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STRUCTURAL BEHAVIOR FACTOR FOR MASONRY STRUCTURES

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SUMMARY

A range of values of structural behavior factor *q* for each masonry construction system is given in the basic document of EC 8, presuming a substantial amount of ductility capacity even in the case of unreinforced masonry construction systems. Detailed values for each construction system, however, need to be determined by National Annexes according to the national characteristics of masonry structures. Since the experimental data on the basis of which the proposed *q*-factor values could be evaluated are lacking, a research project has been recently carried out to investigate the actual behavior of typical unreinforced masonry structures on a simple uni-directional shaking table. Six models representing buildings of two different structural configurations and two different types of masonry materials have been tested, a two-story terraced house with main structural walls orthogonal to seismic motion and a three-story apartment house with uniformly distributed structural walls in both directions. Four models of the first and two models of the second type, built at 1:5 scale, have been tested. In the case of the terraced house, two models have been built as either partly or completely confined masonry structures. On the basis of experiments a conclusion can be made that the ranges of values of structural behavior factor *q*, proposed in EC 8 for different masonry systems, are adequate. In the particular case studied it has been found that the value of q = 1.5 may be used for the case of unreinforced structures of both types and q = 2.0 for the confined terraced house. The study indicated that the values depend not only on the system of construction, but also on the properties of masonry materials and structural configuration of the building under consideration. Therefore, experimental research is needed for the assessment of a particular class of values of behavior factor *q* factor for a particular structural type. The values cannot be assessed by means of only ductility tests of structural walls.

INTRODUCTION

Low-rise unreinforced or confined masonry buildings - family houses represent the major part of masonry construction in Europe. Because of a predominant non-ductile behavior of unreinforced masonry, the recent version of Eurocode 8: Design of structures for earthquake resistance (EC 8 [1]) limits the construction of unreinforced masonry buildings to the zones, where the design ground acceleration values at a site (a_gS) do not exceed 0.20 g. No such limitations are given for the construction of confined masonry building systems. In both cases, however, the seismic resistance needs to be verified by calculation, unless the buildings are in conformity with the requirements for simple masonry buildings in the case of which

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the calculations are not mandatory. In seismic resistance verification, the design seismic loads are correlated with the design seismic resistance of the structure under consideration. In order to determine the design seismic loads, a range of values of structural behavior factor q for each construction system is given in the basic document of EC 8, presuming a substantial amount of ductility capacity even in the case of unreinforced masonry construction systems. Detailed values for each construction system, however, need to be determined by National Annexes according to the national characteristics of masonry structures. Since the experimental data on the basis of which the proposed q-factor values could be evaluated are lacking, a research project has been recently carried out at Slovenian National Building and Civil Engineering Institute (ZAG), aimed at obtaining reliable experimental information regarding the energy dissipation capacity of some typical simple European masonry construction systems. Not all details of research will be presented. Only some most important information will be given and relevant conclusions will be presented in this paper.

DESIGN SEISMIC LOADS AND STRUCTURAL BEHAVIOR FACTOR *q*

For decades, the design philosophy of seismic codes, including parts which regulate the design and construction of masonry buildings, is based on:

- no collapse requirement, and
- damage limitation requirement.

Therefore, the ultimate limit state, associated with collapse, and the serviceability limit state, associated with the occurrence of damage, need to be verified. However, because of specific characteristics of masonry structures and masonry materials, there is usually no need to check the serviceability limit state. Since masonry buildings are rigid and deformations during earthquakes are small, the requirements for the serviceability limit are in most cases automatically fulfilled if the structure is verified for the ultimate state.

Taking into account the regularity of masonry buildings whose response is not significantly affected by contributions from higher modes of vibration, lateral force method of analysis will provide adequate results. According to EC 8, the seismic base shear force F_b for each horizontal direction in which the building is analyzed, is determined as follows:

$$F_b = S_d(T_1)\, m\, \lambda, \tag{1}$$

where:

$S_d(T_1)$ = the ordinate of the design spectrum at period T_1,
T_1 = fundamental period of vibration of the building for lateral motion in the direction considered,
m = total mass of the building above the foundation or above the top of a rigid basement,
λ = correction factor, accounting for the fact that in building with at least three stories and translational degrees of freedom in each horizontal direction, the effective modal mass of the 1st mode is smaller than the total building mass.

Masonry buildings are rigid structures with natural periods of vibration T_1 ranging between T_B and T_C, where the response spectrum is flat. Therefore, the ordinate of the design spectrum for masonry buildings can be determined by:

$$S_d(T_1) = a_g \, S \, \eta \frac{2.5}{q}, \tag{2}$$

where:

a_g = design ground acceleration on type A ground (rock or rock-like formation),
S = soil factor,
η = damping correction factor (η = 1 for 5% viscous damping),
q = structural behavior factor.

