Structural Engineering and Mechanics, Vol. 22, No. 4 (2006) 449-467
DOI: http://dx.doi.org/10.12989/sem.2006.22.4.449

Experimental study and modeling of masonry-infilled concrete frames with and without CFRP jacketing

Chao-Hsun Huang† and Yu-Chi Sung‡

Department of Civil Engineering, National Taipei University of Technology, 1, Sec. 3, Chung Hsiao E. Rd., Taipei 106, Taiwan

Chi-Hsin Tsai‡†

Futai Engineering Co., 6F, 19-5 San Chong Rd., Taipei 115, Taiwan

(Received March 21, 2005, Accepted November 15, 2005)

Abstract. Most existing concrete structures in Taiwan are considered nonductile due to insufficient transverse reinforcement and poor detailing of frame elements. Such features are fairly typical for buildings constructed prior to 1997, at which time the local building code was revised based on ACI 318-95. Among these structures, many contain perimeter or partition walls made of concrete or clay brick for architectural purposes. These walls, though treated as non-structural components in common design practice, could affect the structural behavior of the buildings during an earthquake. To study the behavior of such structures under seismic load, experiments were conducted on concrete frames of various configurations to show the force-deformation relationships, damage patterns, and other characteristics of the frames. For further interest, similar units with columns jacketed by carbon-fiber-reinforced-polymer (CFRP) were also tested to illustrate the effectiveness of this technique in the retrofit of concrete frames.

Keywords: CFRP; infill; nonductile frame; seismic retrofit.

1. Introduction

Post-earthquake reconnaissance after the 1999 Chi-Chi Earthquake raised a serious concern regarding the structural safety of low-to-midrise concrete structures, especially ones that contain masonry infills (Loh and Tsai 2000, Tsai *et al*. 2000). These infills, usually constructed of clay bricks and mortar, could be beneficial to the seismic performance of the structures if constructed properly. Nonetheless, they could also be detrimental during an earthquake by creating problems such as soft-story, torsional irregularity, and short-column effects if they are poorly arranged.

Since earlier versions of the local building code did not contain the likes of the seismic provisions as it does today, many existing buildings would not survive an earthquake that reaches a certain level of intensity. For buildings not meeting current code requirements (CPA 1999), a structural

† Assistant Professor, Corresponding author, E-mail: steve@ntut.edu.tw
‡ Associate Professor
‡† Civil Engineer

upgrading or retrofit scheme should be considered. The scale of such retrofit is usually divided into two different levels - the element level and the structural system level. Since the latter often requires a thorough investigation on the structure, it is expected to generate a better result. In cases where resources are limited and compromises have to be made, element retrofit provides a simple alternative once critical areas of the structures have been identified.

Current engineering techniques used in the retrofit of concrete elements include, but are not limited to, supplemental confinement, steel jacketing, section enlargement, and addition of wing walls. In the first option, confinement provided by carbon-fiber-reinforced-polymer (CFRP) jacketing is often selected due to its high strength-to-weight ratio, ease of construction, and resistance against corrosion. According to findings of Li *et al*. (2003) and Harajli and Rteil (2004), the overall enhancement in strength and ductility of structural elements retrofitted with CFRP is satisfactory. Moreover, its advantages in both cost and construction time easily surpass many other retrofitting methods; therefore, it is becoming one of the most popular methods for seismic retrofitting.

In order to ensure the safety of existing buildings, further knowledge on their structural behavior will be necessary. Researchers such as Klingner and Bertero (1978), Kahn and Hanson (1979), Bertero and Brokken (1983), and Mehrabi *et al*. (1996) have conducted experimental investigations on the lateral stiffness and strengths of concrete frames infilled with reinforced and unreinforced masonry panels. For design and analysis, Holmes (1961), Stafford Smith and Carter (1969), and Saneinejad and Hobbs (1995) proposed the idea of equivalent diagonal strut and derived systematic methods to calculate the mechanical properties of such struts. Based on the equivalent strut model, a computer program for the inelastic damage analysis of buildings, IDARC2D, was developed by Valles *et al*. (1996) in the National Center for Earthquake Engineering Research (NCEER) at Buffalo, New York. One of the verifications on this program has been reported by Madan *et al*. (1997) based on experimental data of masonry-infilled steel frames.

In this study, six 2:3 scale concrete frame specimens were tested at the National Center for Research on Earthquake Engineering (NCREE) in Taipei, Taiwan (Huang and Tsai 2003). To investigate the seismic performance of concrete frames in different configurations, four of the six frames were constructed with either full or partial masonry infills inside while the others remain in the form of bare frames. In order to evaluate the effectiveness of the carbon-fiber confinement, half of the specimens were jacketed with CFRP sheets in column areas. An analytical investigation was performed using IDARC2D and was compared to the experimental results, as being presented in this paper.

2. Experimental investigation

2.1 Concrete frame units

Six concrete frame units, as shown in Fig. 1, were constructed and tested under simulated seismic loading. A 22 cm thick full or partial brick infill was constructed in four of the six units to represent different building configurations. For identification purposes, pure frame units, frames with partial infills, and frames with full infills were designated as BMNF, BMNFH10B, and BMNF10B, respectively, with the extension ‘F’ indicating CFRP-jacketed units, as shown in Table 1. In order to replicate the properties of concrete columns in nonductile frames, all column confinement was

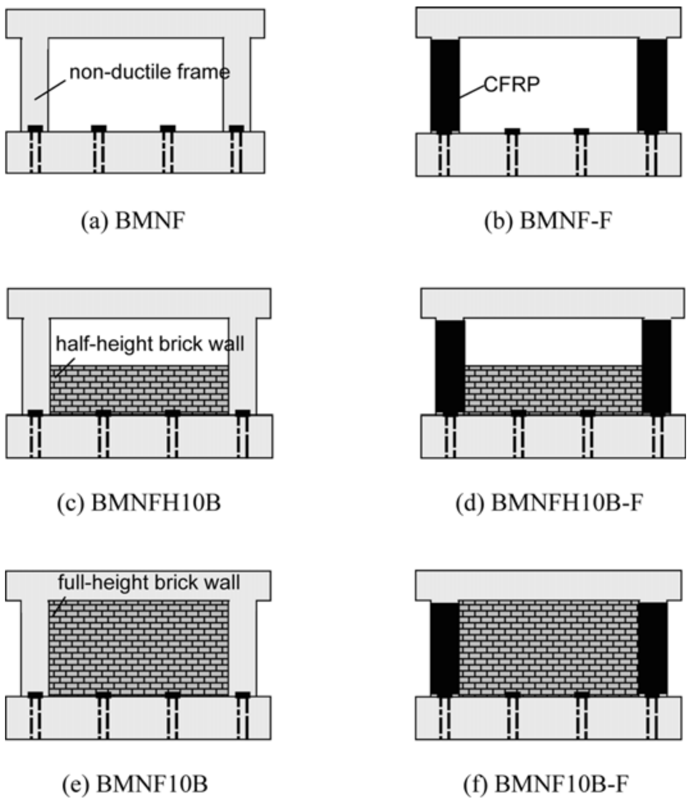


Fig. 1 Concrete frame units

Table 1 Frame properties

Unit ID	Infill	Retrofit	f_c' (MPa)	f_y (MPa) Long.	f_y (MPa) Trans.	HC	HBD	HS
BMNF	-	-	18.8	394	521	1.6	0.008	0.16
BMNF-F	-	CFRP	21.0			2.5	0.005	0.33
BMNFH10B	85 cm	-	19.6			1.8	0.009	0.15
BMNFH10B-F	85 cm	CFRP	19.6			1.8	0.009	0.36
BMNF10B	Full	-	19.7			1.0	0.005	0.15
BMNF10B-F	Full	CFRP	18.3			2.0	0.004	0.36

provided by insufficient transverse reinforcement with 90° hooks at both ends of the hoops.

