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Cyclic behaviour of brick masonry walls

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ABSTRACT: An experimental program on the seismic behaviour of old brick masonry walls is presented. Shear and compression tests on full scale walls have followed a broad investigation on the basic material mechanical parameters. The experimental results are discussed with reference to some of the existing models for the estimation of the strength of structural walls. Different failure modes can be predicted on the base of the properties of bricks and mortar. The relevance of the aspect ratio on the shear strength of a wall is discussed.

1 INTRODUCTION

The evaluation of seismic vulnerability of existing buildings has been recognized as a major problem either because of the large number of buildings constructed before the development of rational seismic codes and because of the lack of knowledge about the properties of the material used and about the seismic response of single elements and whole complex structural systems.

Effective models for the numerical simulation of such buildings need to be more refined in the constitutive relations of the structural elements and of their connections rather than to be able to represent the exact geometrical configuration.

The experimental testing of masonry elements is the only viable method to investigate the conditions under which different events may take place, and to quantify the variation of the basic mechanical properties. Only on this basis rational and effective numerical models could be developed.

2 OBJECTIVES AND METHODS

The main objective of the research presented in this paper is the experimental evaluation of the resistance and ductility of masonry walls, as a function of material properties, aspect ratio and load combinations.

The experimental results are a support for the evaluation of the effectiveness of existing structural models, and to suggest needed improvements.

A further objective consists in the evaluation of the suitability and reliability of simple tests for the estimate of the fundamental global mechanical properties which are used in the models.

The experimental program has included five full scale tests and preliminary tests on bricks, mortar, mortar joints and wallettes. The capabilities of several models to predict the correct failure mode and shear strength have been checked and discussed. The overall cyclic behaviour as a function of the failure mode is of interest.

3 EXPERIMENTAL RESULTS

3.1 *Preliminary tests*

A number of preliminary tests had been performed on bricks, mortar and small masonry assemblies, to define the main mechanical properties of the constituent materials. All specimens had been tested according to the Rilem Guidelines (1988). The mortar was prepared with hydraulic lime and sand, in volumetric ratio 1:3.

The main results are presented in table 1. The wallettes tested in compression were 250 (thickness, two unit breadth) x 765 x 680 (height) mm.

Particular care was used in testing brick triplets to evaluate the apparent cohesion and friction coefficient between bricks and mortar. It is believed that the best estimate for the relation between shear strength and normal stress is given by equation (1), which is represented by the straight line in figure 1:

$$\tau = 0.206 + 0.813\ \sigma \qquad (1)$$

The mechanical properties of the masonry were therefore similar to those of a very good quality old masonry.

3.2 *Full scale cyclic tests*

A total of five specimens were prepared, all having a width d = 1.5 m and a thinckness t = .38 m, with a height h of 2 m (three walls) and 3 m (two walls). The purpose was to perform one preliminary monotonic test, and to explore two values of aspect ratio and two values

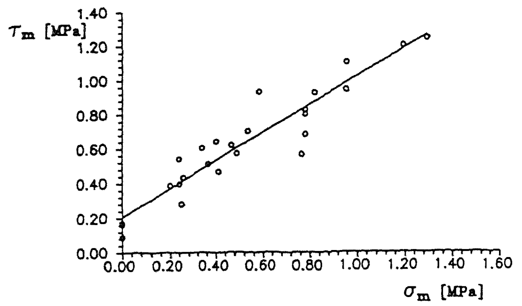

Figure 1: Results of the shear tests on mortar joints (triplets).

of nominal vertical compression stress σ_m (0.4 and 1.2 MPa). The reasons for these choices have been discussed by the same authors (1991) and will not be discussed again here.

The tests have been performed according to the scheme presented in figure 2: the vertical force was applied first and the horizontal cyclic load was then applied keeping the valves of the vertical jacks closed. No rotation was therefore theoretically possible, and a double bending condition was present. As a consequence of the testing mode the vertical load increased together with the horizontal load, depending upon the stiffness degradation of the wall. The axial load increment was percentually more significant for a lower nominal value, but the forces in the jacks were monitored with continuity.

The results of the tests are summarized in table 2 and in figures 3 to 6, some comments are presented in what follows.

