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RECENT ADVANCES IN EARTHQUAKE-RESISTANT DESIGN OF MASONRY BUILDINGS: EUROPEAN PROSPECTIVE

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ABSTRACT

In the last decades, considerable experimental and analytical research in seismic behaviour of masonry walls and buildings has been carried out. The investigations resulted into the development of methods for seismic resistance analysis and design, as well as new, seismic resistant technologies and construction systems. After many centuries of traditional use, and decades of allowable stresses verification, clear concepts for limit states verification of seismic resistance of masonry buildings have been recently introduced in the seismic codes. In the paper, some basic aspects of seismic resistance analysis and design of masonry walls and buildings, as well as experimental background for recent proposals of modelling of seismic behaviour of masonry structures is discussed.

KEYWORDS

Masonry buildings, structural walls, construction systems, seismic resistance analysis, seismic codes.

INTRODUCTION

Masonry in a wide variety of forms has been used as structural material for thousands of years. Some very old stone and brick masonry buildings still exist, proving that masonry successfully resists loads and impacts of environment and provides shelter to people and their goods for a long period of time. For their importance and value, many of those buildings have been classified into mankind's historical and cultural heritage of highest category.

As a rule, masonry buildings have been constructed in the past on the basis of experience. If designed, the concept of allowable stresses has been followed until very recently. Although many buildings have been built in earthquake-prone areas, their structures were aimed at resisting vertical loads only. Many such masonry buildings suffered damage and collapsed, and have caused loss of life when subjected to strong earthquakes. Consequently, masonry has been long not considered as a suitable material for the construction of buildings in seismic zones. Modern materials, such as steel and reinforced-concrete, have almost replaced structural masonry in many parts of the world.

However, masonry survived. In the last decades, a considerable research in the behaviour of masonry walls and buildings subjected to seismic actions has been carried out. The behaviour of masonry buildings during earthquakes has been analyzed, and experiments to determine the basic parameters of seismic resistance of masonry walls and buildings have been conducted. The investigations resulted into the development of methods for seismic resistance analysis and design, as well as new, seismic resistant technologies and construction systems. Recent improvements in construction technologies and production of resident-friendly masonry units with good thermal insulation properties have resulted into the growing interest in masonry construction.

Masonry is typical composite structural material, which traditionally consisted of masonry units and mortar. More recently, steel reinforcement and grout are added in order to improve strength and ductility. Different systems of masonry construction (typical examples are shown in Figs. 1 to 3) can be classified into:

- plain masonry (Fig. 1),
- confined masonry, where plain masonry or masonry walls with bed-joint reinforcement are confined with horizontal and vertical r.c. elements, cast after the construction of the walls (Fig. 2), and
- reinforced masonry (Fig. 3).

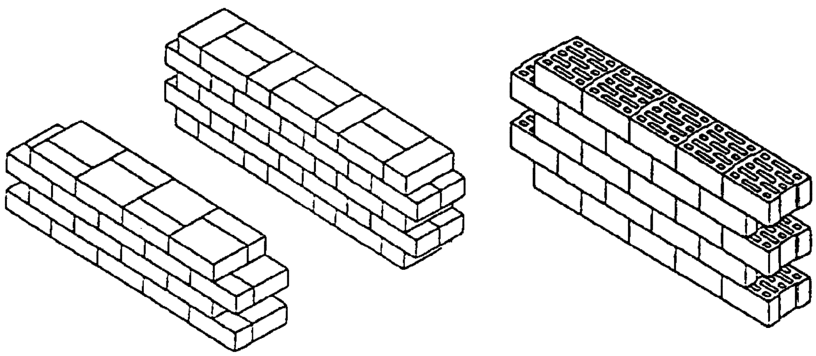

Fig. 1. Examples of bonding arrangements of plain masonry walls (Eurocode 6)

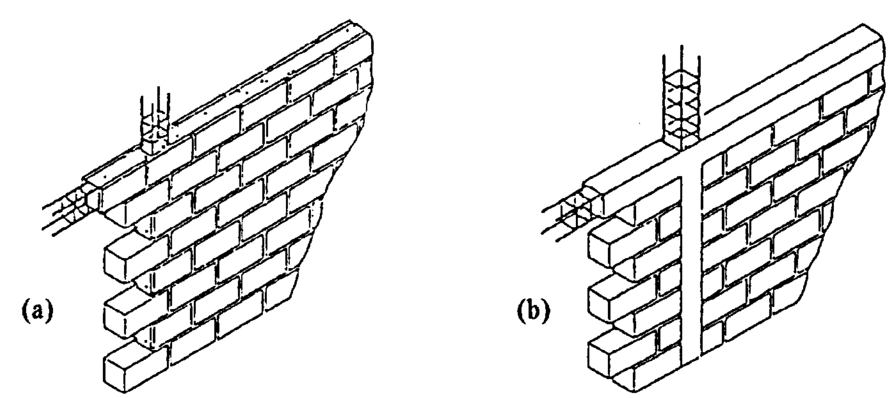

Fig. 2. Confined masonry. Masonry confined within reinforced masonry (a) and (b) reinforced concrete beams and columns (Eurocode 6)