The frames under investigation were constructed at an approximate 70% scale of a typical bay in residential buildings to fit the testing facility. Each unit was 220 cm in height, measured from the column base to the top of the beam, with a span of 300 cm between the centerlines of columns. Both the beam and columns of the frames have a cross section of 30 cm × 50 cm, as shown in Fig. 2, and reinforced with No. 6 longitudinal bars. To ensure that the primary damage would occur in

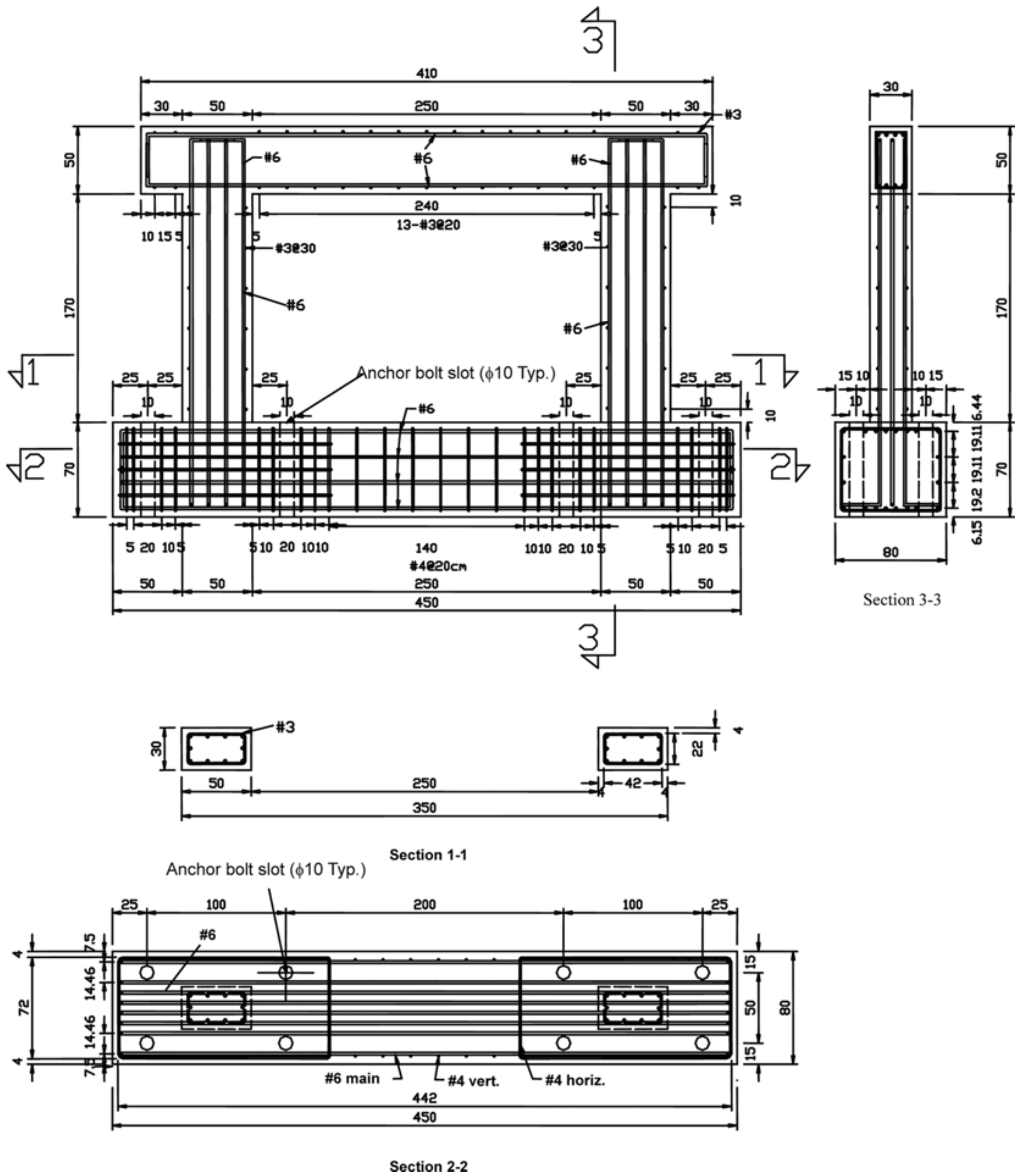


Fig. 2 Concrete frame details (unit: cm)

columns, as observed in many existing (nonductile) concrete structures, the beams were designed conservatively with details conforming to ACI 318-99 and the columns were constructed with non-conforming transverse reinforcement in which the rectangular hoops were spaced at 30 cm on centers with 90° hooks at both ends. Since the main focus of the study was to examine the effects of masonry infill and CFRP jacketing, bar slipping at lap splices was prohibited by making all longitudinal reinforcement continuous throughout the length of the columns.

Table 2 Compressive strengths of clay bricks and mortar

	Compressive strength (MPa)			
Sample No.	1	2	3	Average
Clay brick	20.28	22.61	17.22	20.03
Mortar*	5.40	3.24	4.22	4.29

*Sample size 50 mm × 50 mm × 50 mm

Table 3 Mechanical properties and thickness of CFRP sheets

Product type	FAW200
Modulus of elasticity (MPa)	230535
Ultimate strain	0.021
Allowable strain	0.015
Adhesive strength (MPa)	1.96
Thickness (mm/lamina)	0.11

2.2 Material properties

Ready-mix normal weight concrete with specified compressive strength $f_c' = 21$ MPa was used in all frame units. Longitudinal reinforcement in all framing members was provided by #6, grade 60 bars (f_y = 420 MPa nominal) with transverse reinforcement consisting of #3 or #4 grade 40 bars (f_y = 280 MPa nominal). The actual strengths of concrete and steel reinforcement are shown in Table 1. Clay bricks with nominal dimensions of 110 mm × 220 mm × 55 mm were used for the infills, and the compressive strengths of the bricks and mortar are given in Table 2. Two laminas of CFRP sheets with a total thickness of 0.22 mm were applied on the surfaces of retrofitted columns. The mechanical properties of the CFRP sheets are listed in Table 3.

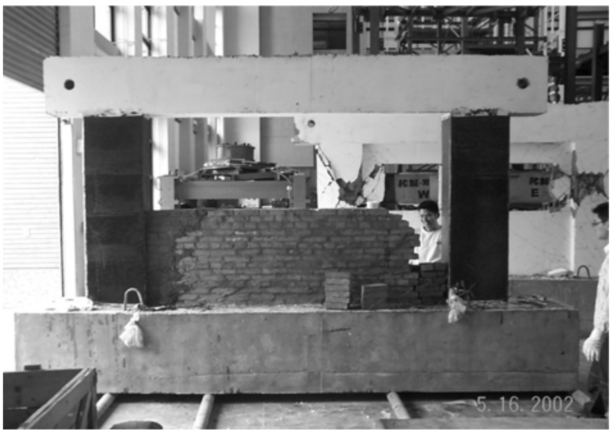


Fig. 3 Completion of brick wall (BMNFH10B-F)

2.3 Construction of specimens

All frame units tested in this project were constructed at the structural laboratory of NCREE with components built up according to the following order:

1. Foundation
2. Frame
3. Masonry infill (if any)
4. CFRP jacketing (for retrofitted units only)
5. Remainder of the brick wall (for frames BMNFH10B-F and BMNF10B-F only), if any (see Fig. 3)

For steps 1, 2, 3, and 5, the construction process was fairly typical and will not be described here. As for CFRP jacketing, the following procedure was taken:

1. For each column to be retrofitted, all four corners of the column section were smoothed to provide a transition zone between adjoining faces. The radius of curvature at these corners was taken as 30 mm to allow the attachment of CFRP.
2. A thin layer of primer epoxy was applied to the surface of these columns and cured for at least two hours under room temperature.
3. The CFRP sheets were applied to the column surfaces with the direction of fibers perpendicular to the longitudinal axis of the column. An overlay of 100 mm or more was used when carbon sheets had to be lapped. Epoxy was again applied to both the column surfaces and the carbon fiber sheets.