Table 1: Results of the preliminary tests; E is the secant Young's modulus at $\sigma = 0.33\ f_u$.

test type	n. of specimens	mean (MPa)	c.o.v.
compression on brick f_b	28	19.72	8.82%
splitting on brick f_{mt}	29	1.26	20.28%
compression on mortar f_m	15	4.33	1.84%
splitting on mortar f_{mt}	14	0.66	8.44%
mod. of rupt. on mortar f'_{mt}	15	1.59	3.47%
direct tension on mortar joint f_{jt}	13	0.073	10.53%
compression on masonry f_u	5	7.92	20.2%
E	4	2991	15.1%

Wall MI1m and MI1, h = 2 m, σ_m = 1.2 MPa.

Wall MI1m was tested monotonically, wall MI1 cyclically. The maximum horizontal load corresponded to the first diagonal crack, and decreased rapidly to a lower value. The failure mode concerned mainly the mortar beds, with slight damage in the bricks.

Wall MI2, h = 2 m, σ_m = 0.4 MPa.

The first failure was due to a shear sliding mechanism located at the top mortar layer, with an apparent friction coefficient between 0.57 and 0.65. Thanks to the axial load increment the horizontal load also increased up to the formation of diagonal cracks. The post peak behaviour was similar to case MI1.

Wall MI3, h = 3 m, σ_m = 1.2 MPa.

The failure mode involved sub-vertical cracks started in the central area of the panel, with extensive brick damage. The cracks extended slowly, cycle after cycle, with a correspondent gradual strength deterioration.

Wall MI4, h = 3 m, σ_m = 0.4 MPa.

The mortar joints collapsed, allowing the formation of two wide diagonal cracks. The increment of the vertical load was in this case critical to avoid a flexural failure.

The overal cyclic behaviour seems to be strongly affected by the different failure mechanisms.

4 FAILURE MODELS

The dominant failure modes in the shear and compression tests are associated to the biaxial tension - compression state of stress at the center of the panel. A linear elastic finite element analysis was considered to be appropriate to evaluate the state of stress in the central area of the panels at the onset of the first shear crack. A no - tension condition at the top and bottom boundaries

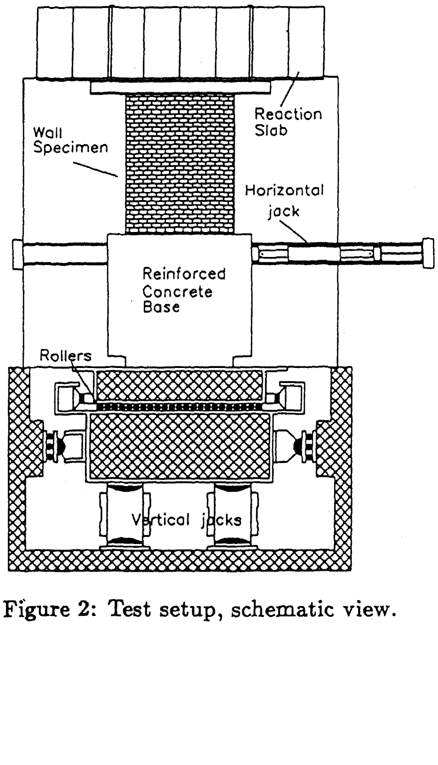

Figure 2: Test setup, schematic view.

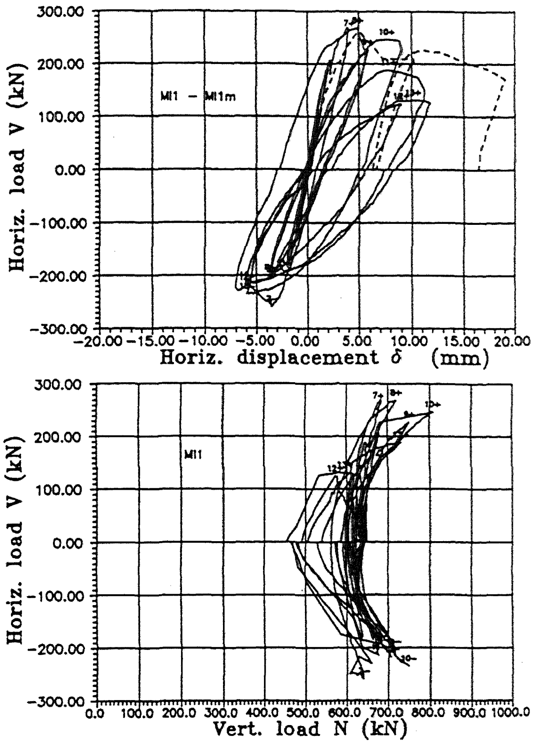

Figure 3: Test on wall MI1 and MI1m (dashed line); $h = 2$ m.

