear is about 10.86 tf, which is found to be 23.1 times and 1.2 times of the design and the yield base shear, respectively.

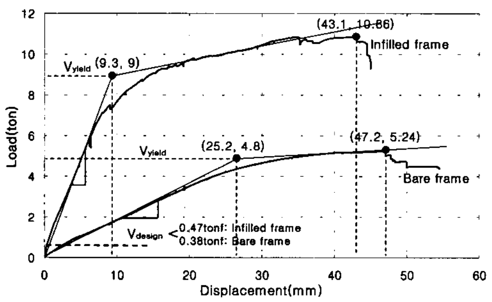


Fig.4 Lateral load versus roof drift

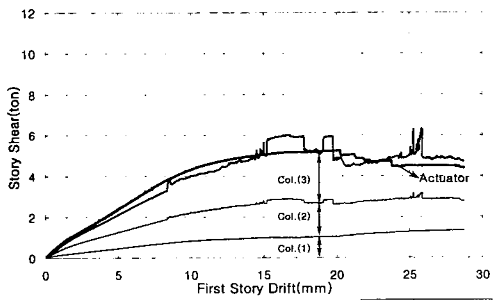
3.3 Column Shears versus Total Lateral Load

Figure 5 shows the applied lateral load and the sum of the column shears measured in the load cells as the functions of the interstory drift at the first story. In the case of BF shown in Figure 5(a), the sum of the values measured in the load cells is found almost equal to the applied load, which proves the accuracy of the load cells. For IF in Figure 5(b), the shear force measured in the load cell is found to be approximately 20% of the applied lateral load(17% at interstory drift ratio(I.D.R.) 2 mm and 22% at I.D.R 8.4 mm). This means that the masonry infills carry approximately 80% of the total base shear and significantly contribute to the increase in the total strength of the structure. The initial stiffnesses at I.D.R. of 2 mm appear to be 0.6 tf/mm for the column shear and 2.93 tf/mm only for the infill wall. Therefore, the total stiffness of 3.53 tf/mm in IF is much higher than that of the BF(0.56 tf/mm).

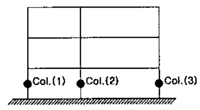
It can also be seen that the contribution of Column(1) to the shear resistance is almost negligible. The masonry infills show linear elastic behavior up to 2.3 mm of the drift of the first story, but hereafter the nonlinear behavior of the masonry infills governs the response of the structure. Since the resistance of the masonry infills is degraded by their cracking at Point A, that is, no more increase is observed thereafter. However, the total resistance of the structure is still increasing due to the remaining capacity of the frame.

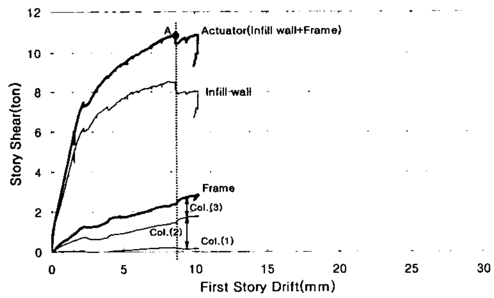
3.4 Interstory Drift versus Story Shear

Figure 6 shows the comparison of the relations between the interstory drift and the story shear. The maximum interstory drift in BF occurred at the first story, the value of which is 28.7 mm(3.7%). The concentration of deformations on the first story after the strength degradation indicates that failure mechanism was formed in the first story. In case of IF, considerably large interstory drift occurred at the second story, when compared with those of other stories, the value of which is 33.2 mm(4.6%). The distinct yielding point can be noticed and the large ductility and strength degradation indicates that the weak-story failure mechanism formed in the second story. Figure 7 shows the vertical distribution of the maximum interstory drift, the maximum story shear, and the initial story stiffness at each story. In this figure, IF shows clearly the concentration of deformation at the second story while BF reveals rather smooth decrease in the story drift over the height. The vertical distribution of the initial stiffness in IF shows that the second story has quite smaller stiffness than the first story. The ratio of the story stiffness of IF to that of BF is not constant. The highest ratio appeared 7.8 at the first story while the lowest ratio is 4.4 at the second story.