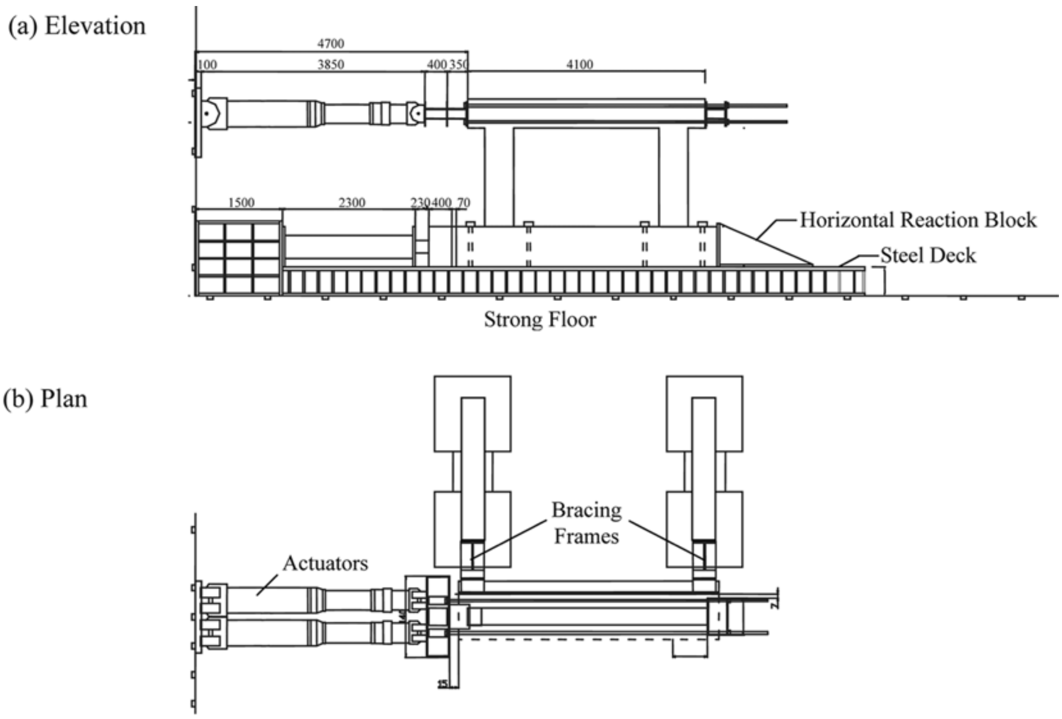


Fig. 4 Unit setup

2.4 Instrumentation

Since the main purpose of the experiment was to observe the frames' behavior under lateral loading, no vertical load was applied in the specimens except for the self-weight of the frames and walls. The lateral load was applied through a pair of 1000 kN, displacement-controlled MTS hydraulic jacks with a maximum displacement of 1016 mm. (See Fig. 4) The horizontal displacement of the frame was measured at the midspan of the top beam by a linear variable differential transducer (LVDT). Deformations at other locations were taken by dial or rotation gauges. The measured data were then transferred through a TML SHW-50D signal transmitter and a TML THS-1100 data collector, and then processed in the computer.

2.5 Quasi-static tests

The specimens were subjected to predetermined displacement excursions in a quasi-static pattern, as shown in Fig. 5. The displacement at the end of each cycle was progressively increased from 2.44 mm (drift ratio = 0.125%) in the first cycle to 39.0 mm (drift ratio = 2.0%), 58.0 mm (drift ratio = 3.0%), or 117.0 mm (drift ratio = 6.0%), at which significant damage occurred.

2.6 Determination of the yield point

In the analysis of nonlinear structures, the force-deformation relationship most frequently adopted is the 'bilinear model'. This model is typically used for structures or structural elements with a linear force-deformation relationship in both the elastic and the inelastic range. For concrete structures, unfortunately, the force-deformation relationship is not actually bilinear, and certain assumptions have to be made. In this study, the ductility of each concrete frame was calculated from the following equation:

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{1}$$

where Δ_u and Δ_y are the ultimate and yield displacement of the frame, respectively. Since the actual yield point of a concrete frame is hard to define, a method similar to the one proposed in FEMA-356 for the estimation of effective stiffness of nonlinear structures was used to obtain the yield point

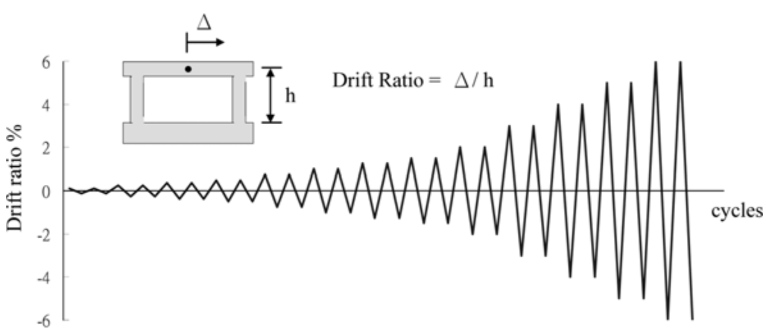


Fig. 5 Displacement history

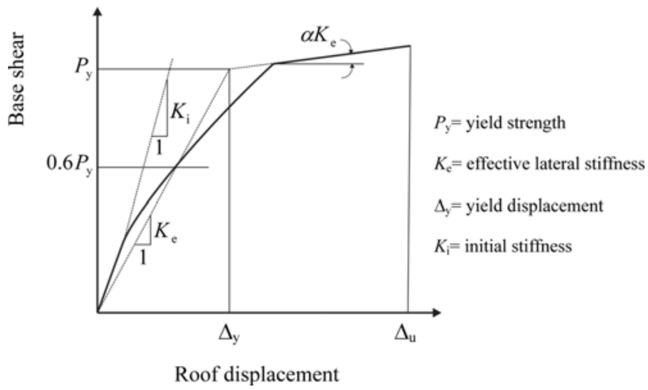


Fig. 6 Yield displacement of nonlinear structures (FEMA-356)

of each frame (See Fig. 6). According to this method, the yield strength of the structure, P_y, has to be identified first. Since the force-deformation curves of all frame units exhibited a mild-to-medium level of strength degradation instead of a plateau or hardening session after the strength reached a maximum value, P_{max}, this maximum strength was defined as the yield strength of the specimen, P_y. With P_y identified, the effective lateral stiffness of the frame, K_e was defined as the average stiffness between the origin and a point on the strength envelop where $P = 0.6P_y$, i.e.,

