Transaction A: Civil Engineering
Vol. 17, No. 3, pp. 184–193

Shaking Table Study of a Full-Scale Single Storey Confined Brick Masonry Building

M.T. Kazemi[1,*], M. Hoseinzadeh Asl[1], A. Bakhshi[1] and F. Rahimzadeh Rofooei[1]

Abstract. *In order to evaluate the seismic behavior of confined masonry buildings, based on Iranian seismic code design, a single storey full-scale unreinforced confined brick masonry building has been constructed on the shaking table facility at the Earthquake Engineering Research Center (EERC) of Sharif University of Technology. The 4 by 4 meter model consists of four brick masonry walls confined with reinforced mortar tie-columns and steel bond beams, with a traditional jack-arch roof system. Three of the wall panels contained openings of different sizes and geometries. The model was subjected to the scaled earthquake records of Bam, Tabas and El Centro, as well as a harmonic acceleration with gradually increasing amplitude. The test results indicated that, for moderate strength earthquakes, the provisions provided by the Iranian seismic code is appropriate for life safety. Results also verified that proper workmanship of ties plays an important role in the integrity and stability of the unreinforced masonry buildings. Settlement of the masonry walls and subsequent reduction in frictional resistance between wall and roof horizontal steel bond beams were effective on the out-of-plane failure of the walls. Based on experimental observations, some suggestions are made to improve the current seismic code of Iran.*

Keywords: *Shaking table test; Confined masonry; Bond beam; Tie-column; Opening effect.*

INTRODUCTION

Historical and recent earthquakes of Iran (Tabas 1978, Manjil 1990 and Bam 2003) [1], as well as some neighboring countries, have caused a high number of casualties and injuries, vast damage in housing construction and significant losses in local and national economies [2]. Masonry buildings are widely constructed for housing in urban areas of Iran and in some other countries. These types of building basically consist of unreinforced masonry wall panels with or without confining elements [3]. The wall panels are usually made of clay brick and cement mortar. The confinement consists of vertical and horizontal confining elements usually of reinforced concrete, reinforced mortar or steel sections.

In recent years, considerable research has been carried out, both experimentally and analytically, in the area of masonry buildings. For instance, an experimental investigation aimed at determining seismic load capacity and reinforcement requirements for single-storey masonry dwellings in the United States has been conducted at the University of California, Berkeley [4]. The study included testing of four typical masonry houses, with both unreinforced and partially reinforced wall panels. In a similar way, results of a large experimental program carried out on twenty-four half scale models of two-storey masonry buildings have been presented by Benedetti et al. [5]. Dawe and Seah [6] investigated the behavior of masonry infilled steel frames experimentally by using large-scale specimens. They compared the experimental results with those of the three analytical models: the single degree-of-freedom model, the braced frame model and the equivalent strut model. In order to study and define failure criteria of unreinforced grouted brick masonry, a series of biaxial compression tests were performed by Badarloo et al. [7] on the full-scale brick specimens.

Some experimental research has been done to investigate the behavior of infills with openings [8,9]. A majority of the proposed analytical models have been verified for solid infill panels only. However, infills in most buildings have large window or door openings and a lack of knowledge on the behavior of infills with openings has led many designers to ignore such infills

1. *Department of Civil Engineering, Sharif University of Technology, Tehran, P.O. Box 11155-9313, Iran.*
*. *Corresponding author. E-mail: kazemi@sharif.edu*

Received 18 August 2009; received in revised form 9 March 2010; accepted 20 April 2010

when determining the seismic strength of the structural system.

Seismic loading, especially when there is little vertical axial pressure on the wall panels, has a significant effect on the out-of-plane seismic resistance of masonry buildings. The ABK Joint Venture [10] performed the most extensive series of dynamic tests on the out-of-plane behavior of unreinforced masonry walls in the early 1980s, and these remain the primary source of today's guidelines for seismic design and evaluation [FEMA 306 [11] and FEMA 356 [12]]. In another study, Griffith et al. [13] conducted a series of static and dynamic tests on 14 unreinforced confined masonry wall panels by subjecting them to out-of-plane forces. They concluded that displacement, rather than inertia force amplitude, determines whether an unreinforced masonry wall will collapse during inertial out-of-plane loading or not.

Despite more than three decades of research into this subject, there remain areas of masonry seismic behavior that have not been explored experimentally. The purpose of the research presented herein is to evaluate the seismic behavior of confined unreinforced masonry buildings, which are designed and constructed based on the Iranian Code of Practice for Seismic Resistance Design of Buildings [14], named Standard No. 2800 (S2800). Effects of in-plane and out-of-plane loadings, wall confinement and wall panel openings are investigated. For this purpose, a single storey large-scale brick masonry building specimen was constructed, in accordance with the Iranian seismic code, and tested on the shaking table of the Earthquake Engineering Research Center at Sharif University of Technology (EERC-Sharif).

TEST SETUP

Figure 1 shows the single-storey large-scale unreinforced confined brick masonry building that was constructed on the shaking table of EERC-Sharif [15]. The structural layout of the building conforms to the requirements of S2800. In order to follow the current construction method, local experienced masons were employed to build the whole structure. The building plan dimensions were approximately 3.4 × 3.4 m as shown in Figure 2 and the overall height of the building was 2.71 m.