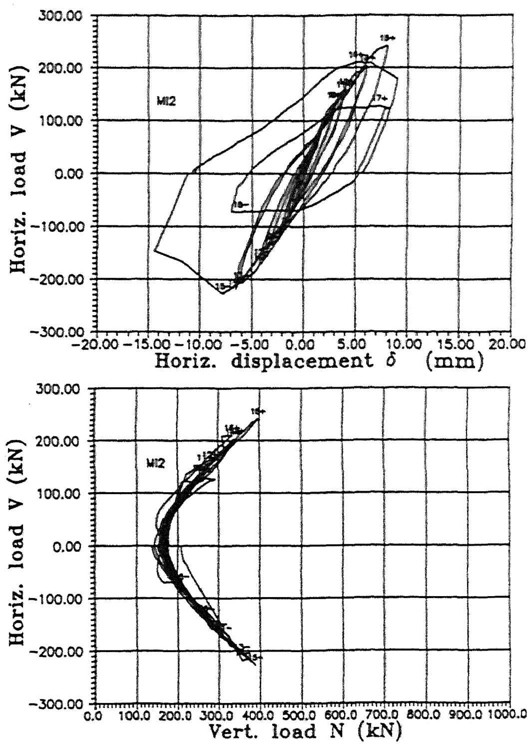

Figure 4: Test on wall MI2; $h = 2$ m.

was assumed, to simulate the presence of flexural cracks in the bed joints, which were detected in the tests.

A first interpretation of the results was tried considering the referential tensile strength criterion (Turnsek and Sheppard 1980), which assumes that shear cracking happens when the maximum principal tensile stress reaches a critical value f_{tu}. Table 2 shows how the maximum tensile stress in each wall at first cracking is considerably variable, and seems to assume two roughly constant values for the two different values of aspect ratio. It has to be noticed that this seems to be true even when the orientation of the principal stresses is approximately the same. This result instills some doubt on the accuracy of shear failure models based on the tensile strength of masonry assumed as a constant parameter, although these models have the advantage of being of simple practical use.

The f.e. analyses also showed that the walls with h/d = 1.34, in comparison with the walls with ratio h/d = 2, were characterized by higher values of the ratio X = σ_x/σ_z at the center of the panel, where σ_z is the normal stress in the vertical direction (i.e. perpendicular to bed joints) and σ_x the normal stress in the horizontal direction (i.e. perpendicular to head joints). The influence of this parameter on the strength of masonry subjected to biaxial states of stress was pointed out by Dialer (1991), considering the results of a series of tests on biaxially loaded masonry panels. Although the influence of this parameter is rather evident from the experimental point of view, it is still not clear whether this influence has to do with an increased performance of the head joints, which takes advantage of the compression σ_x, or with other factors which still have to be detected.

A comparison with the failure criterion developed by Mann and Muller (1988) was done, where the strength of the head joints is also taken into account by means of a cohesion c_{xz} and a friction coefficient μ_{xz}, in addition to the strength of the bed joints which is characterized by the cohesion c_{zx} and the friction coefficient μ_{zx}. The criterion defines three failure conditions, namely friction failure in the bed joints, tensile cracking of bricks, and compressive failure of masonry. The latter condition is not relevant to our experimental results, while the crack pattern of the walls may be associated to the first two types of failure (or a combination of them). The criterion in its extended version assumes that at ultimate condition the head joints develop the maximum shear strength which is compatible with their failure domain. The failure conditions for these two mechanisms are given by the follouing equations:

$$\tau = \tau_j = \frac{c_{zx} + \mu_{zx}\sigma_z + (c_{xz} + \mu_{xz}\sigma_x)\frac{2\Delta_x}{\Delta_x}\mu_{zx}}{1 + \frac{2\Delta_x}{\Delta_x}\mu_{zx}} \qquad (2)$$