$$K_e = \frac{0.6P_y}{\Delta_{P = 0.6P_y}} \tag{2}$$

and the yield displacement Δ_y was calculated from

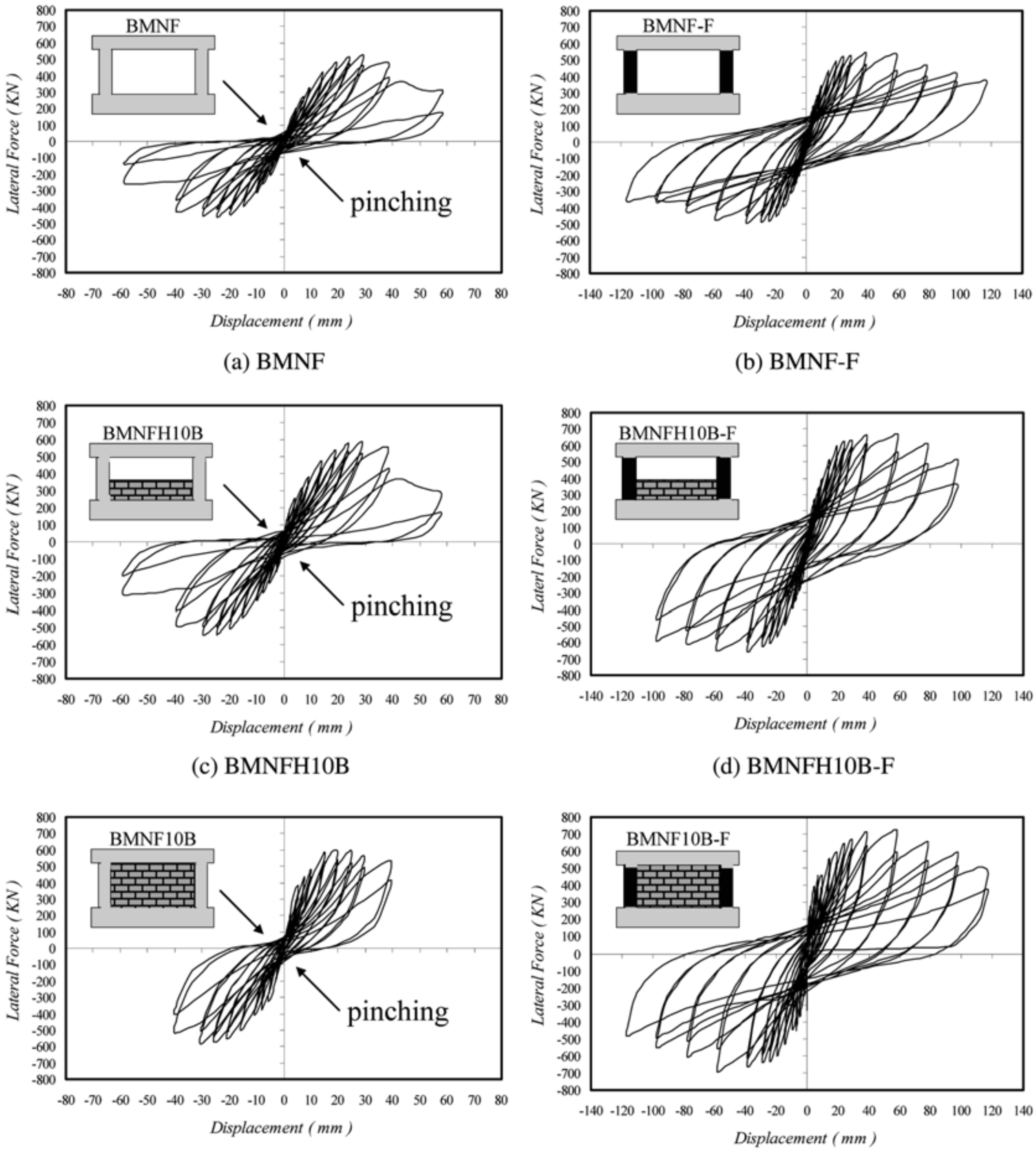
$$\Delta_y = \frac{P_{max}}{K_e} \tag{3}$$

Finally, the ultimate displacement Δ_u was taken as the displacement on the envelop curve where the residual strength P_{res} is equal to $0.8P_{max}$, and the ductility of the frames could be determined from Eq. (1).

3. Test results

3.1 Effects of the infills

Lateral load versus top displacement responses were recorded for the six specimens, as given in Fig. 7. An envelope curve was taken in both loading directions for each unit and then averaged to give the strength envelope of the frames, as shown in Fig. 8. It can be found that for both unretrofitted and retrofitted units, the addition of infills increased the maximum lateral strength of the frames. For unretrofitted units, the lateral strength of a frame, P_{max}, was raised from 518 kN for the pure frame (BMNF) to 583 kN for the partially infilled frame (BMNFH10B) and to 594 kN for the fully infilled frame (BMNF10B), which correspond to relative improvements of 12.5% and 14.6%, respectively. For units retrofitted with CFRP, the strength enhancement was even more obvious. In these frames, the lateral strength of the frames was increased from 542 kN for the pure



(a) BMNF (b) BMNF-F

(c) BMNFH10B (d) BMNFH10B-F

(e) BMNF10B (f) BMNF10B-F

Fig. 7 Hysteresis loops of tested units

frame (BMNF-F) to 666 kN (a 22.9% increase) and 719 kN (a 32.6% increase) for frames with partial and full infills, respectively (See Table 4).

Beside lateral strengths, the ductility of the concrete frames also benefited from the addition of full-height infill panels. Table 4 shows that for unretrofitted units, the displacement ductility μ was increased from 3.04 for the pure frame (BMNF) to 5.88 for the infilled frame (BMNF10B); and for units retrofitted with CFRP, the ductility was increased from 5.95 (BMNF-F) to 9.36 (BMNF10B-