WALLS

The wall panels had a thickness of 205 mm and were made of unreinforced clay brick with average dimensions of 200 × 100 × 50 mm, with 15 mm mortar thickness. All bricks were presoaked to decrease water absorption from mortar joints. Low strength mortar was used to mimic the weak materials that exist in many masonry buildings in stocktickerIran. In order to investigate the effect of coating, the south-side wall was coated with gypsum plaster, with an average thickness of 15 mm, making a wall thickness of 220 mm. Each wall panel had a height-to-thickness ratio of 10.8, which, excluding the plaster, approximates the largest permissible value of 10 as permitted in S2800.

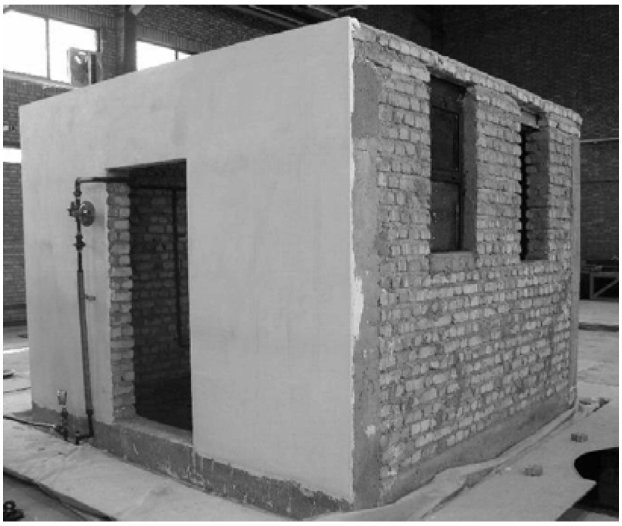


Figure 1. Confined masonry model on the shaking table.

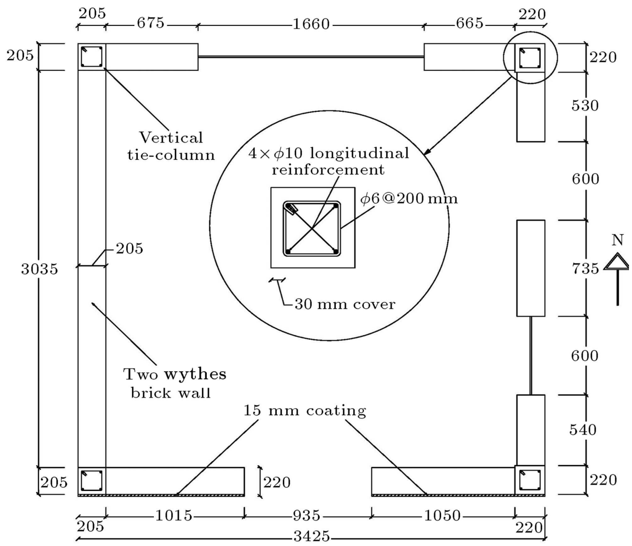


Figure 2. Plan of the building model (all dimensions in mm).

OPENINGS

Figure 3 shows the elevation views of the building with openings of different shapes and geometries. The geometry of the openings was based on the maximum allowable dimensions given in S2800. The south side panel included a 935 × 1820 mm door opening. The east side wall had two typical 1065 × 600 mm window openings, one with a steel window frame and the other