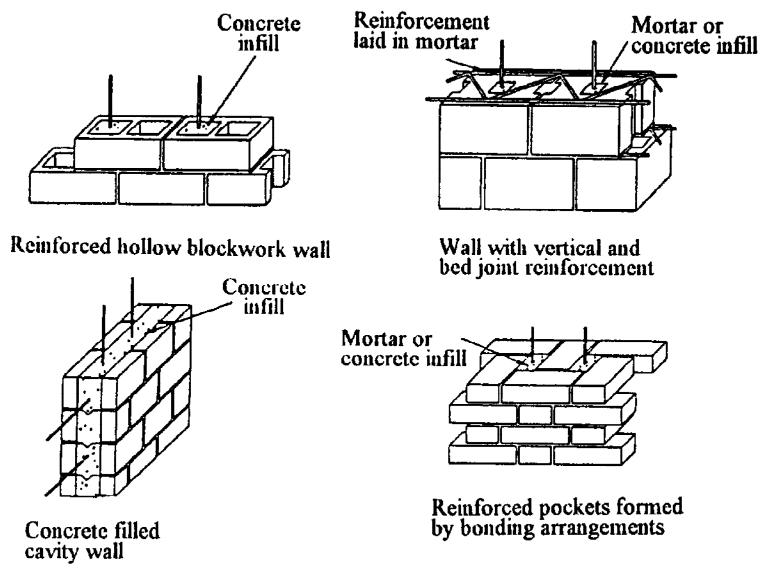

Fig. 3. Examples of reinforced masonry (Eurocode 6)

Because of the way of construction, the seismic behaviour of each system is different: whereas plain masonry represents non-ductile structural material, confined and especially reinforced-masonry represent systems of improved strength and ductility.

Whereas plain and confined masonry construction systems are determined by the shape and quality of masonry units and mortar type, reinforced masonry can be further subdivided according to reinforcing technology. In the case of grouted reinforced hollow unit masonry and reinforced cavity masonry, vertical and horizontal reinforcement is placed in the central part of the wall and grouted with high strength cement grout. In a way, relatively thin shells of hollow masonry units, which usually have two large vertical holes and chanelled webs to accomodate vertical and horizontal reinforcement, respectively, and outer wythes of a wall in the case of reinforced cavity masonry, act as a form when grouting the reinforcement. Specific technology is required for grouting the hollow parts of the walls containing reinforcing steel. For this reason, the construction of grouted reinforced masonry is limited to developed countries, such as U.S.A. and New Zealand. In most other countries, however, normal or specially shaped hollow or perforated masonry units are used to accomodate reinforcement: horizontal steel is placed in the mortar within the bed-joints or suitable grooves in the units, and vertical steel is placed in appropriate pockets, cavities or holes in the units, embedded in mortar or concrete.

It is generally believed that reinforced grouted masonry is *the* earthquake-resistant type of masonry construction. This, in a way, is certainly true: because of its similarity with reinforced-concrete shear-wall structures, a reinforced grouted masonry structure would be most probably stronger than masonry structure of the same structural configuration and size, built in one of the other possible masonry construction systems. However, as post-earthquake observations and experiments indicate, buildings built in traditional masonry construction systems, which are widely used in most earthquake-prone countries and include plain, confined, as well as a variety of types of reinforced masonry, can be also earthquake resistant, if adequately designed and constructed.

In the last decade, a comprehensive and coordinated research in earthquake resistant masonry construction has been carried out in the U.S.A. (Noland, 1990). A number of excellent papers regarding structural design of reinforced grouted masonry have been published as a result of research (see for example: Shing *et al.*, 1990; Priestley and He, 1990; Leiva and Klingner, 1993; Seible *et al.*, 1994). Most of practical design issues, resulting from this and other research, are compiled in the book written by Paulay and Priestley (1992). Taking this into consideration, and having in mind at the same time a large number of masonry buildings in many earthquake-prone countries of the world, which will not be constructed in reinforced grouted masonry construction systems, the author of this contribution decided to focus his discussion on earthquake-resistance analysis and design of traditional, not grouted, masonry construction.