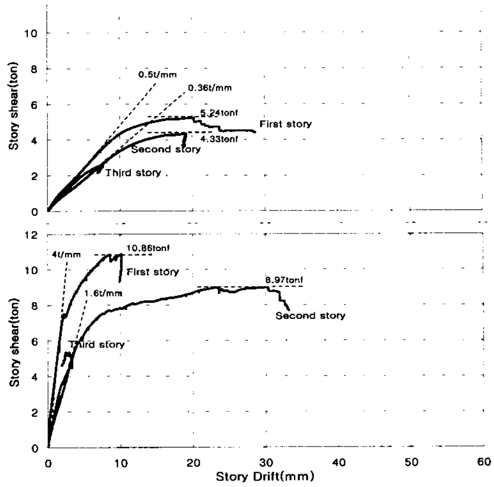
(a) Bare Frame(BF)





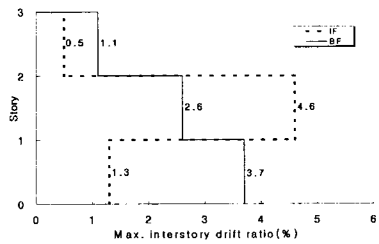
(b) Infilled Frame(IF)

Fig.5 Column shears and total lateral load

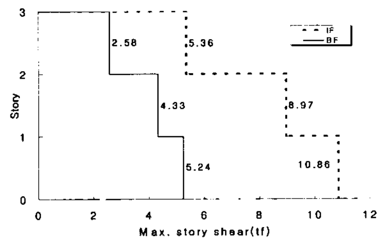


(b) Infilled Frame(IF)

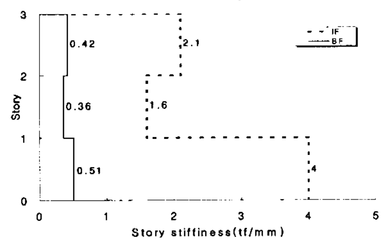
Fig.6 Interstory drift versus story shear



(a) Maximum interstory drift



(b) Maximum story shear



(c) Initial story stiffness

Fig.7 Vertical distribution of max. interstory drift, max. story shear, and initial story stiffness at each story

3.5 Local Deformations

Figure 8 shows the maximum rotations in the ends of the columns and beams up to the failure of the structures. In case of BF, rotations in the ends of columns are within the range of approximately 0.04 rad. and the largest values are obtained at the bottom of the exterior column located at the larger span in the first story. The failure occurred in the mode of the flexural compressive crushing at the upper end of the interior column at the first story, and the experiment should be stopped at this point. In the beam, the largest value of 0.026 rad. was observed in the end of the longer span at the exterior joint of the second floor.

The IF showed somewhat different responses: though the rotation in most of the column ends were within the range of 0.02 rad., the infilled wall in the second story was splitted into two parts horizontally, which made the interior column split into two short columns due to the relative movement of the separated infills. This resulted in the formation of the plastic hinges at the mid-height and at the bottom of the interior column in the second story and lead to the rotation of 0.081 rad. at the bottom of this column. Somewhat large rotation was observed at the bottom of the interior column in the first story. In the IF, it can be seen that relatively larger rotations have occurred in beams than in columns. The largest value of rotation was obtained at the right end of the beam in the longer span of the second floor: the left end of the beam in the shorter span of the third floor also show relatively large rotation.

Figure 9 shows the histories of rotations in the ends of beams and columns connected to the interior and the exterior joint at the second floor. In the BF, Figure 9(a), shows the ends of column(1) and (2) have about twice larger rotation than those of beams(3) and (4) under the same load.

In contrast to the above results, the IF reveals in Figure 9(b) that the rate of the increase in the rotations in the ends (1), (3) and (4) is half of those in the ends (2) and (5) at the initial stage. However, while those of the ends (1), (3) and (4) showed almost no increase in rotations, the rotations at the ends (2) and (5) rapidly when the lateral load exceeded the proportional-limit strength. The collapse due to the weak-story failure mechanism, which consisted of the plastic hinges at 2 columns and one shear failure at the remaining column, rendered the experiment to stop.

