Testing of confined masonry walls with different connection details

V. Sigmund & Dj. Matosevic
University of Osijek, Faculty of Civil Engineering, Osijek, Croatia

15 WCEE
LISBOA 2012

SUMMARY:
According to the Eurocodes horizontal and vertical confining elements should be bonded together and anchored to masonry by manner of construction (toothing) or by mechanical connectors (dowels). Both methods cause problems during construction and it is a common practice, at least in Eastern Europe, in trying to avoid them. The authors studied the influence of three different types of connection details between the masonry panel and r/c tie-columns on the resistance and displacement capacity of confined masonry walls within a scope of the Croatian project "Seismic design of infilled frames". Confining elements around the masonry wall increased stiffness and lateral load capacity. Connection among the masonry wall and ties increased ductile behaviour of the confined masonry. Existing equations for calculation of the lateral strength in EC6 are either under- or over estimating the observed values.

Keywords: confined masonry, connection details, seismis design

1. INTRODUCTION

Throughout the last centuries masonry structures were constructed according to generally accepted rules and experience. The modern way of design concept must include specific verification and computational proofs of load bearing capacity and serviceability of structures and masonry structures are not exception. However, inhomogeneous medium, i.e., the composite character of the masonry and its poor ductility considerably complicate the application of generally accepted numerical methods. Those were developed for construction materials and structures having definite elastic and plastic properties. Masonry mechanical characteristics are closely related to the skill of construction workers and execution control. In confined masonry structures, masonry wall is surrounded by concrete ties that improve the masonry behaviour. Joint connection details between masonry and ties have to be designed to ensure their common action during an earthquake.
According to the Eurocode 6 horizontal and vertical confining elements should be bonded together and anchored to masonry by manner of construction (toothing) or by mechanical connectors (dowels). Both methods cause problems during construction and it is a common practice, at least in East Europe, in trying to avoid them. Within this study we tried to determine the influence of connection details to the behaviour of confined-masonry walls exposed to constant vertical and cyclic horizontal loading. The required size and distribution of shear connectors on the masonry-tie connection should be determined. Therefore, we have tested confined masonry wall one-story one-bay specimens with three different connection details. production and testing conditions were the same and differences came from detailing. For statistical reasons we tested three specimens of each connection detail. The behaviour of confined panels with smooth connection at the masonry - concrete interface, smooth connection with steel dowels in the bed joints and traditional tooth type joints were compared to the behaviour of control unreinforced masonry wall of the same dimensions. It has been found that connection type did not influence the initial lateral stiffness and resistance of specimens, but significantly influenced the displacement capacity of confined panels.

1.1. Test specimens

The prototype wall represents a wall in residential confined masonry building at the ground floor with 25 m^2 of attributed floor area. The wall was designed according to the Eurocodes 6 (EC6) and 8 (EC8) and scaled down to 1:1,5 on the basis of a true model that maintains complete similarity implying that the prototype and model (specimens) have the same material properties. Three different types of models were made (with three specimens for each type: (A) masonry wall ends in a vertical line and there is no additional connection between the masonry and tie-columns except adhesion and mortar joints; (B) masonry wall ends in a toothed manner so that masonry and concrete tie interlock; (C) masonry wall ends in a vertical line and connection between the masonry and tie-column is obtained by U-shaped stirrups (dowels). Additional Model (D) was made as unreinforced masonry wall without tie-columns. Geometry of prototypes and specimens are presented in table 1.1.

Table1.1. Geometric data for prototypes and specimens

Model Type	Description of masonry – vertical concrete tie connection	Prototype l/h/t (m)	Specimen l/h/t (m)	Number of specimens	Designation of specimen
A	No			3	A1,A2,A3
B	Toothed	2,16/2,48/0,28	1,44/1,65/0,19	3	B1,B2,B3
C	U-shaped dowels			3	C1,C2,C3
D	No vertical ties			1	D

The specimens were built by standard local materials: hollow clay bricks V-5 with dimensions b/h/t=25/19/19 (cm) and with declared properties for compression strength 15 MPa and volume weight of 7,9 $kN/m^{3.}$ Tie-column concrete was C30/37 (obtained f_{ck}=35,6MPa), mixed mortar was made "in situ" in volume proportion of cement:lime:sand=1:1:5 and with designed nominal strength of 5MPa. Heights of the masonry units in specimens were scaled in order to ensure an equal number of bed joints in the specimen and prototype and they were l/h/t=25/13/19 (cm). Yield stress of the longitudinal and transverse steel was f_y =515 MPa.