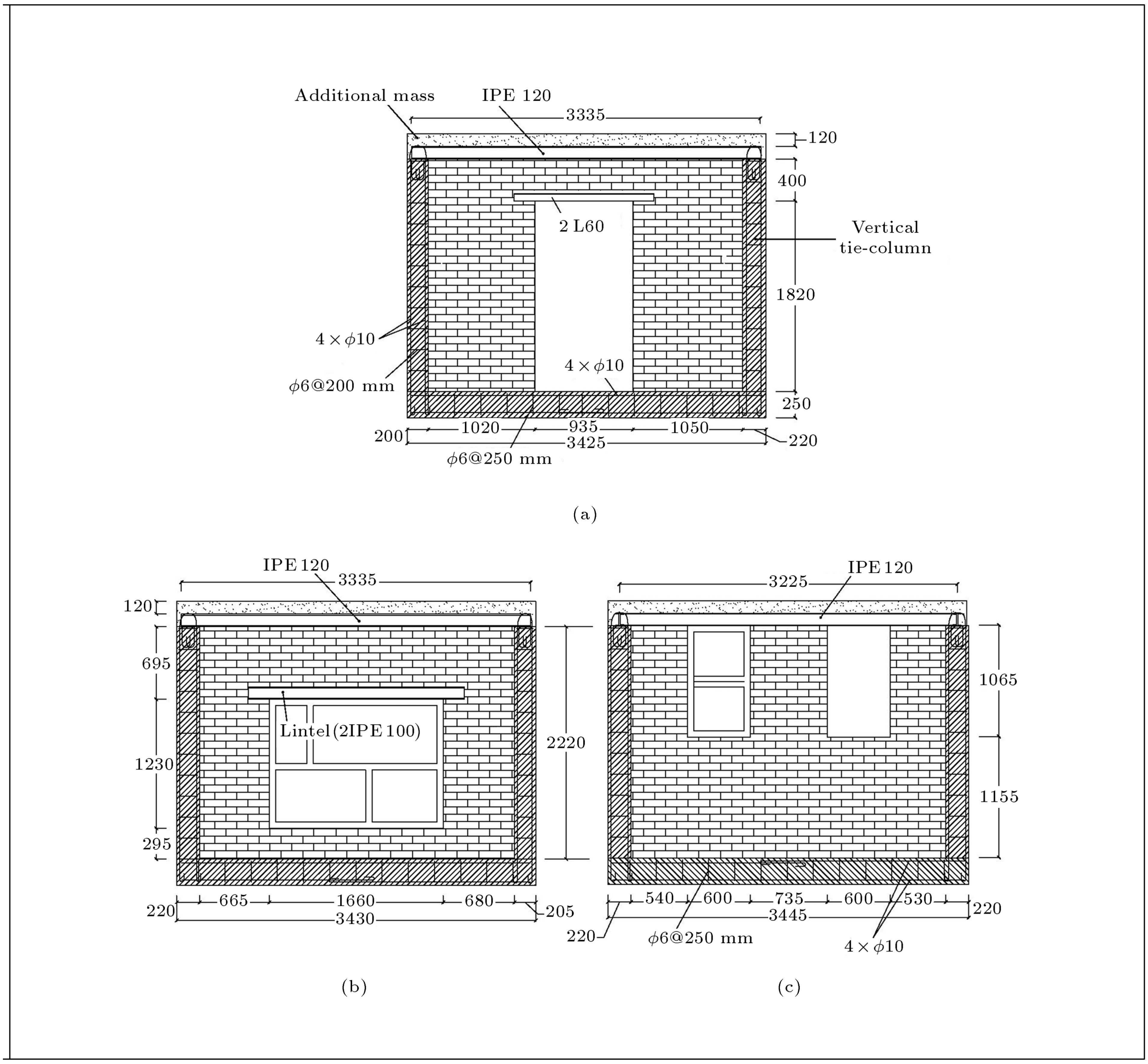


Figure 3. Elevation view of the model building. a) South wall, b) North wall and c) East wall.

without a window frame, to enable investigation of the effects of differing window frames on the wall response. The north side panel had the largest window opening (1660 $\times$ 1230 mm), with a steel window frame and a double IPE lintel placed above the window frame, extending 200 mm inside the wall at each end. The openings had no special concrete boundary elements.

CONFINEMENT

Wall panels were confined at building corners with reinforced mortar tie-columns; at the top, with steel bond beams of an IPE120 profile and at the bottom, with a R/C strip foundation. The tie-columns and foundation strips had cross-sections of 200 $\times$ 200 mm and 250$\times$250 mm, reinforced with four 10 mm diameter longitudinal bars tied with 6 mm diameter stirrups, and spaced at 200 mm and 250 mm, respectively. The dimensions and reinforcement were based on the minimum allowable values given in S2800. Tie-columns were constructed with the same mortar as used for the construction of walls. The tie-columns were well connected with the steel bond beams at roof level and were base-supported on the floor foundation. In order to connect the steel bond beams to vertical tie-columns, a steel base plate was embedded and anchored on the vertical tie-column by means of U-shaped 10 mm anchorage bars. Each end of a bond beam was welded to an anchor plate, and the extended longitudinal reinforcing bars of the tie-columns were bent and welded to these.

ROOF

The roof system consisted of four IPE120 beams filled with typical brick jack arches and covered with 120 mm thick debris and cement grouting to simulate the additional mass of the supported floor, as occurs in buildings. Figure 4 shows the roof framework. As emphasized in S2800, the beams were tied to one another by means of x-crossing steel bars to ensure the integrity and rigidity of the roof.

In order to secure the specimen to the shaking table, 16 high strength 5 mm diameter bolts, spaced at 800 mm under the wall, were affixed to the steel deck of the shaking table and were anchored in the concrete foundation.

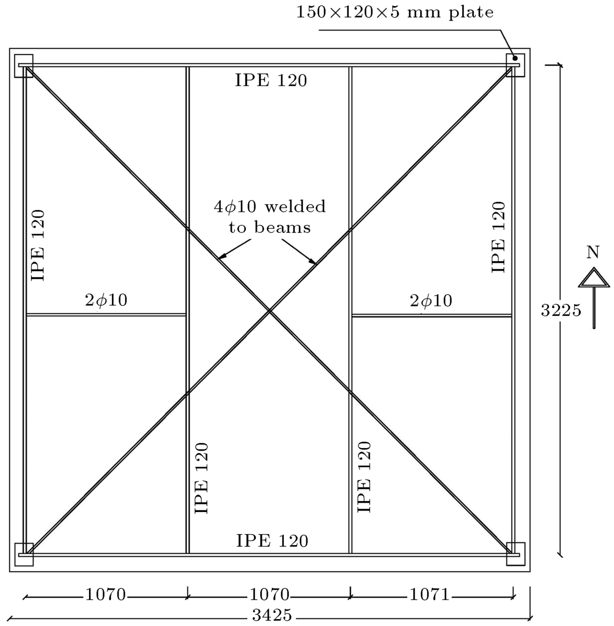


Figure 4. Roof framework.

MATERIAL PROPERTIES

A series of material tests was performed to determine the strength properties of mortar, brick, foundation concrete and reinforcements. Table 1 shows the mean strength of the materials used. The total mass of the model was estimated to be 14300 kg.

TEST SEQUENCES

The experimental program was divided into three phases, namely:

(i) Testing for dynamic characteristics of natural frequency and damping ratio.

(ii) Testing for response to earthquake-induced support motions.

(iii) Testing for response to near-resonance harmonic excitation, with increasing amplitude.

In the first phase of testing, the basic dynamic characteristics of the undamaged building were studied using devices that set the structure into low amplitude free vibration. The free-vibration tests were performed by using a rubber hammer to apply a small impulse force at the roof level. Standard calculations from the time history response indicated that the natural frequency of the structure was approximately 13.3 Hz and 12.6 Hz in

Table 1. Material properties.