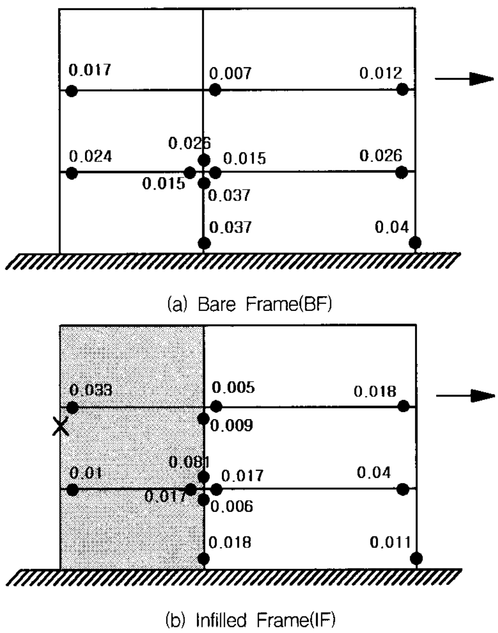
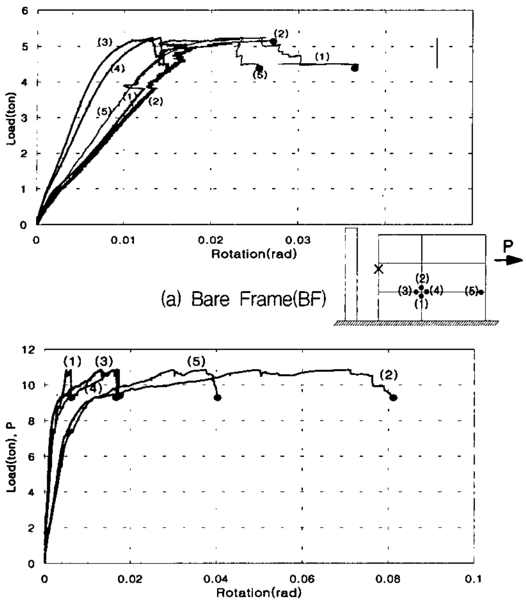


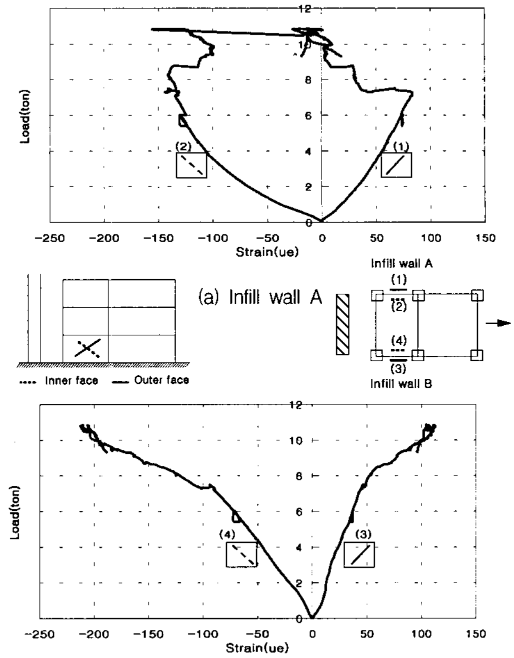
Fig.8 Maximum rotation in the ends of the columns and beams



(a) Bare Frame(BF)

(b) Infilled Frame(IF)

Fig.9 Histories of rotations in the ends of beams and columns



(a) Infill wall A

(b) Infill wall B

Fig.10 Strains in masonry infills

3.6 Strains in Masonry Infills

Figure 10 shows the strains measured with the strain gauges at the surface of the masonry infills in the first story. In the case of masonry infill wall A, load-resistant capacity appears to have decreased rapidly after the occurrence of the horizontal cracks in the joint of the masonry under the load of 7 tf. Masonry infill wall B appears to have resisted the lateral load to the last in compression and tension. The maximum strain appears to be 210×10^{-6} in compression and 110×10^{-6} in tension.