As specified in EC 8, the capacity of a structural system to resist seismic actions in the non-linear range generally permits the design for forces smaller than those corresponding to a linear elastic response. To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements and other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one by introducing the behavior factor q.

In a qualitative and simplified way, the definition of behavior factor q is explained in Figure 1, where the seismic response curve of an actual structure, idealized as a linear elastic - perfectly plastic envelope, is compared with the response of a perfectly elastic structure having the same initial elastic stiffness characteristics.

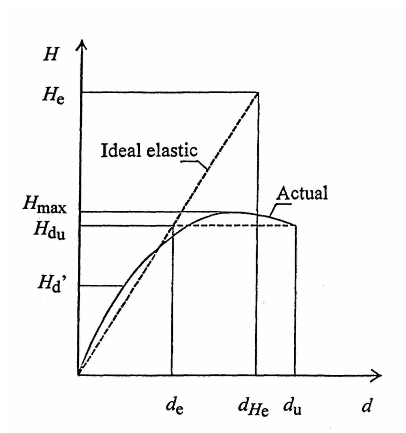


Figure 1. Definition of structural behavior factor *q*

As a result of energy dissipation capacity of the actual structure, expressed by the global ductility factor $\mu_u = d_u/d_e$, there is no need for the structure to be designed for strength, i.e. for the expected elastic load H_e. The structure is designed for the ultimate design load H_{du} and the ratio between the two is called the behavior factor q:

$$q = H_e/H_{du}. \tag{3a}$$

In other words, behavior factor q is an approximation of the ratio of seismic forces which the structure would experience in the case of an elastic response, to the minimum seismic forces that may be used in the design with a conventional elastic model, still ensuring a satisfactory response of the structure. Following the definition in Figure 1, structural behavior factor can be also expressed in terms of the global ductility factor $\mu_u = d_u/d_e$ as follows:

$$q = (2\,\mu_u - 1)^{1/2}. \tag{3b}$$

A range of values of q factor for different systems of masonry construction is proposed in the recent draft of EC 8:

- for unreinforced masonry: $q = 1.5 - 2.5$,
- for confined masonry: $q = 2.0 - 3.0$,
- for reinforced masonry: $q = 2.5 - 3.0$.

Typical values of design spectrum ordinates $S_d(T_1)$ for plain and confined masonry buildings, which should be taken into consideration when verifying the seismic resistance of plain and confined masonry structures, are given in Table 1. Although the values have been calculated for the condition of firm soil (soil factor $S = 1.0$), the design spectrum ordinates (which also represent the design values of ultimate base shear coefficients) calculated by considering the lower bound values of structural behavior factors, are much higher than those so far used by national codes. This would of course cause severe design problems after the withdrawal of national codes and would possibly eliminate the construction of plain and confined masonry structural systems even in the zones of moderate seismic intensity.

Table 1. EC 8 design spectrum ordinates $S_d(T_1)$ for plain and confined masonry buildings. Soil factor $S = 1.0$

Seismic intensity (MSK)		VII	VIII	IX
Design ground acceleration		0.1 g	0.2 g	0.4 g
Plain masonry:	$q = 1.5$	0.17	0.33	0.67
	$q = 2.5$	0.10	0.20	0.40
Confined masonry:	$q = 2.0$	0.125	0.25	0.50
	$q = 3.0$	0.08	0.17	0.33

As can be seen in Table 1, the upper bound values of q-factors, however, result into the $S_d(T_1)$ values (ultimate design base shear coefficient values) which are of the same order of magnitude as used in existing national codes. On the basis of the analysis of the effects of recent earthquakes on modern masonry buildings it can be concluded that these values are adequate for seismic resistance verification of typical modern masonry construction (Tomaževič and al. [2]).

Following the simple definition and the observed behavior, some estimates regarding the validity of the proposed values of q-factor have already been carried out on the basis of the results of models of masonry buildings tested on the shaking-table by Tomaževič and Weiss [3]. The values of $q = 2.84$, 2.69 and 3.74 have been obtained for the cases of unreinforced, confined and reinforced masonry buildings, respectively. This also indicated a possibility that the lower bound values of the range proposed by the recent version of EC 8 are conservative. However, without systematic experimental research and analysis of the seismic behavior of masonry buildings during recent earthquakes, it is not possible to define the values of q-factors. Otherwise we are facing risk that the design situation will not be realistic.