Test	Dimensions (mm)	Specimen	Individual (MPa)	Mean
Comp. strength of brick units	205 × 100 × 70	1	5.9	6.4
		2	6.6	
		3	6.6	
Comp. strength of mortar	50 × 50 × 50 prism	1	3.8	3.6 (7 days)
		2	3.3	
		3	3.7	
Comp. strength of concrete	150 × 300 cylinder	1	18.4	19.3 (28 days)
		2	19.3	
		3	20.1	
Yield strength of bars	ϕ10	1	307	292
		2	277	
		3	292	
Ultimate tensile strength of bars	ϕ10	1	397	375
		2	352	
		3	375	

the north-south and east-west directions, respectively. The damping ratio was 0.0267 and 0.0247 in the north-south and east-west directions, respectively (mean values are presented). It is noted that frequencies differed mainly due to varied opening characteristics.

To investigate the wall response to transient excitation, shaking table tests were performed using earthquake accelerogram records to drive the shaking table. The shaking table test sequences are tabulated in Table 2. By comparing the peak ground displacement (PGD) and peak ground acceleration (PGA) given in Table 2, an indication of the type and severity of the earthquakes can be attained [16].

In the third phase, in order to study structural damage and collapse mechanisms, the structure was subjected to a gradually increasing bidirectional harmonic excitation with a maximum acceleration of 2 g and maximum amplitude of 5 mm. The imposed record, as shown in Figure 5, had a 105 sec duration and a frequency of 10 Hz, which was close to the fundamental frequency of the structure that had experienced light cracks under phase two excitations.

TEST OBSERVATIONS

In the second phase, cracks of limited length were initiated in the wall panels (Figure 6). These cracks were mostly observed in the corners of the openings and at the wall-to-roof intersections. Horizontal cracks at the construction joints in the tie columns were also observed. However, this type of crack did not have a significant effect on the seismic strength of the tie-columns. While the applied earthquakes of Table 2 did not initiate the collapse of the building or any severe damage, the opening and closing behavior of mortar joints was observed in corner areas of the east wall openings. In spite of poor material quality, mainly due to workmanship related to the reinforcement of tie columns and connections between columns and bond beams, the overall structural system suffered no collapse during the tests. This strongly suggests that for moderate strength earthquakes, even weakly enforced provisions of S2800, as tested herein for masonry buildings, are adequate for life safety.

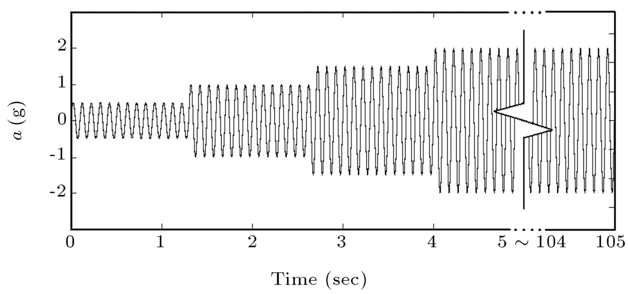


Figure 5. Time history of harmonic excitation imposed at the third stage.

Figure 6. Minor cracks around the corner of the opening of the north wall.

In the third phase, the building suffered considerable damage to the east and west walls and at the wall-to-roof intersections. The east wall had two openings extending up to the roof level with no lintel above. Figure 7 shows the east wall before and after the test. The south side of the east wall suffered less damage, in comparison to the north side of the wall. This was mainly due to the presence of a window frame in the south side, as seen in Figure 7. Figure 8 shows the wide open cracks, appearing at the corners of the door opening of the south side wall.

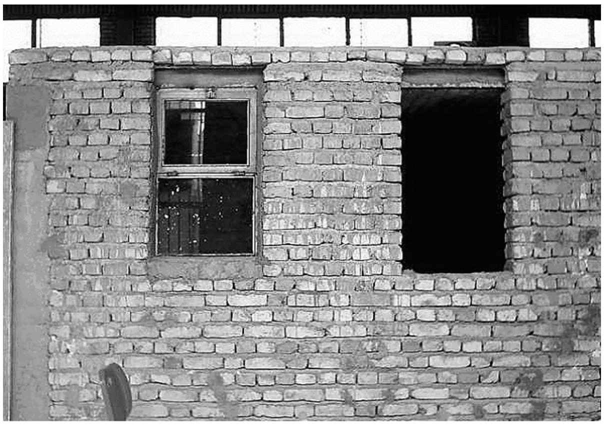
The specimen building was tested approximately ninety days following construction. During this period,

Table 2. Shaking table test sequences of the second stage.

Run	Input Motion	East-West Direction			South-North Direction		
		PGA (g)	PGD (mm)	S_{pa} (g)	PGA (g)	PGD (mm)	S_{pa} (g)
1	25% BAM 2003	0.2	86	0.34	0.16	51	0.53
2	10% TABAS 1978	0.09	95	0.21	0.08	37	0.19
3	50% EL. CENTRO 1940	0.16	54	0.41	-	-	-
4	50% BAM	0.40	171	0.67	0.32	101	1.06
5	20% TABAS	0.17	189	0.42	0.17	74	0.37
6	100% EL. CENTRO 1940	0.32	108	0.81	-	-	-

the masonry walls experienced vertical settlement due to the shrinkage of the mortar and creep. This resulted in gradual separation between the walls and the upper steel bond beams, and the consequent weakening of the connection between them. This resulted in reduced lateral support of the wall panels at these junctures, although such support is also partially dependent upon the amount of gravity load carried by the confining bond beam. Figure 9 shows the west and north side walls after the test. The upper part of the west wall collapsed under out of plane dynamic forces. As shown in Figure 4, roof joists were oriented in the north-south direction and the roof dead load was mainly transferred to the north and south side bond beams, generating more friction between the bond beams and the south and north walls. From Figure 9, it can be observed that the north side wall had suffered less damage at the top due to larger lateral frictional resistance.