Recent advances in European design codes, where for the first time a clear concept for limit states verification has been introduced also for masonry structures, also deserve broader attention. Namely, after several years of preparation and drafting, European standard Eurocode 8: *Design provisions for earthquake resistance of structures* (EC 8) has been recently approved by CEN (European Committee for Standardization) members and accepted as pre-standard (Eurocode, 1994; 1995a). *Specific rules for masonry buildings* in EC 8, as a supplement to the requirements of Eurocode 6: *Design of masonry structures* (Eurocode, 1995b), have been prepared on the basis of recommendations of the CIB Commission W23 on wall structures, subcommission for masonry structures in seismic areas (International, 1987), an international working group, not limited only to representatives of CEN member states. Besides the introduction of concept for limit states verification of seismic resistance, some basic design criteria and construction rules regarding materials and types of construction, as well as basic guidelines regarding structural analysis are also given in the masonry part of EC 8. However, no detailed prescriptions can be found regarding structural modelling and numerical calculations. Therefore, some recent proposals of methods for earthquake resistance analysis and design of masonry walls and buildings, which follow the design philosophy of Eurocodes, and are based on the results of experimental research, will be discussed in this contribution, too.

GENERAL CONSIDERATIONS AND DESIGN SEISMIC ACTION

The safety of a structure against earthquakes is a probabilistic function, which depends on the expected seismic action and capability of the structural system to resist the earthquake. According to Eurocode 8, the following general relationship shall be satisfied for all structural elements:

$$E_d \leq R_d, \tag{1}$$

where R_d is the design resistance capacity, calculated by taking into account characteristic strength values and partial safety factors γ_M of elements' materials, and E_d is the design value of the action effect, which includes seismic design situation, ie. design seismic action A_{Ed}.

Design seismic action depends on dynamic characteristics of the ground motion and structure under consideration. Although masonry is considered as brittle structural material, the experiments and analyses of earthquake damage show that even plain masonry buildings possess a relatively high energy dissipation capacity, what makes posssible the reduction of elastic seismic forces. Hence, the general equation, which determines design seismic action A_{Ed}, can be applied also in the case of masonry structures. According to EC 8, seismic action is given in the form of elastic response spectrum S_e (T), the shape of which is defined by 7 parameters and depends on subsoil characteristics (Fig. 4), whereas the amplitudes depend on *design ground acceleration* a_g. Hence, the following equation determines the design seismic action:

$$A_{Ed} = \frac{S_e(T)}{q} W, \tag{2}$$

where A_{Ed} is the design seismic action, $S_e (T) = a_g\, S\, \eta\, \beta_o$ is the ordinate of the elastic response spectrum, a_g is the design ground acceleration, expressed as a fraction of gravity (g = 9,81 m/s^2), S is the soil parameter, η is the damping correction factor, β_o = 2,5 is the maximum normalized spectral value assumed constant, q is the behaviour factor (elastic force reduction factor), and W is the weight of the building above ground level.

Values of parameters describing elastic response spectrum

Subsoil class	S	β_0	k_1	k_2	T_B (s)	T_C (s)	T_D (s)
A	1,0	2,5	1,0	2,0	0,10	0,40	3,0
B	1,0	2,5	1,0	2,0	0,15	0,60	3,0
C	0,9	2,5	1,0	2,0	0,20	0,80	3,0

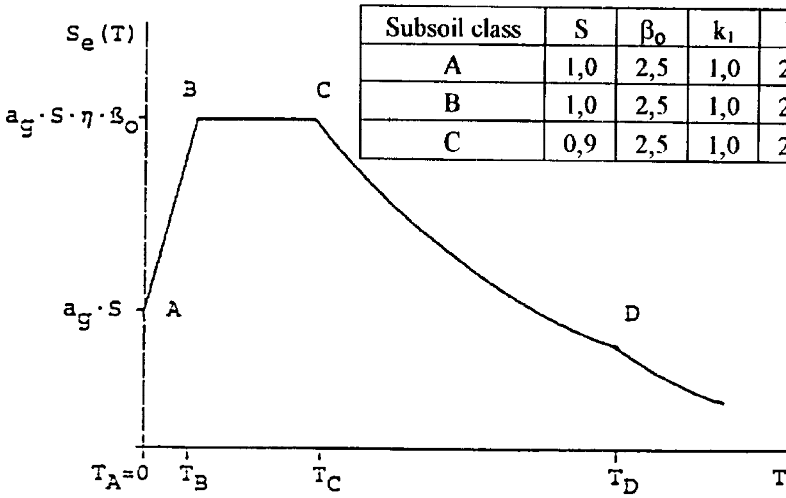

Fig. 4. Elastic response spectrum proposed by Eurocode 8

Assuming S = 1,0 (normal soil conditions), η = 1,0 (at 5 % viscous damping) and β_0 = 2,5 (natural period of vibration between 0,1 s and 0,4 s), which is the normal case of a masonry building, the values of design base shear coefficients (BSC) for different systems of masonry construction are obtained, as given in Table 1.