EXPERIMENTAL PROGRAM AND DESCRIPTION OF TESTS

Six models representing buildings with two different structural configurations and constructed with two different types of masonry materials have been tested on a simple uni-directional seismic simulator, a two-story terraced house with main structural walls orthogonal to seismic motion (models M1 - Figure 2) and a three-story apartment house with uniformly distributed structural walls in both directions (models M2 - Figure 3). Four models of the first and two models of the second type have been tested. In the case of the terraced house, two models (Figures 4 and 5) have been built as either partly or completely confined masonry structures (Table 2).

Table 2. Shaking-table tests - description of tested models

Designation	Type	Material	Bed joint	Remark
M1-1	Terraced house	Calcium silicate	Thin	no confinement
M1-2	Terraced house	Hollow clay unit	Normal	no confinement
M1-1c	Terraced house	Calcium silicate	Thin	confined staircase walls
M1-1d	Terraced house	Calcium silicate	Thin	fully confined walls
M2-1	Apartment house	Calcium silicate	Thin	no confinement
M2-2	Apartment house	Hollow clay unit	Normal	no confinement

Because of the limited capacity of earthquake simulator installed at ZAG, models have been built at a 1:5 scale with special model materials designed to fulfill the requirements of complete model similitude. The correlation between the model and prototype characteristics of masonry has been verified by testing a series of model and prototype-size walls under compression and a combination of compression and cyclic lateral loading. Although good correlation has been obtained between model and prototype masonry (see Figure 6), it should be borne in mind that by testing small scale masonry models only the global behavior and mechanism of the buildings' behavior can be adequately simulated, and not the behavior of structural details.

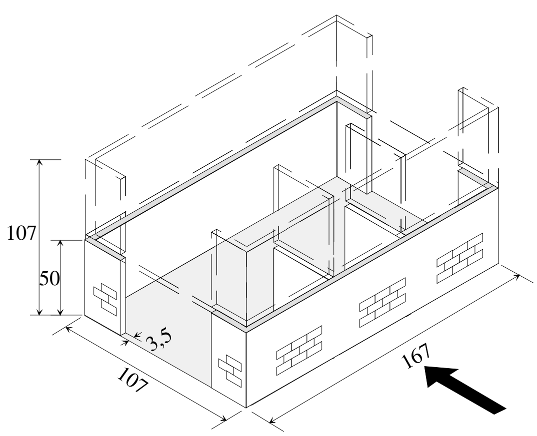


Figure 2. Terraced house (in cm)

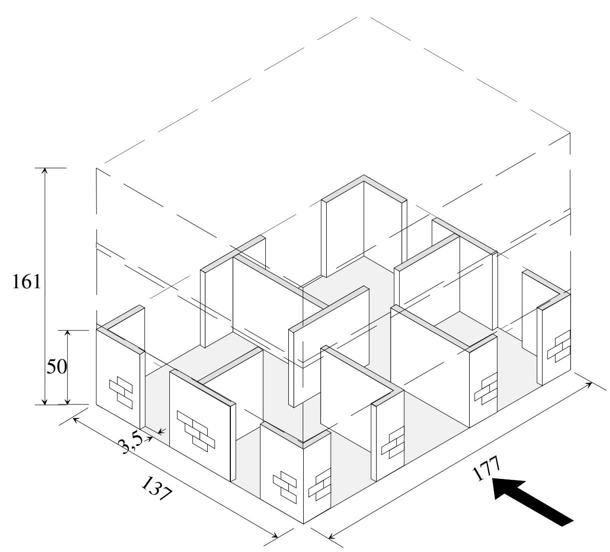


Figure 3. Apartment house (in cm)

The models have been built on foundation slabs which have been fixed before the tests to the steel platform of the shaking table by means of bolts. In order to fulfill the requirements for similitude of dynamic behavior, additional masses (steel bricks) have been fixed to each floor slab to simulate the effect of the live load. The models have been instrumented with displacement meters (LVDTs) and accelerometers fixed on the model at both edges and center of each floor slab.