Some local spalling of mortar cover was observed at the bottom of the columns, but buckling of longitudinal bars was not observed (Figure 10). Figure 11 shows the connection of the tie column and bond beam after the test. As can be seen in the figure, the surrounding bricks of the connection are fully crumbled, but the connection had not suffered severe damage and remained functioning properly.



(a)

(b)

Figure 7. East wall. a) Before the test; b) After the test (third stage).

Figure 9. North and west side walls after the third stage.

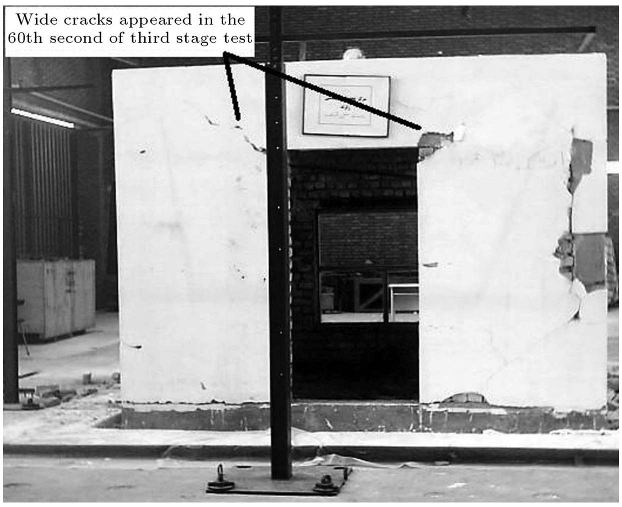


Figure 8. South side wall after the test (third stage).

Figure 10. Spalling of the tie-column.

Figure 11. The connection of a tie-column with bond beam.

ANALYTICAL STUDY

In an effort to gain additional insight into the behavior of the tested model, a general purpose finite element program was used to perform a series of linear dynamic and nonlinear static finite element analyses. Based on elastic dynamic analysis, and comparing the results with the measured first period of the structure, an equivalent value of modulus of elasticity equal to 1200 MPa was obtained for the masonry walls. The in-plane and out-of-plane behavior of confined masonry walls was evaluated. The effect of the additional constant gravity load of the roof and the effect of the window frame on the lateral load bearing capacity of the walls were investigated.

In general, the approach towards the numerical representation of masonry as a composite of bricks and mortar joints can focus on the micro-modeling of an assemblage of component bricks and mortar, or the macro-modeling of masonry as a composite. Macro modeling treats masonry as a homogeneous continuum. It is more practical due to the reduced time and memory requirements, and permits a user-friendly mesh generation. This type of modeling is most valuable when a compromise between accuracy and efficiency is needed. Masonry can be assumed to be a homogeneous material if a relationship between average stresses and strains in the composite material is established.

An isotropic nonlinear 3-D eight-node element capable of cracking in tension and crushing in compression was used to model the masonry (Figure 12). The presence of a crack at an integration point was represented through modification of the stress-strain relations by introducing a plane of weakness in a direction normal to the crack face. In addition, a shear transfer coefficient was introduced to represent a shear