1.2. Testing setup

For each different interface connection type (A,B and C) three specimens were made (nine specimens altogether). One masonry wall specimen (D) was tested for the reasons of comparison. The testing equipment consisted of a steel frame anchored to the strong floor and horizontally supported.

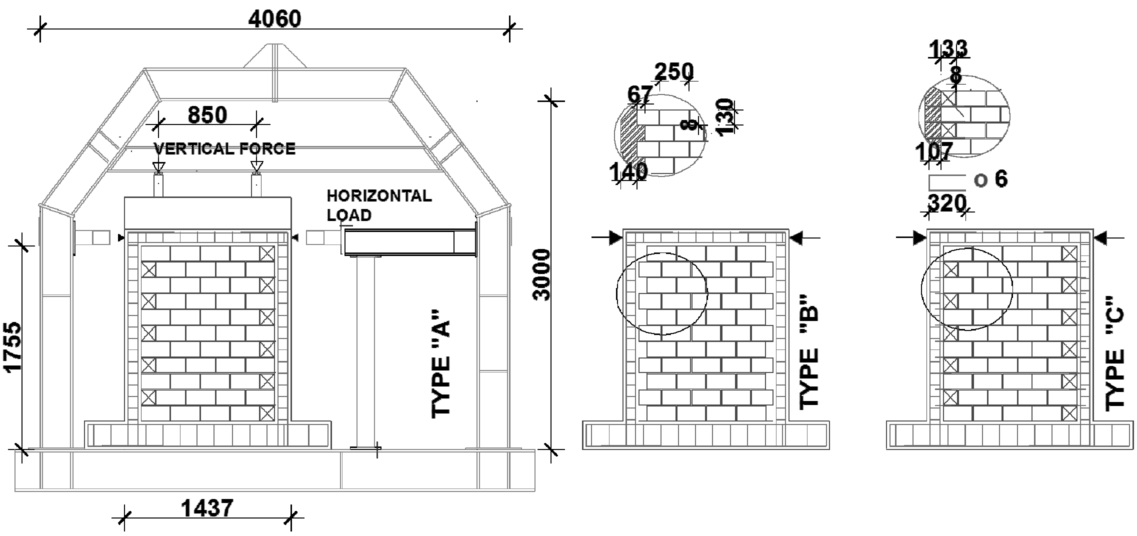


Figure 1.1. Test equipment and three different types of specimens

Four hydraulic actuators were fixed to the frame in order to simulate constant vertical and in plane cyclic lateral loads. The vertical load was applied on the specimen by means of two hydraulic jacks with 500kN capacity placed on a carriage that enabled them to move horizontally. Design vertical load was applied over the reinforced concrete beam b/h=27/28cm placed at the top of the wall over a thin teflon layer in order to evenly distribute the pressure. The constant vertical load, corresponding to the axial stress in the wall of 0.49MPa, was applied during the whole test. It was kept constant, as much as possible, by means of servo-valves mounted on the jacks. The lateral load was applied to the specimen by double-acting hydraulic jacks with 350kN capacity, placed laterally and fixed to the testing frame.
The cyclical lateral forces were applied in a horizontal direction at the top and in plane of the wall. Each test was first conducted under lateral load control at an increment rate of 10kN and then changed to the lateral displacement control when resistant forces start to decline. Each loading cycle was repeated twice. Loading time history is presented in Figure 1.2.

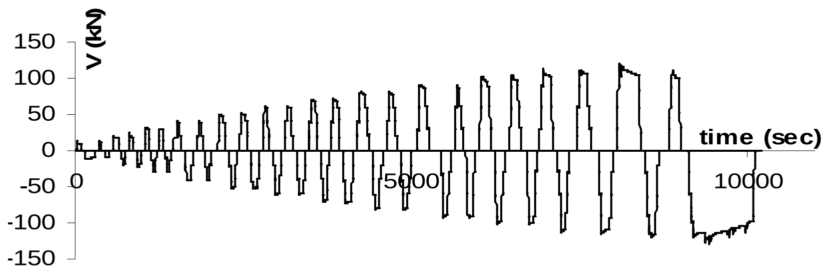


Figure 1.2. Time history of the horizontal force for the specimen B3

The test was ended when the load-deflection curve showed a drop in load to about 80% of the peak lateral load as the lateral displacement increased. The specimens were instrumented to monitor the applied loads at each loading point, horizontal and vertical displacements at the wall ends, horizontal slippage of the foundation beam, diagonal displacements in both directions and strains along the masonry-tie interface. Applied loads and all other measurements were continuously and automatically