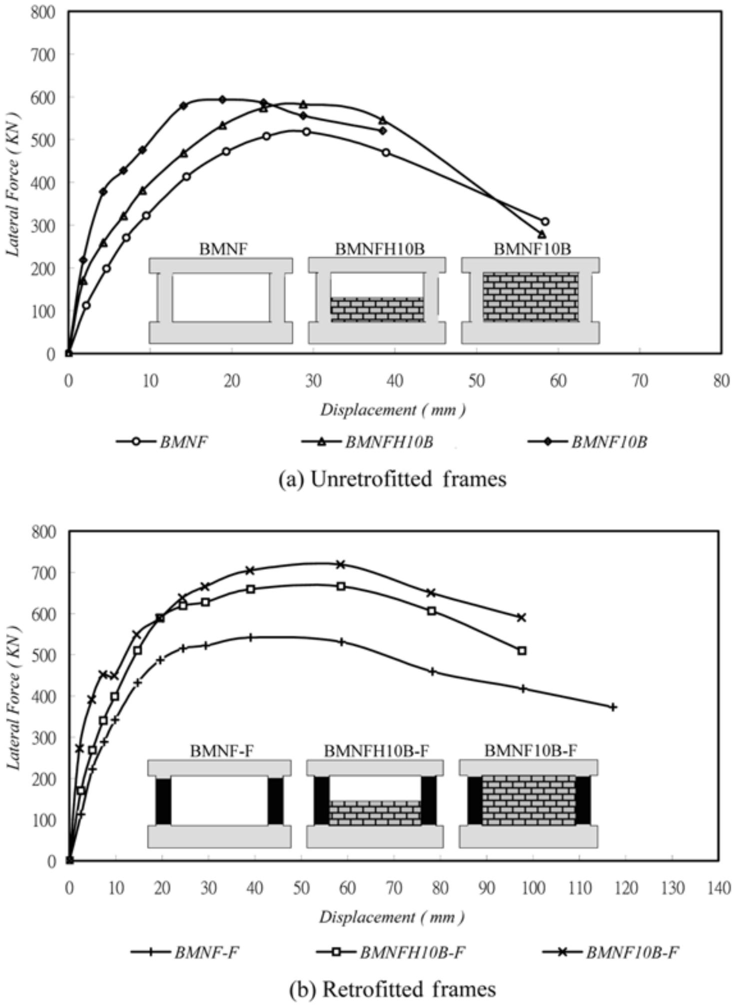


Fig. 8 Strength envelopes of tested frames

Table 4 Strength, ductility, stiffness, and total input energy of tested units

ID	$P_{\max}$ (kN)	Δ_y (mm)	Δ_u (mm)	$\mu = \Delta_u/\Delta_y$	K_i (kN/mm)	K_e (kN/mm)	E_I** (kN-mm)
BMNF	518	15.0	45.6	3.04	49.4	34.5	18474
BMNF-F	542	15.2	90.3	5.95	53.1	35.7	41811
BMNFH10B	583	13.1	44.3	3.38	76.2	44.5	20774
BMNFH10B-F	666	16.3	92.9	5.69	73.3	40.8	53286
BMNF10B*	594	6.5	38.5*	5.88	92.0	90.7	19414
BMNF10B-F	719	10.7	100.2	9.36	126.9	67.1	62269

*The test for unit BMNF10B was terminated before the residual strength dropped below 80% $P_{\max}$ due to safety reasons.

**E_I = total input energy at $\Delta = +\Delta_u$

F), respectively. If a half-height brick wall was constructed instead of a full-height panel, however, the ductility increase might not be valid. Table 4 shows that for unretrofitted units, the displacement ductility slightly increased from 3.04 for the pure frame (BMNF) to 3.38 for the partially infilled frame (BMNFH10B); for units retrofitted with CFRP, the ductility of the pure frame dropped from

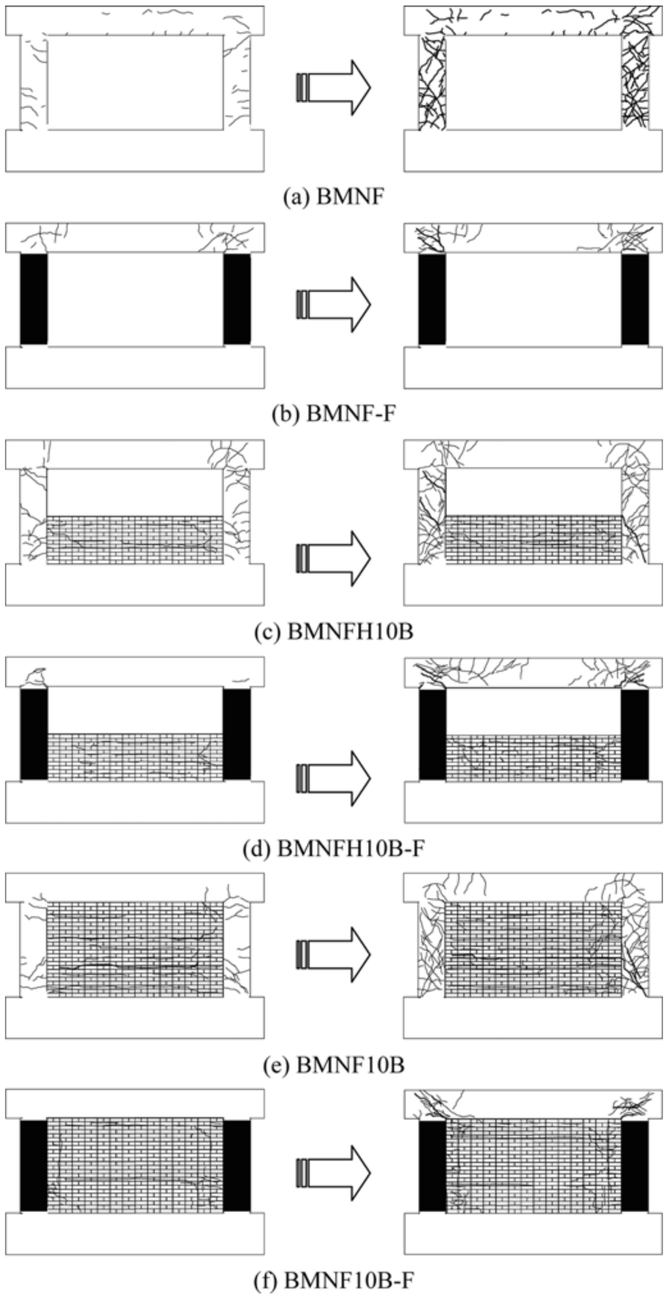


Fig. 9 Crack development in tested units

5.95 (BMNF-F) to 5.69 (BMNFH10B-F) after the addition of the half-height wall. One thing worth noticing is that for the unretrofitted infilled frames, the primary shear cracks in the columns formed a 'K' pattern, as shown in Figs. 9(a), 9(c), and 9(e), instead of the typical 'X' pattern observed in some other literatures (Park *et al.* 1975, Nilson *et al.* 2003). The development of these cracks in unit BMNF10B were even more obvious when sliding failure of the brick wall occurred near the mid-height of the infill, giving an example of what uneven constraints could do to the behavior of a concrete column.

3.2 Effects of the CFRP retrofit

The test results also showed that all three types of frames, i.e., frames with full, partial, and no infills, all achieved a strength and ductility increase after the retrofit. Comparisons between the performances of frames with and without retrofit are given in Fig. 10 through Fig. 12. Table 4 shows that the shear strengths, $P_{\max}$, of the pure, partially infilled, and fully infilled frames were increased by 4.6%, 14.2%, and 21.1% after the retrofit with the displacement ductility improved by 95.7%, 68.3%, and 59.2%, respectively.

The effect of retrofit was also observed through the variation of frame failure mechanisms. For unretrofitted frames, major shear cracks were found in all of the frame columns at the end of the test, as seen in Figs. 9(a), 9(c), and 9(e); for retrofitted units, on the other hand, only minimal damage occurred in frame columns, as shown in Figs. 9(b), 9(d), and 9(f). For the latter, the critical area of the specimens was shifted from the columns to the beam-column joints of the frames due to the strength increase of the columns after the retrofit. As a consequence, the frames were forced to develop another mechanism as the deformation proceeded.