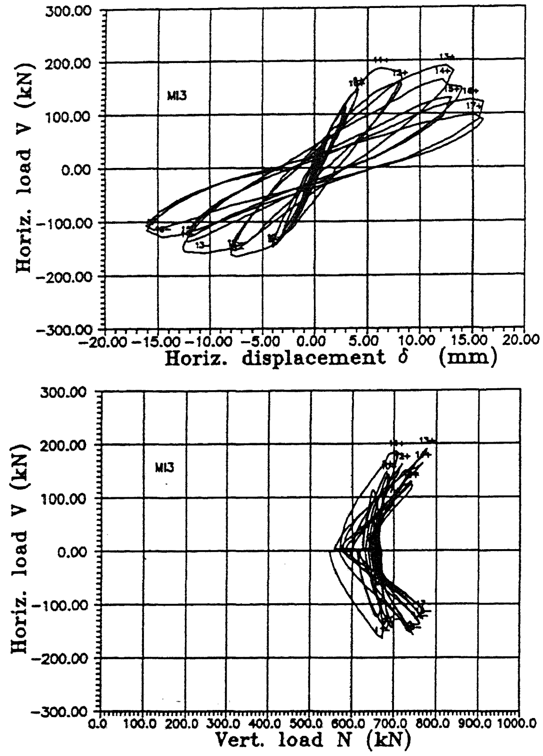

Figure 5: Test on wall MI3; $h = 3$ m.

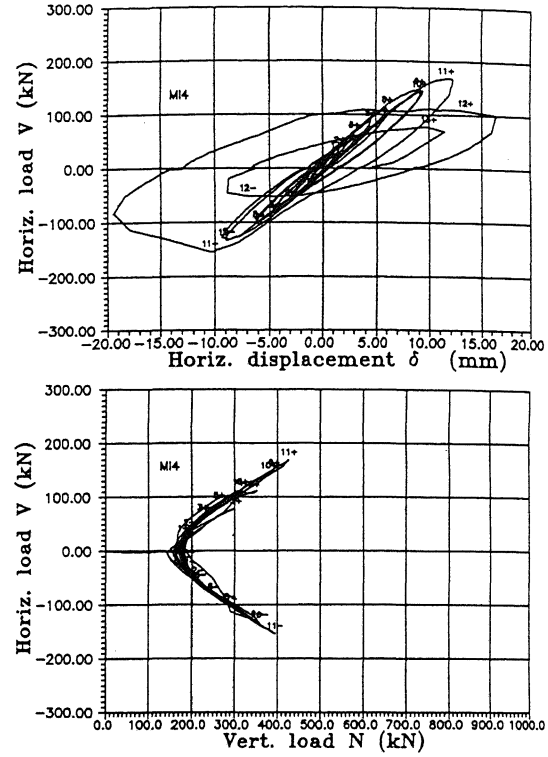

Figure 6: Test on wall MI4; $h = 3$ m.

for bedjoint failure, where Δ_z is the height of the brick and Δ_x is the width of the brick, and by:

$$\tau = \tau_b = \frac{1}{2}(c_{xz} + \mu_{xz}\sigma_x) + \frac{f_{tb}}{\alpha}\sqrt{1 + \frac{(\sigma_z + \sigma_x)}{f_{tb}} + \frac{\sigma_z\sigma_x}{f^2_{tb}}} \qquad (3)$$

for brick tensile failure, where $\alpha \simeq 2.3$, f_{tb} = brick tensile strength.

Following a procedure similar to the one presented by Mann and Muller (1988), the normal stresses σ_x and σ_z obtained in the f.e. analyses were substituted in equations 2 and 3 to calculate the shear stress τ at failure, and then the shear stress at failure was compared with the shear stress σ_{zx} given by the f.e. analyses. The ratios $\gamma_j = \tau_j/\sigma_{zx}$ and $\gamma_b = \tau_b/\sigma_{zx}$ were defined, so that a value equal or less than one would imply that the failure condition for the considered failure mode is met. The values of the input parameters used in the criterion are shown in table 3, where it can be seen that the strength parameters for the head joints are assumed to be less than or equal to the values obtained from the triplet tests. It was found that the best agreement between experiments and numerical application is obtained when the cohesion in the head joints c_{xz} is assumed to be zero in the walls with h/d = 2.0, and when it is assumed to be equal to c_{zx} in the walls with h/d = 1.34. This could be interpreted as if the local friction failure in the head joints in these walls developed prematurely, before the attainment of the failure condition given by bed joint failure or brick tensile failure, because of the low value of compressive stress σ_x. This hypothesis is plausible, but the experimental observations do not allow its verification. With these hypothesis, the calculated values of γ_b and γ_j are given in table 4. According to the failure criterion the failure mechanism for each wall is given, by the lowest value of γ. In the cases where the values of γ_j and γ_b are close, a mixed failure mode can be supposed. The agreement with the observed experimental failure modes is very good. More details of this numerical applications have been presented by Magenes (1991).