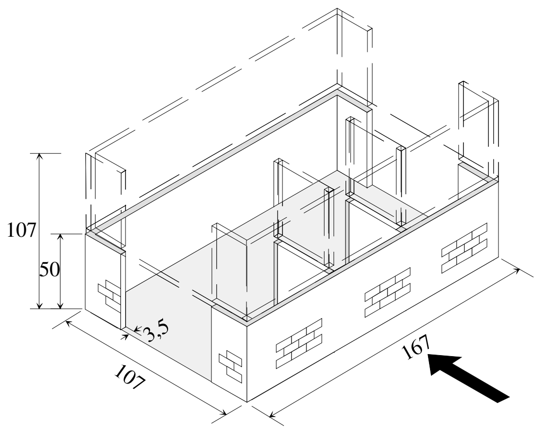


Figure 4. Model M1-1c – Terraced house with confined staircase walls (in cm)

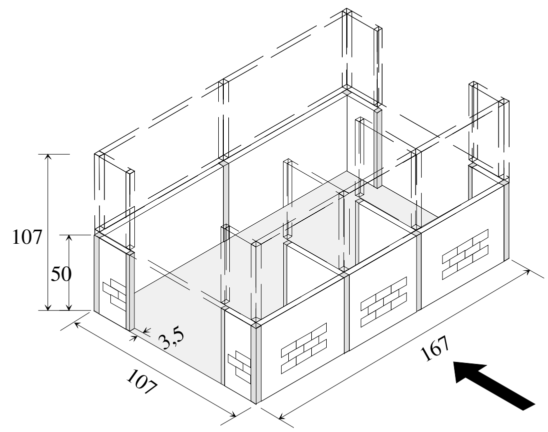


Figure 5. Model M1-1d – Terraced house with fully confined walls (in cm)

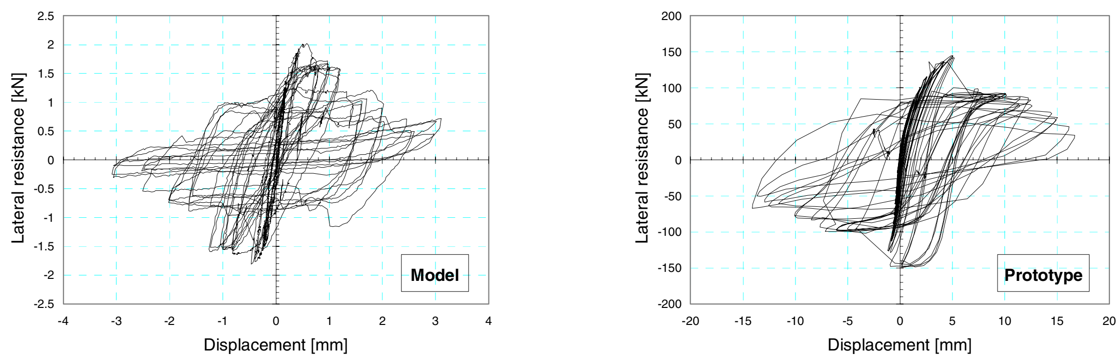
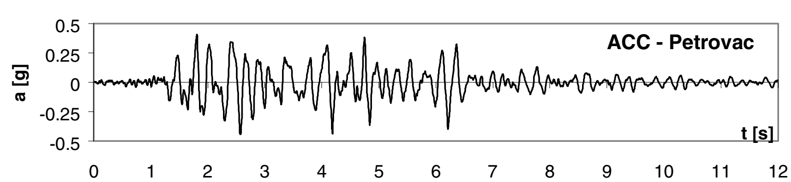


Figure 6. Correlation between the results of typical model (a) and prototype wall (b) lateral resistance tests

(a)



(b)

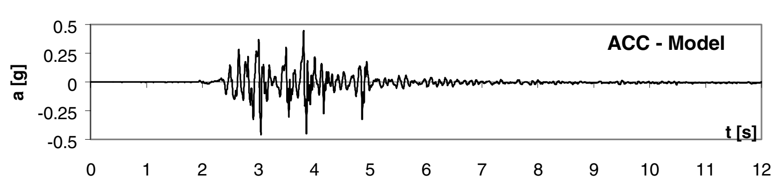


Figure 7. Earthquake acceleration record (a) and (b) typical scaled accelerogram

The north-south component of the earthquake acceleration record obtained at Petrovac during the April 15, 1979, earthquake in Montenegro, with peak ground acceleration of 0.43 *g* has been used to drive the shaking-table. The intensity of shaking was controlled by adjusting the maximum amplitude of the shaking-table displacement, obtained by numerical integration of the accelerogram used as the input in each successive test run, scaled according to the laws of model similitude (Figure 7). The analysis of shaking-table motion, carried out during one of our previous studies (Tomaževič and Velechovsky [4]) has shown that the shape of absolute acceleration spectra of the shaking-table motion, normalized with regard to maximum acceleration, are in good agreement with the shape of one of the previous versions of the EC 8 response spectrum.