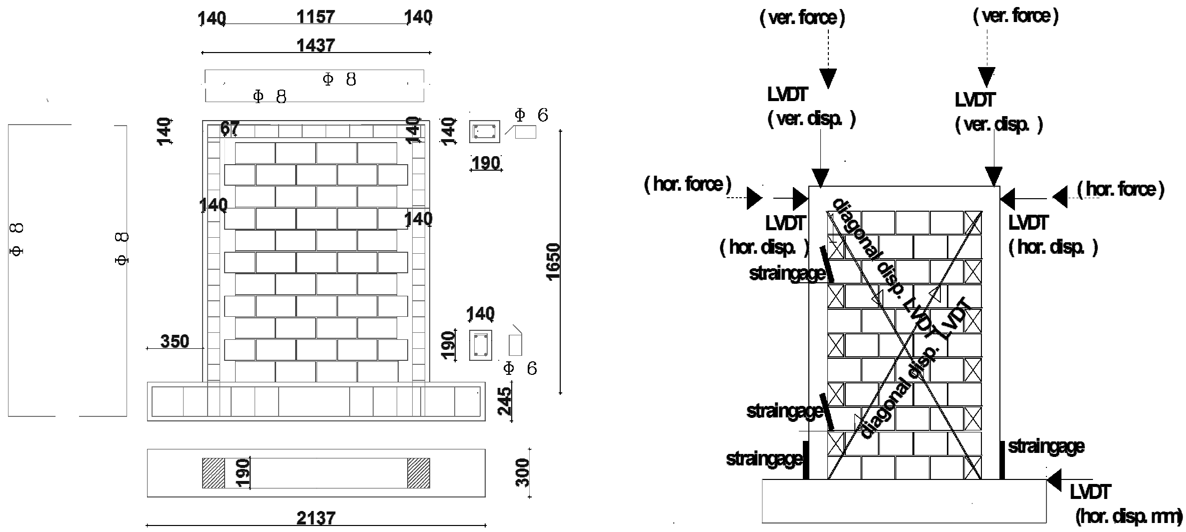


Figure 1.3 Outline of test specimen and measured values

scanned and recorded on the hard disk for later analysis by means of a computerized data acquisition system Dewe-daqbook. An outline of test specimens and the scheme of acquired data during testing are presented in Figure 1.3.

2. TEST RESULTS

2.1. Failure type

All specimens behaved in a similar way in general, with following stages: the cracks in masonry occurred initially at the wall corners, a pattern of diagonally oriented cracks appeared and continually increased, sporadic spaling of the masonry cover shells, extension of cracks from masonry into ties and tie-joints, extensive inner- and outer cracking of the masonry units that led to collapse. There is a slight difference between the stage's occurrence time and especially between the final crack patterns. Specimens (B) and (C) had more small cracks and the main diagonal cracks were not so much pronounced. Diagonal cracks protruded from masonry into the tie-joints. Cracking pattern in (A) type specimens had significant diagonal cracks and many horizontal cracks in tie-columns that indicated its tension failure. Measured hysteresis loops were mostly wide cycles for all three types of specimens. Damage patterns and hysteresis loops for each type of connection (A,B,C) are presented in Figures 2.1 and 2.2. The failure type of all three specimen types could be classified as hybrid (mix of diagonal shear and in-plane moment failure type).