3.7 Failure Modes

Figure 11 shows the development of cracks and corresponding failure mechanisms. In the BF, the final failure of the structure was reached by the crushing of the concrete in the upper end of the interior column at the first story(Photo 2). In the IF, the weak-story failure mechanism was investigated(Photo 3), and this failure mode was initiated by the deepened cracks and large slip in the joint of the masonry infills at the second story, and followed by the shear failure in the exterior column and lastly by the formation of two plastic hinges in the interior column, the distance of which was shortened by the splitting and slipping of infilled wall.

4. CONCLUSIONS

The following conclusions have been drawn from the above test results and analysis.

(1) The behavior of the bare frame model revealed the clear yielding point,

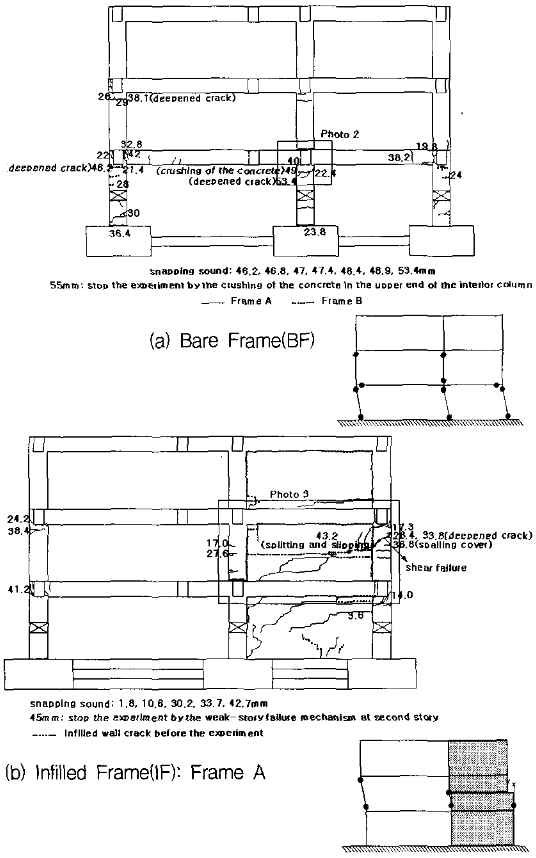


Fig.11 Development of cracks and failure mechanisms
(※ The number in the figure means roof drift)

but small ductility; these may be the typical characteristics in the nonlinear response of a structure with a non-seismic details.

(2) The comparison of the behaviors of BF with those of IF shows that the strength and stiffness greatly increased 2.07 and 5.11 times, respectively, while the lateral displacement capacity decreased negligibly when masonry infillswere used. The displacement ductility ratio of IF was about 2.48 times larger than that of BF.

(3) In IF, the sum of column shears appears to be about 20% of the total lateral load, which implies that the

Photo 2

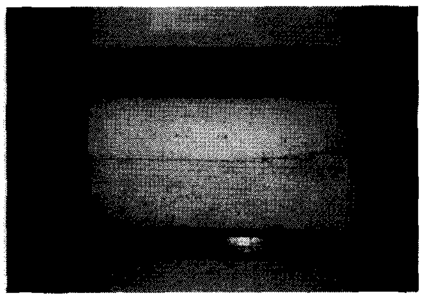


Photo 3

masonry infills carry about 80% of the total shear, thus contributing greatly to the increase of the total strength of the structure.

(4) In BF, the final failure of the structure occurred with the crushing of concrete at the upper end of the interior column at the first story. In IF, cracks at the joints of the masonry infills in the second story, which occurred before the experiment, led to dividing the wall into two parts and slippage through the joint crack. As a result of these movements, shear failure in the exterior column and the formation of plastic hinges in the interior column and finally the weak-story failure mechanism were developed at the second story.

ACKNOWLEDGEMENT

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