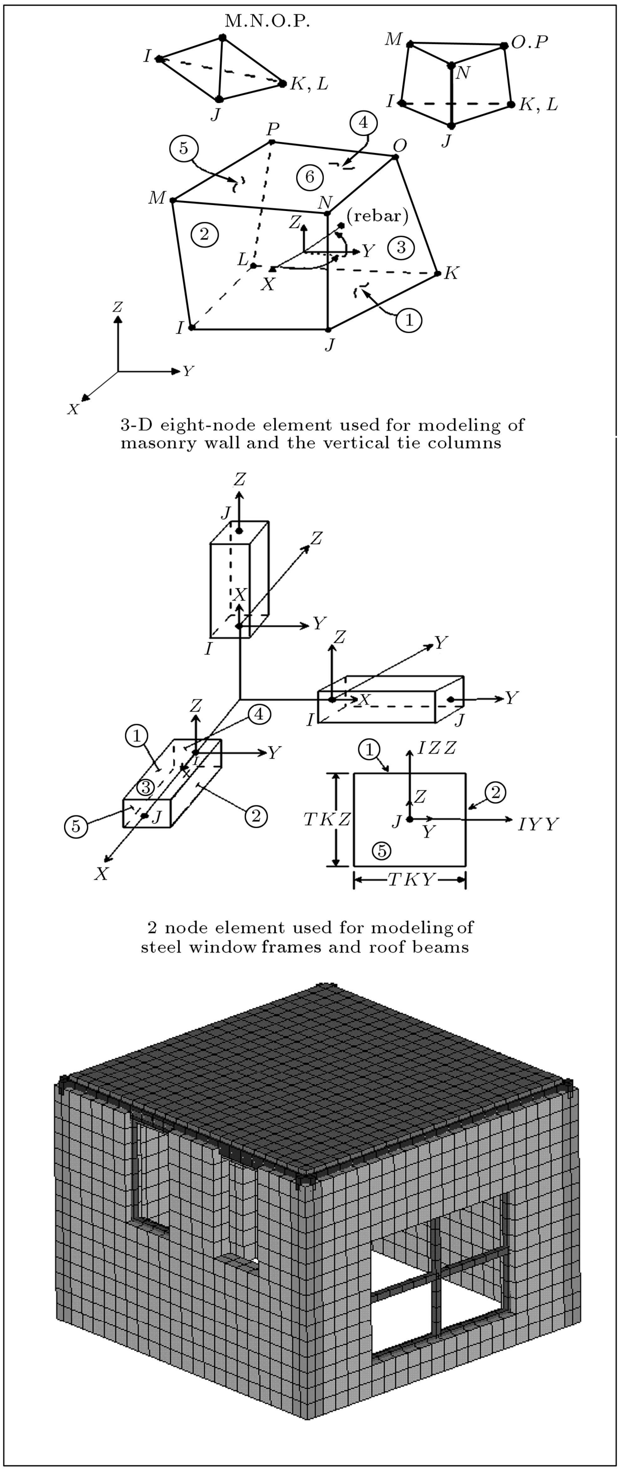


Figure 12. 3-D model of the structure and the finite elements.

strength reduction factor for those subsequent loads that induced shear sliding across the crack face. The cracking and crushing of the material were determined by a failure surface. When the failure surface is reached, stresses in that direction can be modified with an adaptive descent and a gradual drop to zero, to reduce convergence difficulties. The failure surface for compressive stresses was based on the Willam-Warnke [17] failure criterion, which depends on five material parameters defined on the basis of material properties. No stress softening in compression was implemented when crushing occurred.

The 3-D model of the structure is shown in Figure 12. In order to study the weak points of the structure, horizontal components of the El-Centro record was imposed onto the model. The predicted crack pattern is shown in Figure 13. The roof beam and shell elements are not shown. In the upper side of the east side wall, cracks are generated remarkably, which shows good agreement with the test results.

IN PLANE ANALYSIS

Figure 14 shows the finite element models used for analysis. Wall elements were confined by vertical tie column elements and steel I-section bond beam elements. Tie columns were modeled by the same element type as used for the wall elements, but with different material properties. The bond beam was modeled using simple two-node beam elements with six degrees of freedom, including three translations and three rotations at each node. The wall base nodes were fixed in all three degrees of freedom.

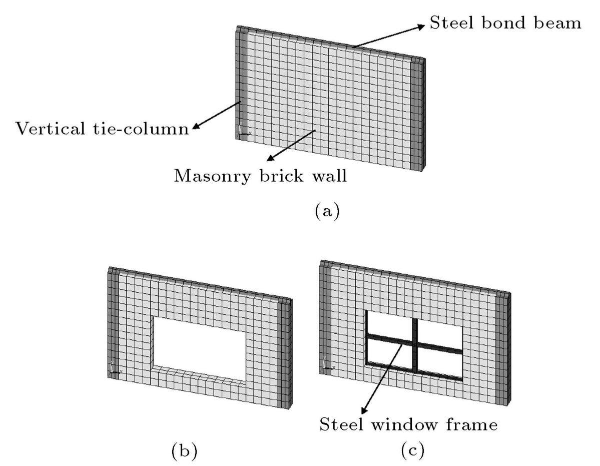


Figure 14. 2-D Finite element models, a) Full wall, b) Wall with an opening but no window frame and c) Wall with opening and with window frame.

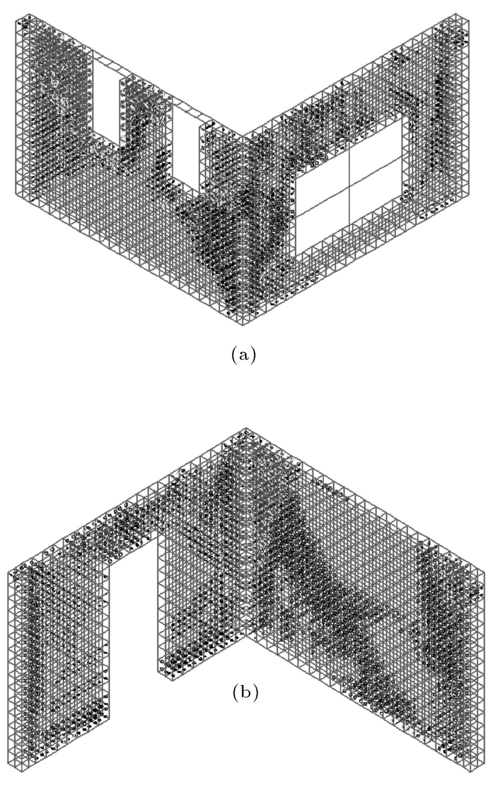


Figure 13. Crack patterns in 3-D analytical model under El Centro ground motion excitation. a) South and west walls; b) North and east walls.

A static nonlinear pushover analysis was used to study the models, which consisted of two load cases. Push 1 was gravity load and Push 2 was a continuation of Push 1, with a monotonic application by increasing in-plane horizontal displacement of the nodes of the upper bond beam elements. In in-plane analysis, upper nodes of the wall and the nodes of the bond beam were constrained to move together in a horizontal direction, so that the imposed displacement was directly transferred to the wall elements with no slippage permitted between the wall and beam elements.