Table 1. Design values of base shear coefficient - Eurocode 8

Seismic intensity (MSK): Design ground acceleration:	VII. 0,1 g	VIII. 0,2 g	IX. 0,3 g
Plain masonry: q = 1,5	0,17	0,33	0,50
Confined masonry: q = 2,0	0,125	0,25	0,375
Reinforced masonry: q = 2,5	0,10	0,20	0,30

In EC 8, the values of behaviour factors, as well as parameters defining the normalized elastic response spectra, are given as indicative. However, no indication is given regarding the values of design ground accelerations. All these values should be specified by National Application Documents (NAD) of CEN member and associate member states. The values used in the calculation of BSC in Table 1 have been specified by Slovenian NAD. As can be seen, the resulting values of design BSC are relatively high. In fact, in the case of Slovenia, they are 100 % greater than they used to be for the verification of ultimate limit state according to former Yugoslavia's seismic code!

In a qualitative and simplified way, the well-known definition of behaviour factor (force reduction factor) is explained in Fig. 5a, where the seismic response envelope curve of an actual structure, idealized by the linear elastic - perfectly plastic envelope, is compared with the response of a perfectly elastic structure having the same initial elastic stiffness characteristics. As a result of energy dissipation capacity of the actual structure, which is expressed by the global ductility factor $\mu = d_u/d_e$, there is usually no need that the structure be designed for strength, i.e, for the expected elastic seismic load H_e. The structure should be designed for the ultimate design load H_u . The ratio between the two is called behaviour factor $q = H_e/H_u$. In the case where the structure has been designed for load H_u', i.e. for ultimate load H_u reduced by a global safety factor γ_Q, the reserve strength (overstrength) results in an increased behaviour factor $q' = \gamma_Q q$.

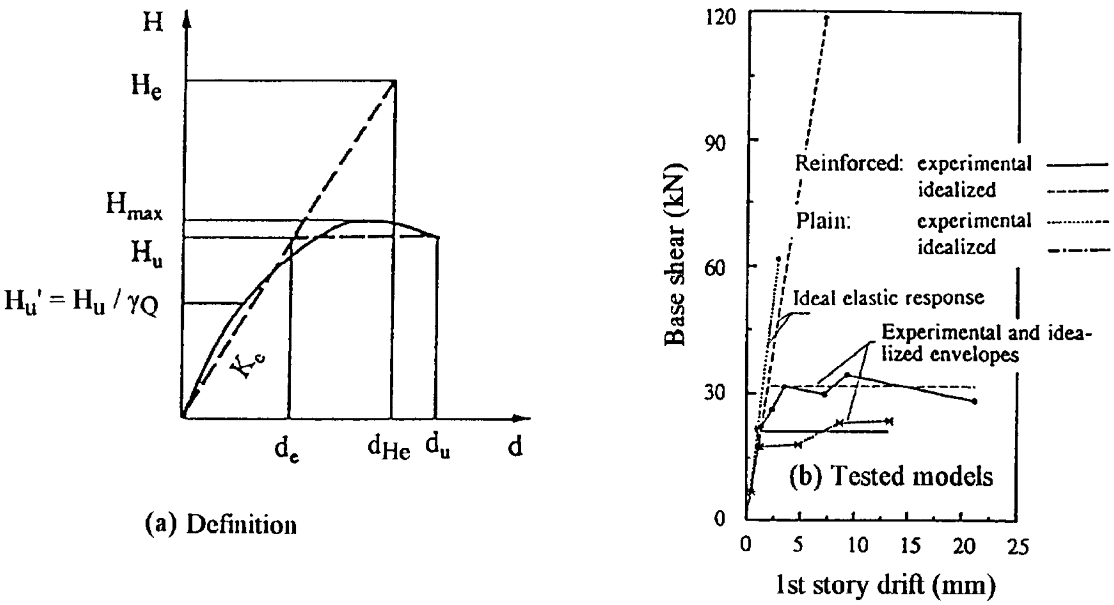

Fig. 5. Evaluation of behaviour factors (after Tomaževič and Weiss, 1994)

Two three-story, plain and reinforced-masonry building models of identical structural configuration have been recently tested at ZRMK by subjecting them to a series of simulated seismic ground motion on a shaking-table (Tomaževič and Weiss, 1994). Taking advantage of the obtained experimental data, an attempt has been made to verify the values of q-factors proposed by EC 8. Namely, the analysis of dynamic behaviour of the tested plain and reinforced-masonry building models has indicated the predominant first mode of vibration, and story mechanism that defines the failure mode. Consequently, the global behaviour of the models, referred to as prototype buildings, defined by the idealized linear elastic - perfectly plastic base shear - first story drift hysteresis envelope, can be used in order to evaluate q-factors according to definition explained in Fig. 5a.