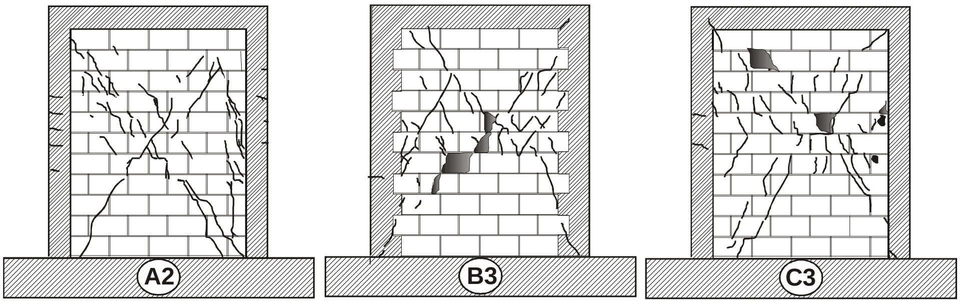


Figure 2.1 Final crack patterns

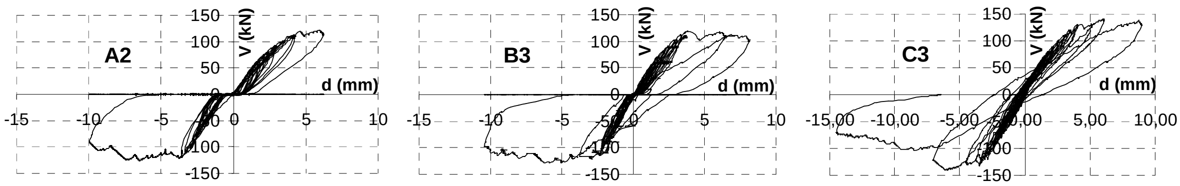
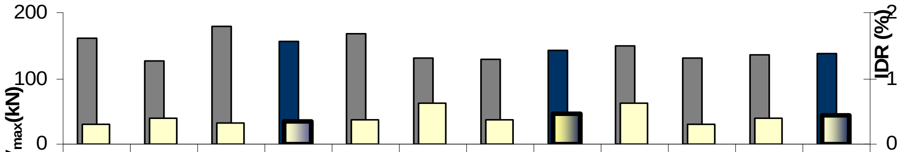


Figure 2.2 Hysteresis loops

Hysteresis loops of the specimens A, B and C initially look alike. After opening of the first large cracks specimen A degraded so fast so that we were not able to load it cyclically afterwards. It behaved in a brittle manner that can be observed also in Figure 2.4. Even after opening of the first large cracks specimens B and C were able to take up new loading cycles with degraded stiffness and fat hysteresis loops. Load capacity of all three specimens did not fall off suddenly.

2.2. Lateral strength

Measured maximum lateral strength, Vmax, for all specimens are presented in Figure 2.3. It is obvious that V_{max} showed no significant dependence on the connection type. The measured average values of Vmax for all specimen series (1,2,and 3) and for all three specimen types A, B and C were almost the same although they occurred at significantly different horizontal displacements (IDR of 0,34%, 0,45% and 0,43% for A,B and C respectively). The measured average hysteresis for each specimen type, are compared in Figure 2.4.



	A1	A2	A3	A mean	B1	B2	B3	B mean	C1	C2	C3	C mean
Vmax (kN)	161,147	125,34	179,6	155,362	167,394	130,74	128,60	142,245	148,667	131,34	136,425	138,811
dVmax (mm)	4,80	6,52	5,50	5,61	6,04	10,21	6,21	7,49	10,19	5,07	6,28	7,18
IDR (%)	0,29	0,39	0,33	0,34	0,37	0,62	0,38	0,45	0,62	0,31	0,38	0,43

Figure 2.3 Comparison of the maximum horizontal resistance force, models A1-C3

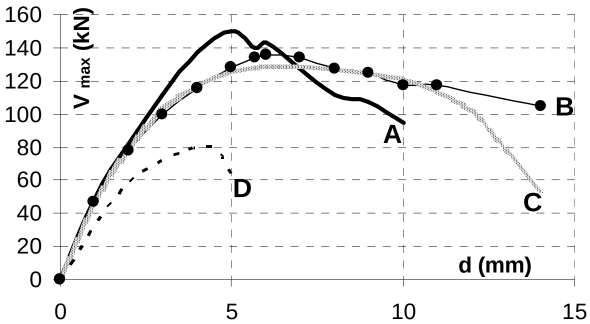


Figure 2.4 Average measured resistant force for series A,B and C

The lateral strength of confined masonry wall panel was calculated by three different equations and the results were compared to the measured values. According to the EC 6 calculated were V_{max} and pure bending moment resistance M_u with respective V_M . From that the failure occurred due to Moment capacity and that was underestimated while the calculated Vmax was overestimated. Other two approaches proposed by Tomažević (1999) and Aničić (1990) were closer to the measured values. Complete results are shown in Table 2.1.