All models have been tested by subjecting them to a step-wise increasing intensity of the shaking-table motion in each subsequent test run until models' final collapse. During the tests, displacement and acceleration responses of the models at each floor have been measured. The behavior of the models during testing has bee video-taped for further analysis of damage propagation. After each test run, the models have been inspected for damage, and the cracks have been marked and photographed. Also, the changes in dynamic properties of the models have been determined by analyzing the records of free vibrations obtained by hitting the top slab of the model with hammer.

TEST RESULTS

All models failed in shear, as expected. Regardless to the structural type and configuration, shear cracks developed in structural walls in the direction of seismic motion, subsequently leading to stiffness and strength degradation and final collapse of the models. The unreinforced terraced house model M1-1 and apartment house model M2-1, made of masonry simulating calcium silicate masonry units with thin bed joints collapsed immediately after the first damage occurred, whereas respective models M1-2 and M2-2, made masonry simulating hollow clay units laid in normal bed joints, though damaged, withstood additional shaking before collapse. The observed phenomenon has yet to be studied and explained. The behavior of partly and fully confined terraced house models M1-1c and M1-1d was significantly improved. Typical damage to the models just before collapse is shown in Figures 8 and 9 for a terraced and apartment house model, respectively.

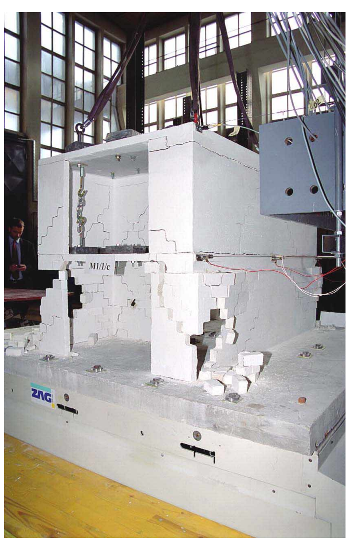


Figure 8. Confined terraced house model M1-1c just before collapse

Figure 9. Apartment house model M2-2 just before collapse

As can be seen, the damage to structural walls was concentrated in the first story, so that typical shear type mechanism of seismic behavior prevailed. Very little damage to the walls in the upper stories has been observed at the moment of collapse in all cases, including confined terraced house models M1-1c and M1-1d. As a result of this mechanism, relative displacements of the upper stories in the non-linear range of vibration were very small compared to the first story drift. As the deformations and the amount of damage were small, the amount of dissipated, hysteretic energy in the upper stories did not exceed 5 % of the energy dissipated in the first story. Taking this into consideration, the conclusion can be made that the resistance envelope of the first story determines the seismic behavior of the tested structures.

On the basis of the recorded displacement and acceleration response time histories and taking into account the masses of the models, concentrated at each floor level, the maximum values of the base shear developed in the models during the individual phases of testing, have been calculated. The values have been expressed in a non-dimensional form in terms of the base shear coefficient (BSC), which is the ratio between the base shear resisted and the weight of the model. The values are plotted against the first story rotation angle (the ratio between the relative story displacement and story height), hence obtaining the lateral resistance - displacement envelopes of a critical story in a non-dimensional form (Figure 10).

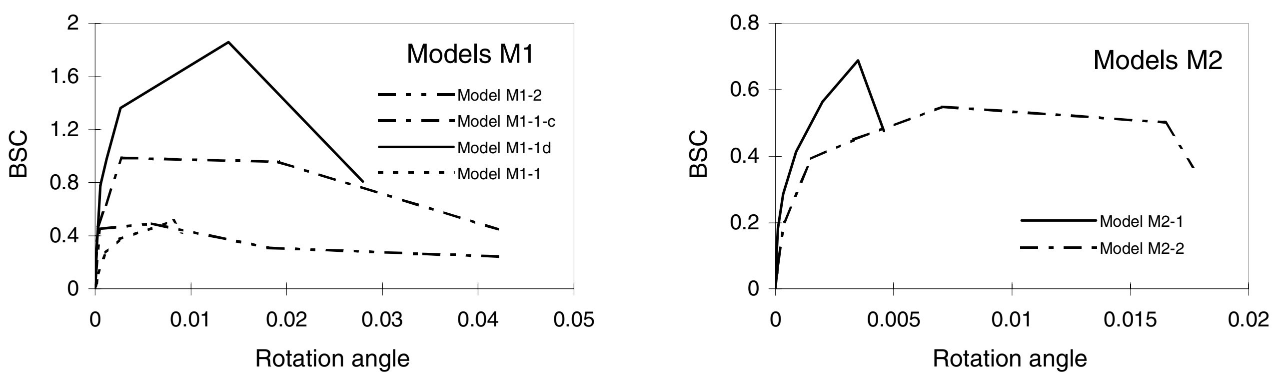


Figure 10. Experimentally obtained base shear coefficient - story rotation angle relationships

As can be seen, the values of story rotation angle at the instant of time where the stiffness of the models significantly changed as a result of damage occurred to structural walls (damage limit), are very close in all cases. The values of 0.25 % have been measured in the case of the terraced house models M1 and the values of 0.3% in the case of the apartment house models M2. It can be also seen that the damage limit values of story rotation angle coincide with or are very close to the values where the maximum resistance has been attained.