In order to study the effect of the gravity load of the roof, which imposed an initial axial pressure on the wall panel, two identical models with different loadings were constructed. In the first model, in addition to self weight loads, a distributed gravity load with a magnitude of 5 kN/m was applied on the bond beam elements. In the second model, no additional load was applied besides the self weight of the wall. After applying gravity loads, a static nonlinear pushover analysis was performed to obtain the roof displacement versus base shear reaction of the wall. Figure 15 compares the load-displacement response of the two models. It is evident from the curves that vertical surcharge significantly improves the shear capacity of the wall. In the nonlinear region, after cracking, this effect is more evident.

In order to study the effect of window frames, two similar models, with and without window frames, were used (Figures 14b and 14c). Both models had a large opening in the middle of the wall, of the same size as the north wall opening in the experimental model.

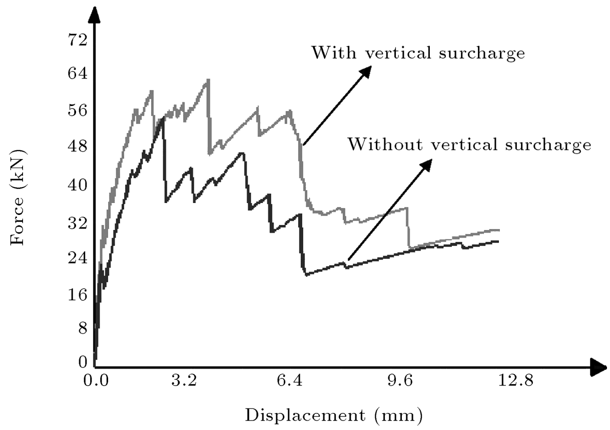


Figure 15. In plane load-displacement response of a full wall with and without vertical surcharge.

L60×60 × 6 cross-sections were used to model the window frame elements (Figure 14c). Figure 16 compares the load-displacement responses of two models. As can be seen in Figure 16, the presence of the window frame does not have a significant effect on cracking strength and initial stiffness, but can improve wall behavior in the post-cracking region and can result in an increase of wall ductility.

OUT OF PLANE ANALYSIS

One of the main failure mechanisms of masonry buildings is out-of-plane failure [10]. Vertical surcharge on the wall is one of the parameters that can significantly affect the out-of-plane resistance of masonry walls. In order to study this effect, two identical models, with and without vertical surcharge, were used. The models were similar to those used in in-plane analysis. It was assumed that the out-of-plane displacement of confining tie columns and the bond beam are constrained by the roof and side walls. Beside the base of the wall, it is assumed that the all confining bond beams and tie-columns are fixed in an out of plane direction. Out of plane slippage between an upper steel bond beam and wall elements was taken into consideration by defining contact elements at the interface. The coefficient of friction between the wall and bond beam was assumed to be 0.2. Based on experimental observations, no slippage was assumed between the wall and tie column elements. Similar to the in-plane nonlinear pushover analysis described in the previous section, a static nonlinear pushover analysis was done by applying a monotonically increasing uniform body force to wall elements in an out-of-plane direction. The pushover curve was obtained by plotting the displacement of the middle of the wall versus applied body force, as force per unit mass of the wall in the out-of-plane direction. Figure 17 compares the load-displacement response of two models. Both models have equal stiffness in the elastic region before cracking occurs. It is also evident that vertical surcharge has little effect on the cracking strength of the wall. In the post cracking region, vertical surcharge considerably improves the wall strength and significantly increases the failure strength of the wall, up to 67 percent. It is observed that by proper modeling of masonry structures, their seismic performance may be predicted appropriately.

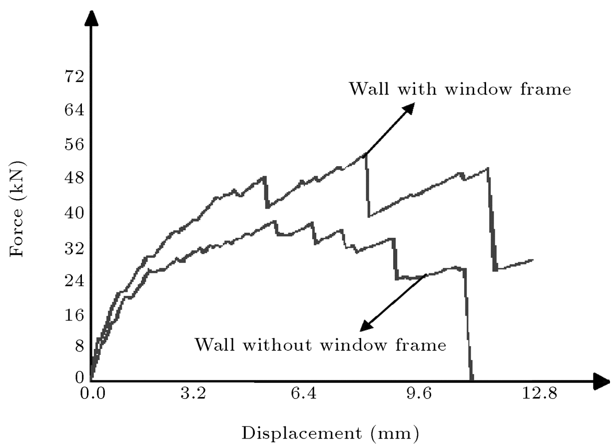


Figure 16. In plane load-displacement response of a wall panel with and without window frame.

CONCLUSION

The test results showed that for moderate strength earthquakes, the provisions given by the Iranian seismic code for confined masonry buildings satisfies the life safety criterion. Proper workmanship of the reinforcements of the tie columns and, especially, proper connections between vertical tie columns and horizontal bond beams has an important effect on stability and the collapse prevention of confined masonry buildings.

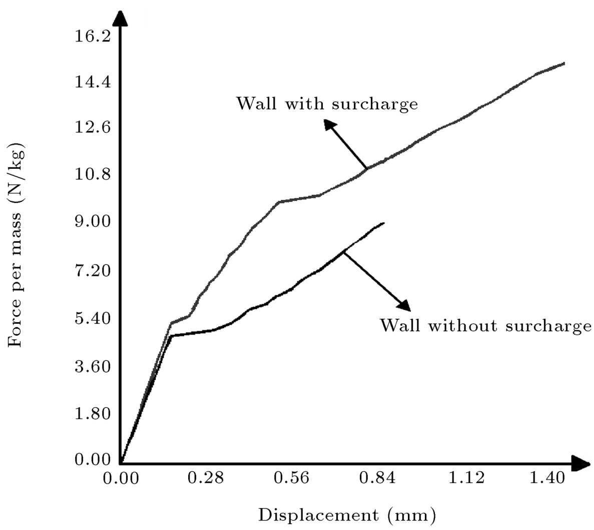


Figure 17. Out of plane load-displacement response of a wall panel with and without vertical surcharge.