Other than strength and ductility, the energy-dissipation of the retrofitted frames was also improved. Fig. 7 shows that the stiffness of the retrofitted frames did not have a significant drop as was experienced by unretrofitted units during the load reversals. In other words, the 'pinching' in the hysteresis loops was eliminated in the retrofitted units. This behavior, normally observed in ductile frames only, indicated that the CFRP-retrofitted frames were able to maintain a higher level of integrity under reversed cyclic loading even when inelastic deformation had taken place. By increasing areas enclosed by the hysteresis loops, the retrofit provided frames with a higher energy-

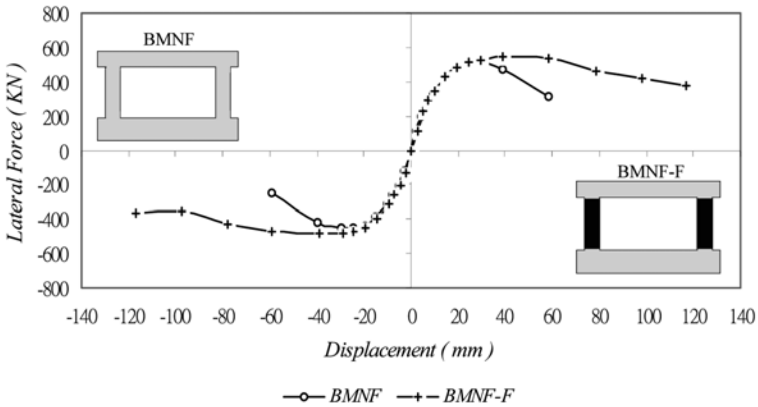


Fig. 10 Strength envelopes of pure frames

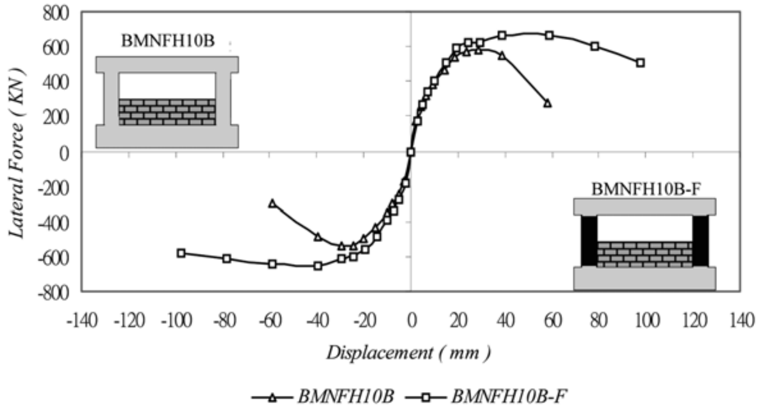


Fig. 11 Strength envelopes of partially infilled frames

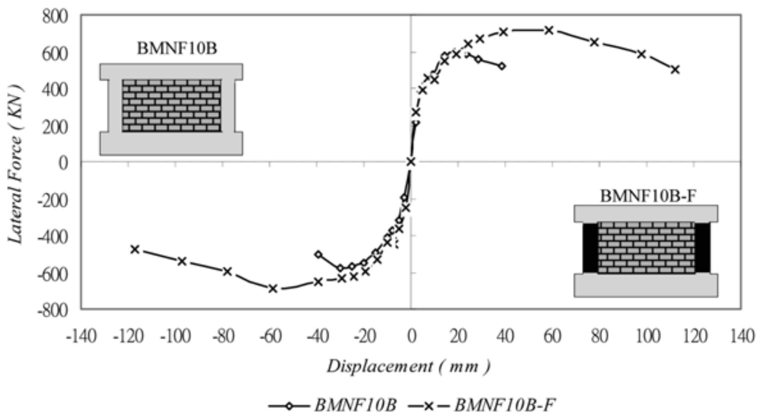


Fig. 12 Strength envelopes of fully infilled frames

dissipating capability under seismic actions. A preliminary analysis showed that the total strain energy (the area under the strength envelope) absorbed by the pure, partially infilled, and fully infilled frames at the ultimate stage ($\Delta = \Delta_u$) was increased by 128%, 115%, and 254%, respectively, after the retrofit.

4. Analytical procedure

An analytical investigation was conducted to emulate the behavior of the test units using the program IDARC2D, Version 4.0. The strengths of concrete, steel reinforcement, and clay bricks obtained from lab tests, as given in Tables 1 and 2, were input into the program to generate corresponding member properties. To estimate the strength of concrete in CFRP-jacketed members, the following constitutive model was employed.

The strength and deformability of structural concrete usually rise with the increase of confining pressure. According to Scott *et al.* (1982) the stress-strain relationship of confined concrete can be represented by the following equation:

$$f_c = f'_{cc}\left[-\left(\frac{\varepsilon_c}{\varepsilon'_{cc}}\right)^2 + 2\left(\frac{\varepsilon_c}{\varepsilon'_{cc}}\right)\right] \tag{4}$$

where ε_c is the strain of the concrete, and f'_{cc} and ε'_{cc} are the maximum concrete stress and the corresponding strain, respectively. The values of f'_{cc} and ε'_{cc} can be calculated from the following equations (Li *et al.* 2003):

$$f'_{cc} = f'_c + f'_{cs} + f'_{cf} \tag{5}$$

$$\varepsilon'_{cc} = 0.002\left[1 + 2.24\tan^2\left(45^{\circ} + \frac{\phi}{2}\right)\frac{f_l'}{f'_c}\right] \tag{6}$$

in which f_c' is the unconfined strength of concrete, f'_{cs} and f'_{cf} are the strength increases of concrete due to the confinement provided by transverse reinforcement and CFRP jacketing, respectively, and f_l' is the total lateral confining pressure provided by both transverse reinforcement and CFRP. According to Scott's model, f'_{cs} can be calculated from the following equation:

$$f'_{cs} = k \times f'_c \times \left[-2.254 + 2.254\sqrt{1 + \frac{7.94 \times f'_{l1}}{f'_c}} - 2\left(\frac{f_{l1}'}{f'_c}\right)\right] \tag{7}$$

where f'_{l1} is the effective lateral confining pressure from the transverse reinforcement and k is the confinement effectiveness coefficient; and based on Li's model, f'_{cf} can be obtained from

$$f'_{cf} = f'_{l2} \times \tan^2\left(45 + \frac{\phi}{2}\right) \tag{8}$$

in which f'_{l2} is the effective lateral confining pressure provided by CFRP

$$f'_{l2} = k_c\frac{2 \times n \times t \times E_{cf} \times \varepsilon_{cf}}{h} \tag{9}$$

where
- h = overall depth of the member
- k_c = shape coefficient
- n = total number of CFRP sheets
- t = thickness of CFRP per lamina
- E_{cf} = (longitudinal) elastic modulus of CFRP
- ε_{cf} = allowable strain of CFRP fibers

and ϕ is the angle of internal friction of concrete:

$$\phi = 36^{\circ} + 1^{\circ}\left(\frac{f'_c}{3.43}\right) \le 45^{\circ} \quad (f_c' \text{ in MPa}) \tag{10}$$

After getting f'_{l1} and f'_{l2}, the total confining pressure can be obtained:

$$f_l' = f_{l1}' + f'_{l2} \tag{11}$$

Since IDARC2D accommodates only fully infilled frames, a small adjustment had to be made in the modeling of perforated infilled frames such as BMNFH10B and BMNFH10B-F. For these units, an imaginary floor level was defined at the top of the brick wall to create an additional gridline;

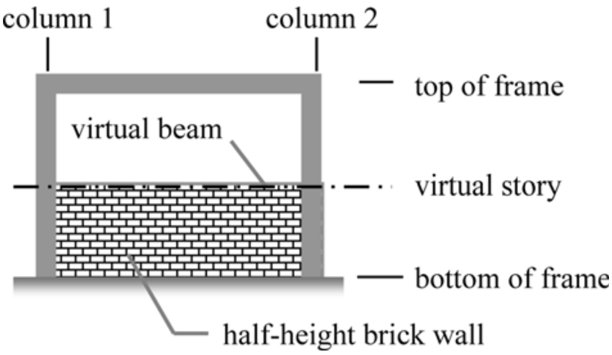


Fig. 13 Modeling of partially infilled frames

then a virtual beam element was assigned at this gridline to seal off the panel underneath, as shown in Fig. 13. With the masonry infill completely enclosed, a legitimate model was created for each of the frames.