As regards the influence of different quality of masonry materials on the seismic behavior of the tested building types, it can be seen that the models of both, terraced house and apartment house structural type, made of model materials simulating calcium silicate masonry units (models M1-1 and M2-1) exhibited substantially more brittle behavior than the models of the same type, but made of model materials simulating hollow clay units. However, there has been not much difference observed as regards the resistance. The confinement of structural walls with vertical r.c. confining elements in the case of the terraced house models M1-1c and M1-1d proved to be a successful measure of improving the seismic behavior of the terraced house type of structure as regards both lateral resistance and displacement capacity.

The results of tests are summarized in Table 3, where the values of the maximum attained base shear coefficient BSC_{max}, evaluated on the basis of the known masses of the models, concentrated at floor levels, and measured floor acceleration responses, as well as the measured values of the story rotation angle at the damage limit Φ_{dam}, maximum attained resistance Φ_{Hmax} and ultimate limit (before collapse) Φ_u are given. In order to be considered as prototype values, model values of BSC_{max}, given in Table 3, should be scaled by taking into consideration the actual correlation between the strength of prototype and model masonry, following the laws of model similitude (in this particular case the approximate value of force scale factor is $S_F = 0.6$).

Table 3. Parameters of seismic resistance of the tested models at characteristic limit states

Model	BSC_{max}	Φ_{dam}	Φ_{Hmax}	Φ_u
M1-1	0.52	0.19 %	0.82 %	0.91 %
M1-2	0.49	0.25 %	0.56 %	3.98 %
M1-1c	0.99	0.26 %	0.26 %	3.96 %
M1-1d	1.86	0.25 %	1.31 %	2.63 %
M2-1	0.69	0.33 %	0.33 %	0.43 %
M2-2	0.55	0.30 %	0.66 %	1.66 %

EVALUATION OF STRUCTURAL BEHAVIOR FACTOR *q*

In order to evaluate the values of structural behavior factor q, the simple philosophy explained in Figure 1 and basic definitions given in Equations 3a and 3b have been followed. For each of the tested models, the theoretical elastic response in terms of the calculated maximum elastic base shear has been compared with the maximum base shear value BSC_{max} (Table 3), evaluated on the basis of the known masses of the models and measured acceleration response, as well as with the value BSC_u obtained by the idealization of the experimental resistance envelopes shown in Figure 6. In order to retain the character of simplicity, experimentally obtained base shear coefficient - story rotation angle relationships have been idealized as bilinear elastic-plastic relationships.

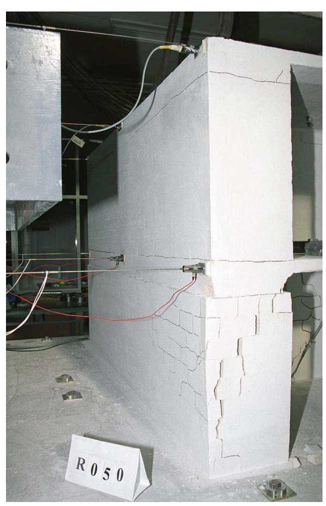


Figure 11. Typical damage to structural walls at ultimate damage limit (model M1-1c)

In the idealization of the experimentally obtained resistance envelopes, the story rotation angle (relative story displacement) at the point where 20 % of strength degradation has occurred, has been defined as the ultimate story rotation angle (displacement) which the structure can resist without risking collapse. This assumption has been considered in the cases where no sudden collapse of the models (models M1-1 and M2-1) has been observed during the shaking tests. The rotation angle (displacement) at 20 % of strength degradation has been considered as ultimate in the evaluation of the idealized ultimate global ductility factor of the structure μ_u. However, it has to be noted that substantial damage to structural walls of the models has occurred at that stage. Therefore and in order to fulfill also the "damage limitation" requirement, only part of the available displacement capacity has been taken into account in the evaluation of behavior factor q on the basis of global ductility of the structure (Equation 3b), limited by the displacement value where severe damage to structural walls occurs. This value has been arbitrarily chosen to be 3-times the value of story rotation at the damage limit $\Phi_u = 3\ \Phi_{dam}$. As can be seen in Figure 11, where typical damage to structural walls at the maximum permissible story rotation angle (ultimate damage limit) is shown, the damage at this point can be considered severe. Typical examples of simple evaluation of the values of behavior factor q are shown in Figures 12 and 13 for the terraced and apartment house models, respectively.