5 CONCLUSIONS

The experimental shear-compression tests on full scale walls have shown a prevalence of failure modes triggered by stress situation in the center of the panels, with two possible out comes:

- frictional failure of the mortar joints, typical of a lower axial action, and
- tensile cracking of bricks, typical of a higher axial action.

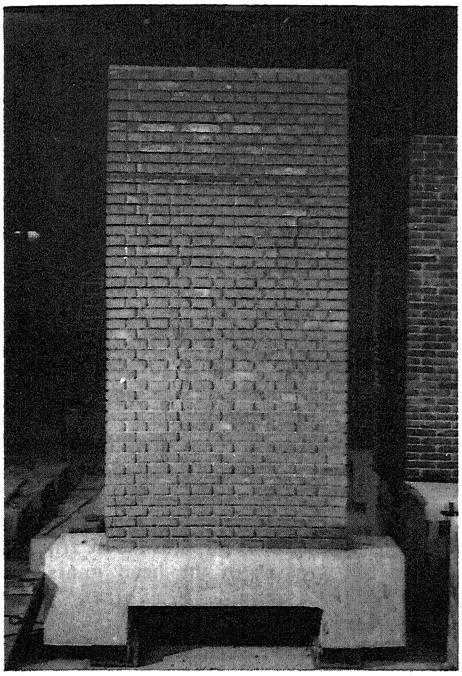

Figure 7: Wall MI3 a the end of the test: tensile failure of bricks.

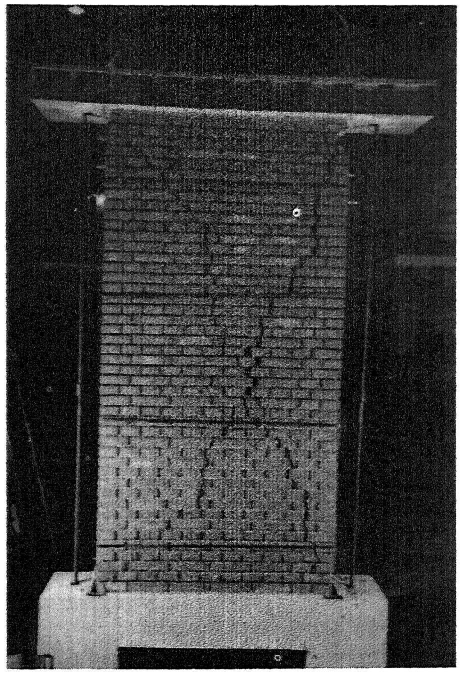

Figure 8: Wall MI4 at the end of the test: joint failure.

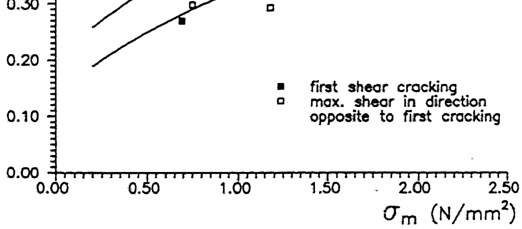

Figure 9: Shear strength of the panels compared with ideal curves associated to constant values of the referential tensile strength f_{tu}; $\tau_u = V_u/dt$, $\sigma_m = N/dt$

Mixed mode failures were also reported. The influence of the failure mechanism on the post-peak hysteretic behaviour (strength and stiffness degradation) has proved to be very significant.