To represent the hysteretic behaviors of the frames, a three-parameter "Park Model" (Park *et al.* 1987) was adopted in the analysis. This model incorporates nonlinear structural behaviors such as stiffness degradation, strength deterioration, non-symmetric response, slip-lock (pinching), and a trilinear monotonic envelope. In IDARC2D, the degrees of stiffness degradation, strength deterioration, and pinching effects of a structure are defined by parameters HC, HBD and HS, respectively. Typical values of these three parameters range from 1.0 to 10.0, 0.1 to 0.4, and 0.0 to 1.0, respectively, according to Valles *et al.* (1996). In the current analysis, values of the above parameters are obtained through a trial-and-error process for each frame and are listed in Table 1.

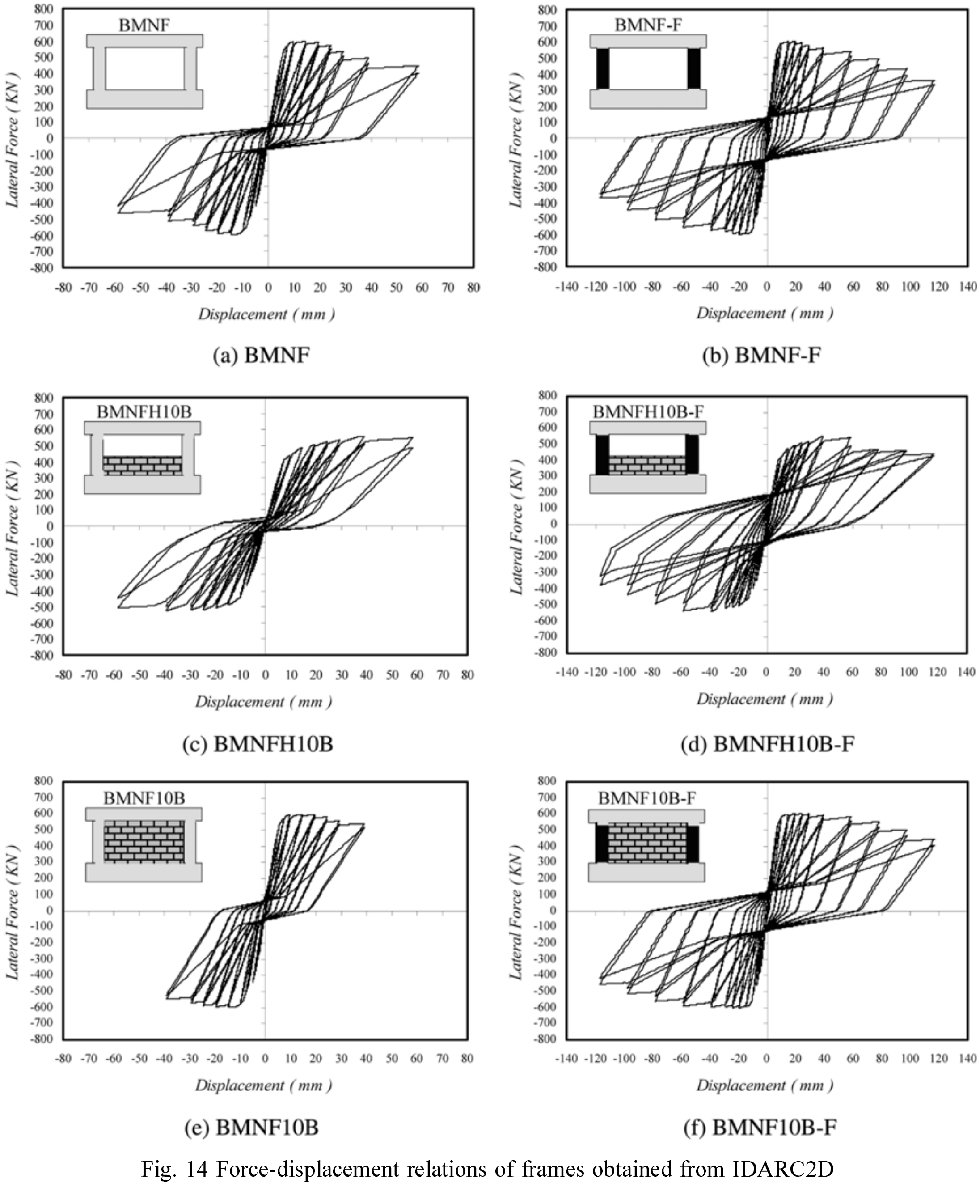
After the structural models were constructed, the displacement history given in Fig. 5 was applied at the top of each frame. The force-deformation relationships of these frames obtained from this analysis are given in Fig. 14.

Comparisons between the results of the analytical procedure and the experiment are given in Fig. 15. For frames whose failure was governed by flexure, IDARC2D provided a good representation of the nonlinear behavior of the frames. Figs. 15(b), 15(d), and 15(f) show that the force-deformation relationships obtained from this analysis successfully demonstrated the strength and stiffness degradations of frames BMNF-F, BMNFH10B-F, and BMNF10B-F in the inelastic regions with only limited errors in the maximum lateral strengths. For frames failed in shear, however, the program did not show the same kind of accuracy in the prediction of the strength degradation, as shown in Figs. 15(a), 15(c), and 15(e).

5. Conclusions

A quasi-static test has been performed on a series of concrete frames with or without masonry infills. The test results showed that the lateral strength and ductility of the frames were considerably improved when a full-height brick panel was constructed inside of the frames; however, similar improvements were not necessarily found in partially infilled units.

The test results also showed that the strength and ductility of nonductile concrete frames were substantially increased after being retrofitted with CFRP-jacketing. Load-deformation curves



(a) BMNF (b) BMNF-F

(c) BMNFH10B (d) BMNFH10B-F

(e) BMNF10B (f) BMNF10B-F

Fig. 14 Force-displacement relations of frames obtained from IDARC2D

demonstrated that the retrofitted frames were able to develop higher strengths than the unretrofitted units and conserve most of this strength after yielding. Consequently, it is concluded that CFRP-jacketing should improve the seismic resistibility of existing concrete structures if used properly.

As for the analytical procedure, it is shown that for retrofitted (ductile) frames, IDARC2D