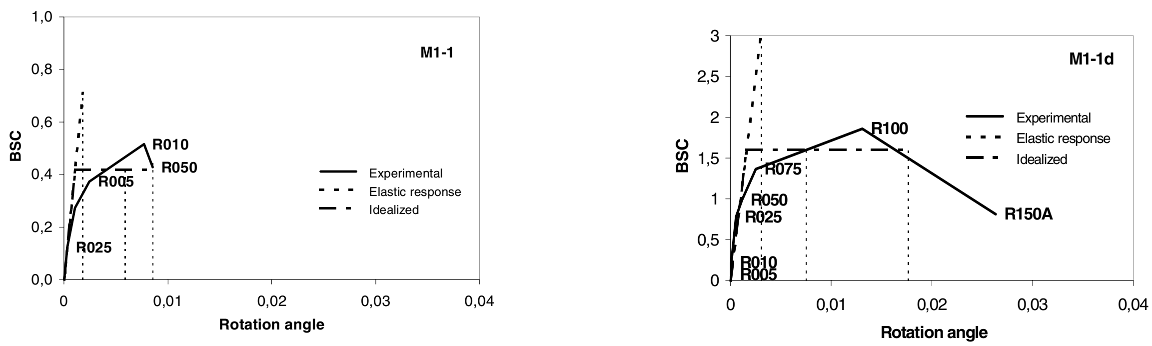


Figure 12. Base shear coefficient - story rotation angle relationships obtained for plain and confined terraced house models

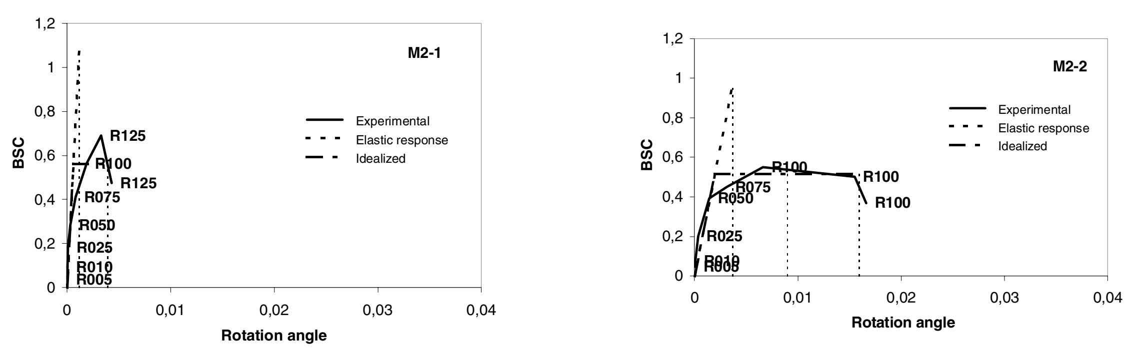


Figure 13. Base shear coefficient - story rotation angle relationships obtained for apartment house models

The values of behavior factors q, resulting from experimental envelopes, are presented in Table 4. In the calculations of the elastic response of the models, effective stiffness of the model structure, i.e. the measured stiffness at the occurrence of the first significant damage to structural walls (damage limit), has been taken into account, and not the initial stiffness of the model, measured before the shaking-table tests. EAVEK (Fajfar [5]), a commercial computer program for seismic analysis of multi-story buildings, has been used. Following the definition of behavior factor q, the response of the elastic structure subjected to shaking-table motion during the testing phase in which the maximum resistance of the model has been attained, has been calculated.

Table 4. Values of structural behavior factor q evaluated from experiments

Model	$q = BSC_e/BSC_{max}$	$q = BSC_e/BSC_u$	$q = (2\,\mu_u - 1)^{1/2}$
M1-1	1.34	1.53	2.09
M1-2	1.84	2.06	2.20
M1-1c	2.44	2.56	2.99
M1-1d	1.63	1.91	2.88
M2-1	1.55	1.91	2.61
M2-2	1.74	1.85	2.84

It can be seen that, generally speaking, the evaluation of the values of behavior factor q on the basis of the observed ductility capacity of the models resulted into higher values than the evaluation on the basis of simple correlation of theoretical elastic and observed base shear responses. It can be also seen that despite the differences observed in the behavior of unreinforced masonry models during the shaking-table tests (brittle behavior of models M1-1 and M2-1 made of calcium silicate units against "ductile" behavior of models M1-2 and M2-2 made of hollow clay units - see Figure 10), the values of behavior factor q, evaluated on the basis of simple definition, are of the same order of magnitude for the cases of both, terraced and apartment house structural types. In the case of the terraced house models with confined structural walls (models M1-1c and M1-1d), however, the observed improved behavior resulted also in the increased evaluated values of behavior factor q.