Table 2.1 Lateral resistance verification

	V_{max} (kN)	M_{Rd} (kNm)	Related V_M (kN)	V_{Rd} (kN) experimental (type A)		Deviation (%)
				V_{max}	V_u	$(V_u - V_{rd})/V_u$
EC6	186,57	130,6	80	155,36	149,32	-25%
Aničić	129,5	-	-			13%
Tomaževič	128,3	-	-			14%

2.3. Idealization of experimental results

Measured hysteresis envelopes (primary curves) of all specimens was simplified by a bilinear curve according to Tomaževič (1999). Ultimate resistance force, V_u, was evaluated from the condition of equal energy dissipation of an actual and idealised wall panel. Both loading cycles, positive and negative, were considered for evaluating the maximum lateral force and its degradation. Effective stiffness, K_e , was evaluated according to Frumento (2009.) as secant of the experimental hysteresis envelope at a base-shear value of $0.7 {*} V_{max}$. The ultimate displacement, d_u , of the wall corresponds to displacement at which base shear decreased to $0{,}8 {*} V_{max}$. Obtained hysteresis envelope curve and associated bilinear idealisation for three different specimens types are given in Figure 2.5.

The characteristic values of the hysteresis envelope curves and for corresponding bilinear curves of all tested specimens are given in Table 2.2. From the bilinear idealizations could be observed that cracking of all specimens occurred at IDR (inter story drift ratio) of 0,17-0,19 % and the shear forces at cracking, V_{cr}, decreased from specimens A to B to C; ultimate base shears, $V_{u,}$ decreased from specimens A to B to C but the ultimate story drift (Inter story drift ratio=d_u /h %) increased from specimens A to B to C; the ultimate ductility μ_u increase from Type A (1,84) to Type B (2,36) and to Type C (3,14). Ultimate ductility factor corresponding to walls type C is 70% greater than ductility ratio of the specimens type A.

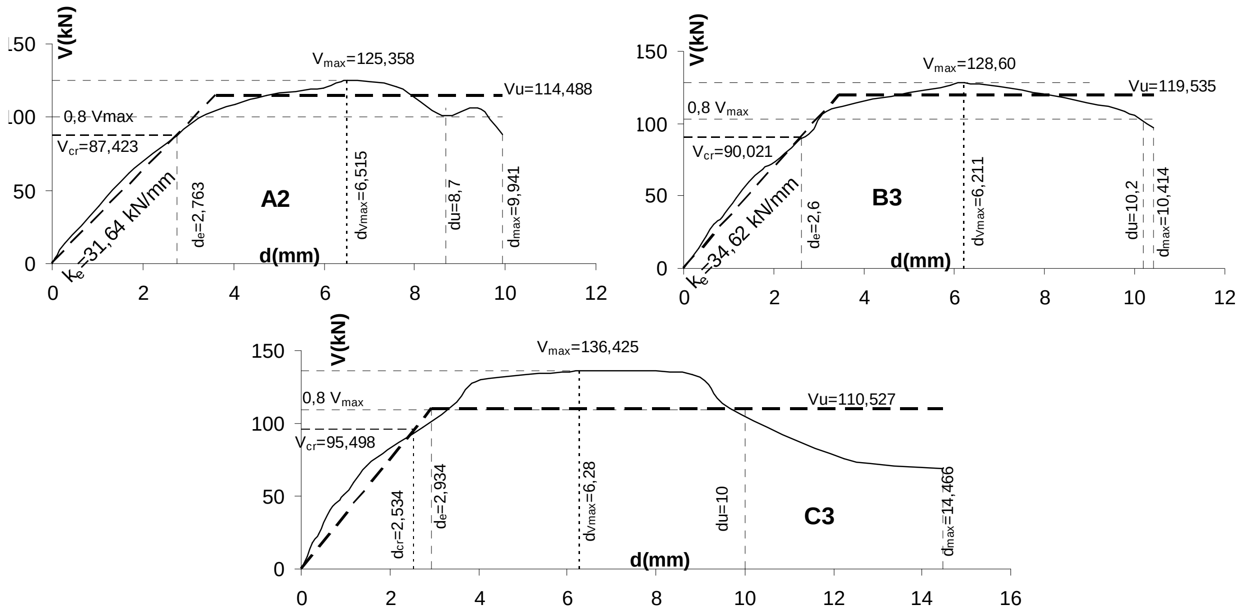


Figure 2.5 Experimentally obtained and idealized hysteresis envelopes for one specimen of series A,B and C