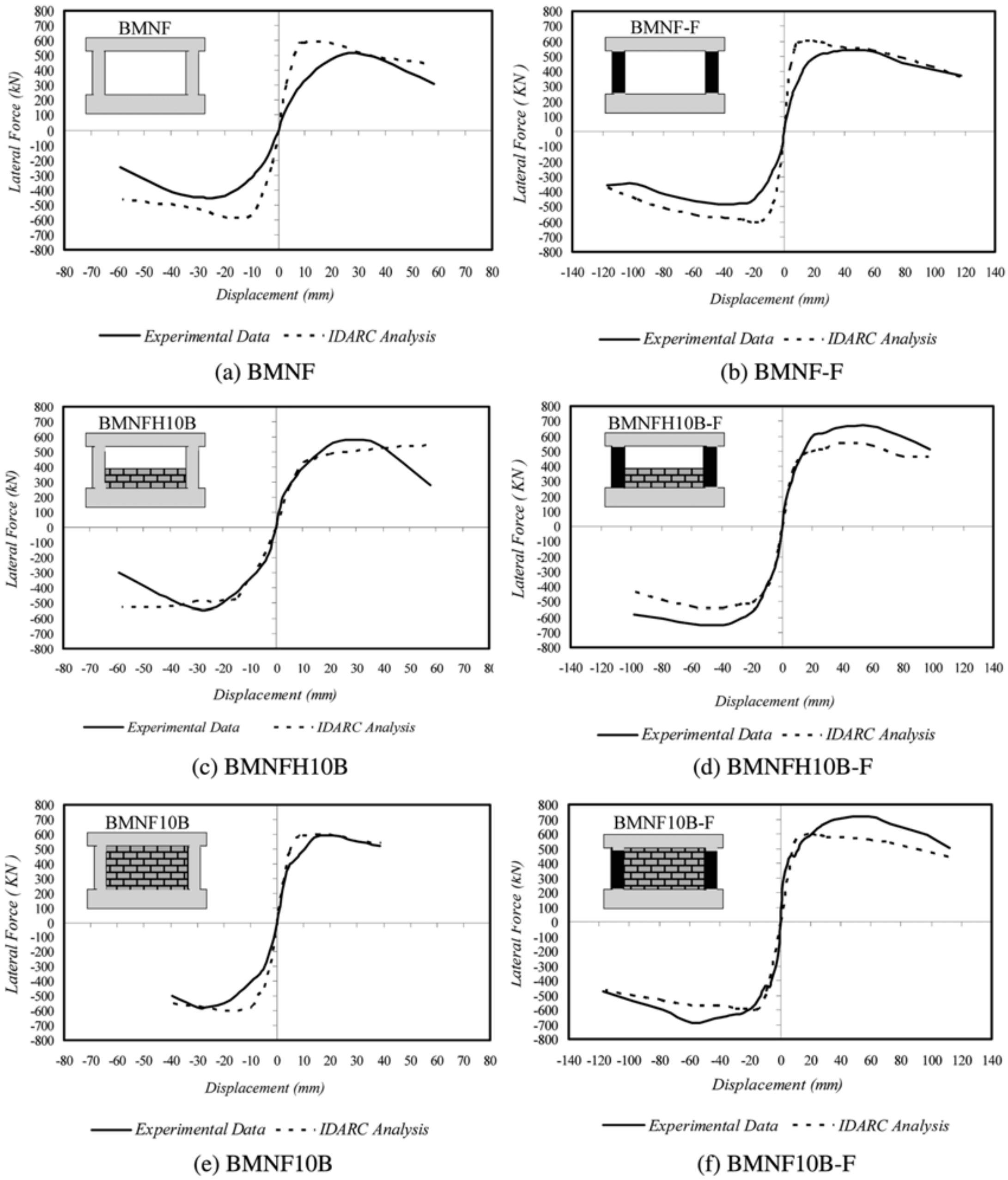


Fig. 15 Comparisons between analytical and experimental results

provided a good representation of the frame behavior even after yielding occurred. Nonetheless, this program was unable to display the shear mechanism in unretrofitted frames, and the precision in the analysis of partially infilled frames needed to be improved. Therefore, additional studies should be conducted on frames with perforated infills or with failure pattern governed by shear.

Acknowledgements

The research reported was funded by National Science Council of Taiwan under Grant # NSC-91-2625-Z-027-004. Test specimens and facilities were provided by the Taiwanese National Center for Research on Earthquake Engineering. The assistance and contribution by Dr. K. C. Chang from National Taiwan University was also acknowledged.

References

ACI Committee 318 (1995), *Building Code Requirements for Reinforced Concrete* (ACI 318-95) *and Commentary*, American Concrete Institute, Detroit, Michigan.

American Society of Civil Engineers (2000), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA Publication No. 356, Federal Emergency Management Agency, Washington, D.C.

Bertero, V.V. and Bokken, S. (1983), "Infills in seismic resistant buildings", *J. Struct. Eng.*, ASCE, **109**(6), 1337-1361.

Construction and Planning Agency (1999), *Seismic Design Code for Buildings*, CPA, Ministry of Interior, Taiwan.

Harajli, M.H. and Rteil, A.A. (2004), "Effect of confinement using fiber-reinforced polymer or fiber-reinforced concrete on seismic performance of gravity load-designed columns", *ACI Struct. J.*, **101**(1), 47-56.

Holmes, M. (1961), "Steel frames with brickwork and concrete infilling", *Proc.*, *Institution of Civil Engineers*, London, England, Part 2, **19**, 473-478.

Huang, C.H. and Tsai, C.H. (2003), "Seismic analysis and design of reinforced concrete frames with infill wall panels", Report No. NCREE-03-031, National Center for Research on Earthquake Engineering, Taipei, Taiwan.

Kahn, L.F. and Hanson, R.D. (1979), "Infilled walls for earthquake strengthening", *J. Struct. Div.*, ASCE, **105**(2), 283-296.

Klingner, R.E. and Bertero, V.V. (1978), "Earthquake resistance of infilled frames", *Proc.*, ASCE, **104**(ST6).

Li, Y.F., Lin, C.T. and Sung, Y.Y. (2003), "A constitutive model for concrete confined with carbon fiber reinforced plastics", *Mechanics of Materials*, **35**(6), 603-619.

Loh, C.H. and Tsai, C.Y. (2000), "Responses of earthquake engineering research community on the Chi-Chi earthquake", *Proc. of Canada-Taiwan Workshop on Natural Hazard Mitigation*, Ottawa, January.

Madan, A., Reinhorn, A.M., Mander, J.B. and Valles, R.E. (1997), "Modeling of masonry infill panels for structural analysis", *J. Struct. Eng.*, ASCE, **123**(10), 1295-1302.

Mehrabi, A.B., Shing, P.B., Schuller, M.P. and Noland, J.L. (1996), "Experimental evaluation of masonry-infilled RC frames", *J. Struct. Eng.*, ASCE, **122**(3), 228-237.

Nilson, A.H., Darwin, D. and Dolan, C.W. (2003), *Design of Concrete Structures*, 13th Ed., McGraw Hill, New York.

Park, R. and Paulay, T. (1975), *Reinforced Concrete Structures*, John Wiley and Sons, New York.

Park, Y.J., Reinhorn, A.M. and Kunnath, S.K. (1987), "IDARC: Inelastic damage analysis of reinforced concrete frame – shear wall structures", Technical Report NCEER-87-0008, National Center for Earthquake Engineering Research, State University of New York, Buffalo, New York.

Scott, B.D., Park, R. and Priestley, M.J.N. (1982), "Stress-strain behavior of concrete confined by overlapping hoops at low and high strain rates," *ACI J. Proc.*, **79**(1), 13-27.

Stafford Smith, B. and Carter, C. (1969), "A method of analysis for infilled frames", *Proc.*, *Institution of Civil Engineers*, London, England, **44**, 31-48.

Saneinejad, A. and Hobbs, B. (1995), "Inelastic design of infilled frames", *J. Struct. Eng.*, ASCE, **121**(4), 634-650.

Tsai, K.C., Hsiao, C.P. and Bruneau, M. (2000), "Overview of building damages in 921 Chi-Chi earthquake", *Earthquake Engineering and Engineering Seismology*, **2**(1), 93-108.

Valles, R.E., Reinhorn, A.M., Kunnath, S.K., Li, C. and Madan, A. (1996), "IDARC2D Version 4.0: A computer program for the inelastic damage analysis of buildings", Technical Report NCEER-96-0010, National Center for Earthquake Engineering Research, State University of New York, Buffalo, New York.