The more slender walls (h/d = 2) have shown apparently a shear strength sensibly lower than the squatter walls (h/d = 1.34), for the same axial action. Some f. e. analyses have shown that this fact corresponds to different values of the horizontal compression stress, i.e. the compression stress perpendicular to the vertical mortar joints. A model proposed by Mann and Muller has proved to be able to predict the correct failure mode and to estimate with reasonable accuracy the failure load if different contribution of the vertical mortar joints are considered. Other models, based on the evaluation of the tensile strength of masonry do not seem to be effective in all the tested cases, even if the limited number of tests suggests caution
in drawing definitive conclusions.

The possibility of estimating the masonry strength on the base of the properties of the constituent material seems to be promising.

The evaluation of the role played by vertical mortar joints deserves further experimental and analytical work.

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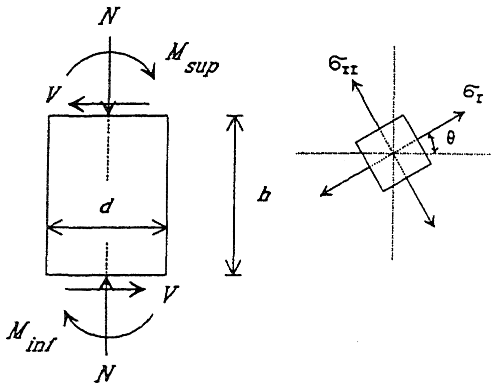
Table 2: Actions at first shear cracking and stresses at the center at the panels according to f.e. analyses.



	MI1m ; MI1	MI2	MI3	MI4
h (m)	2.0	2.0	3.0	3.0
h/d	1.33	1.33	2.0	2.0
N (kN)	580 ; 639	386	705	393
V_t (kN)	259 ; 259	227	185	153
M_t^{inf} (kNm)	185 ; 228	223	228	219
M_t^{sup} (kNm)	335 ; 290	231	327	240
σ_x (MPa)	-1.023 ; -1.123	-.668	-1.245	.691
σ_z (Mpa)	-.066 ; -.05	-.111	-.002	-.017
σ_{xz} (MPa)	-.695 ; -.679	-.602	-.486	-.402
$\sigma_I = f_{tu}$ (MPa)	.287 ; .280	.274	.165	.163
σ_{II} (MPa)	-1.377 ; -1.452	-1.052	-1.413	-0.881
θ (gradi)	-27.7 ; -25.8	-32.6	-19.0	-25.0
$X = \sigma_z/\sigma_x$	.065 ; .045	.165	.002	.024

Table 3: Values of the parameters used in the failure criterion

	γ_j	γ_b
Δ_x (cm)	12	25
Δ_z (cm)	5.5	5.5
f_{bt} (MPa)	0.84	0.84
c_{xz} (MPa) (bed joints)	0.206	–
μ_{xz} (bed joints)	0.813	–
c_{zx} (head joints)	0 – 0.206	0 – 0.206
μ_{zx} (head joints)	0.406 – 0.813	0.406 – 0.813

Table 4: Minimum calculated values of the coefficients γ_j and γ_b for each wall

	γ_j	γ_b
MI1m	1.012	1.006
MI1	1.080	1.12
MI2	0.920	1.096
MI3	1.350	1.165
MI4	1.101	1.249

REFERENCES

Calvi, G.M. & G. Magenes 1991. Experimental evaluation of seismic strength of old masonry structures. *Proc. 9th IBMaC*: 490-497. Berlin: DGfM.

Dialer, C. 1991. Some remarks on the strength and deformation behaviour of shear stressed masonry panels under static monotonic loading. *Proc. 9th IBMaC*: 276-283. Berlin: DGfM.

Magenes, G. 1991. Seismic behaviour of brick masonry: strength and failure mechanisms of masonry piers. Ph. D. Thesis: University of Pavia (in Italian).

Mann, W. & H. Muller 1988. Interpretation of experimental results with an enlarged failure criterion considering the stresses in the head joints. In: *Untersuchungen zum Verhalten von Mauerwerksbauten unter Erdbebeneinwirkung*. Darmstadt: Technische Hochschule (in German).

RILEM 1988. Draft recommendations TC-76 LUM: General recommendations for testing load-bearing masonry. *Materials and Structures*, 21 (123):229-231.

Turnsek, V. & P. Sheppard 1980. The shear and flexural resistance of masonry walls. *Proc. Int. Res. Conf. on EQ. Eng.*: 517-573. Skopje.