Table 2.2 Evaluated parameters of experimental hysteresis envelope

Specimen	Ke	V_{cr} (kN)	V_u	d_{cr} (mm)	d_e	d_{max}	d_u	crack-story drift d_{cr}/h %	ult. story drift d_u/h %	ultimate ductility $\mu_u=d_u/d_e$
	(kN/mm)		(kN)		(mm)					
A1	40,74	112,80	155,86	2,76	3,82	5,70	5,70	0,17	0,35	1,49
A2	31,64	87,42	114,49	2,76	3,62	9,94	8,70	0,17	0,53	2,40
A3	39,33	125,01	177,62	3,20	4,52	7,49	7,36	0,19	0,45	1,63
Mean value A	37,24	108,41	149,32	2,91	3,98	7,71	7,25	0,18	0,44	1,84
B1	35,50	117,18	161,97	3,30	4,56	7,50	7,40	0,20	0,45	1,62
B2	26,92	91,52	129,33	3,40	4,80	12,01	12,01	0,21	0,73	2,50
B3	34,62	90,02	119,54	2,60	3,45	10,41	10,20	0,16	0,62	2,96
Mean value B	32,35	99,57	136,95	3,10	4,27	9,97	9,87	0,19	0,60	2,36
C1	34,48	104,07	126,90	3,09	3,68	13,00	12,23	0,19	0,74	3,32
C2	29,07	92,09	116,30	3,19	4,00	13,59	10,70	0,19	0,65	2,68
C3	37,67	95,50	110,53	2,35	2,93	14,47	10,00	0,14	0,61	3,41
Mean value C	33,74	97,22	117,91	2,88	3,54	13,69	10,98	0,17	0,66	3,14

The characteristic values of the hysteresis envelope curves and corresponding bilinear curves of all tested specimens are given in Table 2.2. From the bilinear idealizations could be observed that cracking of all specimens occurred at IDR (inter story drift ratio) of 0,17-0,19 % and the shear forces at cracking, V_{cr}, decreased from specimens A to B to C; ultimate base shears, $V_{u,}$ decreased from specimens A to B to C but the ultimate story drift d_u/h (%) increased from specimens A to B to C; the ultimate ductility μ_u increase from Type A (1,84) to Type B (2,36) and to Type C (3,14). Ultimate ductility factor corresponding to walls type C is 70% greater than ductility ratio of the specimens type A.

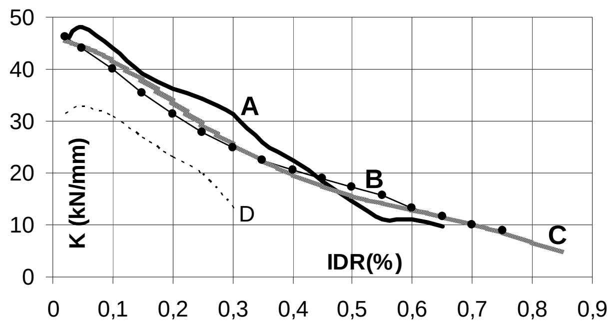


Figure 2.5 Deterioration of secant stiffness for series A,B and C

Table 2.3 Actual stiffness in various displacement stages

Specimen	A (average)	B (average)	C (average)
	kN/mm	kN/mm	kN/mm
Initial stiffness	44,20	41,72	43,45
Average elastic stiffness (bilinear idealization)	37,24	32,35	33,74
Secant stiffness at maximum resistance	28,48	20,41	20,75

3. CONCLUSIONS

Three different connection details between the masonry wall and confining ties have been experimentally investigated under constant vertical and cyclic lateral loading. Confining elements around the masonry wall increased stiffness and lateral load capacity. Connection among the masonry wall and ties increased ductility behaviour of the confined masonry.

While in the EC6 suggested behaviour factors, q, are good for confined masonry walls with good interlocking between the masonry and ties, the equations for calculation of the lateral strength are either under- or over estimating the observed values. Other available equations seem to give better estimate of the lateral strength.