Table 5. Relationships between the cumulative input and dissipated hysteretic energy at the end of shaking-table tests

Model	Input energy E_{inp} (Nm)	Dissipated hysteretic energy E_{hys} (Nm)	E_{hys}/E_{inp}
M1-1	-	-	-
M1-2	2710	1267	0.47
M1-1c	1866	637	0.34
M1-1d	4813	1778	0.37
M2-1	565	94	0.17
M2-2	2352	750	0.32

Although the resistance and displacement capacity can be used as a measure of energy dissipation capacity, taking advantage of the measured response data, energy dissipation capacity of the tested models has been evaluated on the basis of the measured story shear - story drift (relative story displacement) hysteresis loops. The results of calculations are given in Table 5, where for each of the tested models the input energy, induced by the system during the shaking test by hydraulic actuator (energy demand - Bertero and Uang [6]) and dissipated hysteretic energy, are presented. Cumulative values of input and dissipated hysteretic energy from the beginning to the end of the shaking-table tests, are given in the table, as well as the ratio between both. Unfortunately, because of some data acquisition problems, it was not possible to calculate the input and dissipated hysteretic energy for the case of model M1-1.

In order to make the conclusions on the basis of data presented in Table 5, one has to take into consideration that, although both parameters are a function of structural response, input energy (energy demand) is a function of masses of the structure, ground (shaking-table) acceleration time history and velocity response of the structure, whereas the amount of dissipated hysteretic energy is determined mainly by damage propagation mechanism. Therefore, only the comparison of data obtained for the models of the same structural configuration, is reasonable.

In this regard, the differences in the observed behavior of apartment house models M2-1 and M2-2, made of calcium silicate and hollow clay units, respectively, can be also explained by the differences in energy dissipation capacity. Model M2-2 needed 4-times more input energy E_{inp} than model M2-1 to cause collapse. At the same time, energy dissipation capacity E_{hys} of model M2-2 was almost 8-times greater than energy dissipation capacity of model M2-1. As a result, the difference in E_{hys}/E_{inp} ratio is also significant. It is assumed that similar observation could have been made for the case of unreinforced terraced house models M1-1 and M1-2, if the response records of model M1-1 were available in adequate form for the analysis.

As the data for terraced house model M1-1 are missing, it is not possible to evaluate the effect of partial and complete confinement of structural walls with regard to referential model in terms of energy dissipation capacity. However, the difference between the partly and fully confined models M1-1c and M1-1d can be clearly seen. 2.6-times more input energy has been needed in the case of fully confined model M1-1d to cause collapse than in the case of partly confined model M1-1c, and 2.8-times more energy has been dissipated. Obviously, this resulted in almost the same E_{hys}/E_{inp} ratios in both cases.

The decision about which values of behavior factor q to recommend for the design of the tested types of buildings is therefore not a simple one. Namely, following the simple definition of behavior factor according to EC 8, the differences in the assessed values for unreinforced buildings of both structural configuration types are not significant (Table 4). However, significant differences have been observed in the behavior of the models of both structural types, made of the calcium silicate units on the one hand and those made of hollow clay units on the other. Obviously these observations need to be taken into consideration when making the final proposal.

CONCLUSIONS

The ranges of values of structural behavior factor q, proposed in EC 8 for different masonry construction systems, are adequate. However, the study indicated that the values depend not only on the system of construction, but also on the properties of masonry materials and structural configuration, especially structural regularity, of the building under consideration. Therefore, experimental research is needed for the assessment of a particular value for a particular structural type specified on a national basis within the recommended range of values in the basic document. Although such tests are helpful, the values of behavior factor q cannot be assessed by means of only ductility tests of structural walls.

In the particular case studied, the same values of behavior factor q can be proposed for both structural configurations tested. Namely, q = 1.5 and q = 2.0 for the case of unreinforced and confined masonry, respectively. Although the models of both structural types behave in different ways if constructed with different types of masonry units, the damage limitation criteria do not permit to utilize the advantage of the observed improved ductility and to use greater values of behavior factor q in the design in the more favorable case.

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