Gradual separation between the walls and upper steel bond beams resulting from the creep and shrinkage of masonry walls must be considered in the design. Especially when there is little vertical surcharge on bond beams to strengthen the wall-beam connections, it is necessary to use shear connectors between the walls and bond beams to mitigate the out-of-plane failure of the walls. It is likely that reinforced concrete bond beams will have better integration with masonry walls. The use of properly anchored steel window frames in the masonry wall openings plays an important role in maintaining post-cracking wall integrity. An appropriate finite element simulation may also predict the seismic performance of the masonry structures property.

REFERENCES

1. Rahimzadeh Rofooei, F. "The Bam earthquake: the facts and the lessons learnt", *First Int. Conf. on Urban Earthquake Engineering*, Tokyo, Japan (2004).
2. Lee, W.H.K., Kanamori, H., Jennings, P.C. and Kisslinger, C., *International Handbook of Earthquake & Engineering Seismology*, Academic Press (2003).
3. Bakhshi, A., Bozorgnia, Y., Ghannad, M.A., Khosravifar, A., Mousavi Eshkiki, S.E., Rahimzadeh Rofooei, F. and Taheri Behbahani, A. "Seismic vulnerability of traditional houses in Iran", *1st Int. Conf. on Seismic Adobe Structures*, Lima, Peru (2005).
4. UCB/EERC. "Shaking table study of single-story masonry houses", UCB/EERC 79/25 (1979).
5. Benedetti, D., Carydis, P. and Pezzoli, P. "Shaking table tests on 24 simple masonry buildings", *Earthquake Engineering and Structural Dynamics*, **27**(1), pp. 67-90 (1998).
6. Dawe, J.L. and Seah, C.K. "Behaviour of masonry infilled steel frames", *Can. J. of Civil Engineering*, **16**(6), pp. 865-876 (1989).
7. Badarloo, B., Tasnimi, A.A. and Mohammadi, M.S. "Failure criteria of unreinforced grouted brick masonry based on biaxial compression test", *Scientia Iranica, Trans. A*, **16**(6), pp. 502-511 (2009).
8. Chiou, Y.J., Tzeng, J.C. and Liou, Y.W. "Experimental and analytical study of masonry infilled frames", *J. of Structural Engineering*, **125**(10), pp. 1109-1117 (1999).
9. Schneider, S.P., Zagerz, B.R. and Abrams, D.P. "Lateral strength of steel frames with masonry infills having large openings", *J. of Structural Engineering*, **124**(8), pp. 896-904 (1998).
10. ABK "Methodology for mitigation of seismic hazards in existing unreinforced masonry buildings: wall testing, out-of-plane", *ABK Joint Venture*, Topical Report 04, California (1981).
11. FEMA 306 "Evaluation of earthquake damaged concrete and masonry wall buildings, basic procedures manual", FEMA 306, The Partnership for Response and Recovery, Washington DC, USA (2000).
12. FEMA 356 "Prestandard and commentary for the seismic rehabilitation of buildings", FEMA 356, Washington DC, USA (2000).
13. Griffith, M.C., Lam, N.T.K., Wilson, J.L. and Doherty, K. "Experimental investigation of unreinforced brick masonry walls in flexure", *J. of Structural Engineering*, **130**(3), pp. 423-432 (2004).
14. Building and Housing Research Center, *Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard 2800)*, 2nd Ed., BHRC Publications (1999).
15. http://sina.sharif.edu/~civilinfo/Earthquake-center/H.htm
16. Bakhshi, A., Tavallali, H. and Karimi, K. "Structural damage estimation using seismic and spectral parameters", *Proceedings of the Tenth Int. Conf. on Civil, Structural and Environmental Engineering Computing*, Civil-Comp Press (2005).
17. Willam, K.J. and Warnke, E.D. "Constitutive model for the triaxial behavior of concrete", *Int. Association for Bridge and Structural Engineering: ISMES*, Bergamo, Italy, **19** (1975).

BIOGRAPHIES

Mohammad Taghi Kazemi is an Associate Professor in the Department of Civil Engineering at Sharif University of Technology in Tehran, Iran. He received his PhD from Northwestern University, Evanston, Illinois, USA. His research interests include the Seismic Rehabilitation of Existing Structures and Fracture Mechanics.

Masoud Hoseinzadeh Asl is a PhD student in the Department of Civil Engineering at Sharif University of Technology in Tehran, Iran, where he also received his MS. His research interests include Earthquake Engineering, Masonry Buildings and Non-Linear Analysis.

Ali Bakhshi is an Assistant Professor in the Department of Civil Engineering at Sharif University of Technology in Tehran, Iran. He received his PhD from Hiroshima University in Japan. His research interests include Earthquake Engineering, Seismic Risk and Hazard Analyses, Structural Control and Base Isolation Systems.

Fayaz Rahimzadeh Rofooei is a Professor in the Department of Civil Engineering at Sharif University of Technology in Tehran, Iran. He received his PhD from Rensselaer Polytechnic Institute, Troy, USA. His research interests include Active and Passive Structural Control Systems, Nonlinear Structural Analysis, Base Isolation, Seismic Vulnerability Analysis and Retrofitting of Structures and Lifelines.