ACKNOWLEDGEMENT

The research presented in this paper is part of the research project “Seismic design of infilled frames” supported by the Ministry of Science, Education and Sport of the Republic of Croatia and its support is gratefully acknowledged.

REFERENCES

Eurocode 6: Design of Masonry S – Part 1-1: Common rules for reinforced and unreinforced masonry structures. EN 1996-1-1:2004. CEN, Bruxelles

Eurocode 8 (2004) Design of structures for earthquake resistance, Part 1: general rules, seismic actions and rules for buildings. EN 1998-1:2004. CEN, Bruxelles

Frumento S, Megenes G, Morandi P, Calvi G.M, Interpretation of experimental shear tests on clay brick masonry walls and evaluation of q-factors for seismic design, IUSS Press, Pavia, 2009

Lourenco P.B, Barros J.O, Oliveira J.T (2004) Shear testing of stack bonded masonry, Constructing and Building Materials 18, Amsterdam

Tomaževič M (1999) Earthquake resistant Design of Masonry Buildings, Imperial College Press, London

Tomaževič M, Vlatko Bosiljkov, Polona Weiss (2004) Structural Behavior Factor for Masonry Structures, 13th World Conference on Earthquake Engineering, Vancouver

Tomaževič M, Weiss P (2010) Dispalacement capacity of masonry buildings as a basis for assessment of behaviour factor: an experimental study, Bulletin Earthquake Engineering, vol.8, Heidelberg

Matošević, Dj., Sigmund, V., Zovkić, J.: Experimental Testing of Masonry and Masonry Piers, 6th ICCSM Proceedings, CSM, Zagreb, 2009.

Sigmund,V., Matošević, Dj., Bošnjak-Klečina, M., Experimental Tests of Confined Masonry Walls, 14 ECEE, MAEE, Ohrid, Macedonia, 2010.

Costa, A. (2007), Experimental Testing of Lateral Capacity of Masonry Piers. An Aplication to Seismic Assesment of AAC Masonmry Buildings, Universita degli Studi di Pavia

Aničić D., Fajfar P., Petrović B., Szavits-Nossan A., Tomažević M., Earthquake Engineering – Buildings, Belgrade, 1990. (in Croat)

UNDP/UNIDO, 1984. “Volume 5: Repair and strengthening of reinforced concrete, stone and brick-masonry buildings,” Building Construction under Seismic Conditions in the Balkan Region, Project Report RER/79/015

Yoshimura, K., Kikuchi, K., Kuroki, M., Nonaka, H., Tae Kim, K., Wangdi, R. and Oshikata, A., (2004b)., " Experimental study for developing higher seismic performance of brick masonry walls." 13th world conference on earthquake engineering, Vancouver, B.C., Canada, No. 1597

Confined Masonry Network (www.confinedmasonry.org) World Housing Encyclopedia (WHE) (www.world-housing.net)

Schacher, T. (2009). Confined Masonry for One and Two Storey Buildings in Low-tech Environments - A Guidebook for Technicians and Artisans, National Information Centre of Earthquake Engineering, Kanpur, India (www.nicee.org).

Alcocer, S.M., Cesin, J., Flores, L.E., Hemander, O., Meli, R., Tena, A., and Vasconcelos, D. (2003). The New Mexico City Building Code Requirements for Design and Construction of Masonry Structures. Proceedings of the 9th North American Masonry Conference, South Carolina, USA, No. 4B3

Alcocer, S., Arias, J.G., and Flores, L.E. (2004). Some Developments on Performance-Based Seismic Design of Masonry Structures. International Workshop on Performance-Based Seismic Design, Bled, Slovenia.

Aschheim, M., Flanagan, S., Harlander, J., Pitt, C., Alfaro C., Rivas, C.,and Rodriguez, M.E. (2006). "Improving the earthquake resistance and sustainability of confined masonry (Mixto) dwellings in El Salvador." Proceedings of the 8th U.S. National Conference on Earthquake Engineering, San Francisco, California, USA, No.1462

Marinilli, A.,and Castilla, E (2004). "Experimental evaluation of confined masonry walls with several confining-columns."13th world conference on earthquake engineering, Vancouver, B.C., Canada, Accession No. 2129