To compare the elastic and nonlinear behaviour of the tested models, the responses of hypothetical models with elastic stiffness characteristics to maximum shaking-table accelerations, which the actual models had resisted, have been calculated. The results of elastic analysis are correlated with corresponding experimental and idealized hysteresis envelopes in Fig. 5b. As a result of comparison, the values of behaviour factors $q = H_e/H_u = 3.74$ and $q = 2.84$ have been obtained for the reinforced- and plain-masonry building models, respectively. If the requirements of Eurocode 8 were taken into account in the design of prototype buildings (which, in fact, have been designed by an elastic design method) it could be seen that because of material safety factors, which should be taken into account in the calculations, the design resistance capacity of the tested buildings would be lower than experimentally obtained. A substantial reserve in strength (overstrength) would be observed, what further increases the observed q factor values. On the basis of these considerations it follows that the proposed EC 8 values of q factors for plain and reinforced masonry buildings (1.5 and 2.5, respectively) are conservative. This might be one of the main reasons why masonry buildings, designed for lower values of BSC by former seismic codes, resist earthquakes. However, further experimental and analytical studies are needed in order to confirm these observations.

It is clear that, once the safety level of the structure against earthquakes had been decided upon, the determination of both, design resistance capacity and design seismic action is an interdependent procedure. The philosophy of determination of partial and global safety factors used in the calculation of R_d and structural behaviour factors used in the calculation of S_d should be compatible, otherwise the verification according to (1) would lead to wrong conclusions. In order to estimate accurate values of parameters appearing on both sides of equations (1) and (2), realistic data about the seismic behaviour of the structure under consideration and expected seismic ground motion should be known. Since limit states of structural behaviour are considered, mathematical models should be developed on the basis of experimentally verified theoretical hypotheses that take into account the actual behaviour of structures.

IN-PLANE RESISTANCE OF MASONRY WALLS

General

Structural wall is the basic resisting element to seismic loads in a masonry structure. Because of specific requirements regarding structural configuration of masonry buildings in most seismic codes, which are mainly based on tradition and experience, such as distribution of walls in both orthogonal directions, geometric requirements for shear walls (effective height, size and position of openings) and connection between walls and floors, out-of-plane resistance to seismic action is usually not critical. Although it may be verified by usual methods for verification of flexural strength of masonry walls, out-of-plane behaviour will not be the subject of discussion in this contribution. Typical methods for the verification of in-plane resistance of structural walls, as contributing elements to seismic resistance of the whole structure, will be only discussed.

When subjected to in-plane loads, masonry walls fail in different possible failure modes. As the earthquake damage observations and subsequent laboratory simulation of seismic behaviour of masonry walls show, in all systems of masonry construction the following typical failure modes can be distinguished:

- sliding shear failure, which is characterized by the shearing of the wall on horizontal mortar joints at low level of vertical load;
- shear failure, which is characterized by diagonal cracks in the wall, caused by the principal tensile stresses in the wall which exceed the tensile (shear) strength of the masonry;
- flexural failure, which is characterized by the yielding of reinforcing steel at the tensioned side and crushing of masonry units at the compressed zone of the wall's section.

As has been found, the mode of failure depends on the geometry of the wall, boundary conditions, magnitude and type of acting loads, as well as quality of masonry materials. Typical non-dimensional axial-load - bending moment interaction diagram showing the dependence of failure modes on the level of compressive stresses in the wall is shown in Fig. 6. It is not simple to model the non-elastic, non-homogeneous and unisotropic character of masonry. In order to simplify the analysis and design, the values of sectional forces, stresses and strains are determined, based on the gross-sectional geometrical characteristics of the walls, and assuming the elastic, homogeneous and isotropic global properties of masonry as structural material. Practical equations for the calculation of lateral resistance and deformability of masonry walls have been proposed on the basis of extensive series of tests of plain and reinforced masonry walls subjected to simulated earthquake loads. Although simplified, these equations reflect the actual failure mechanisms. Therefore, the values of mechanical quantities, which determine the load-bearing capacity and deformability of masonry walls in these equations, such as:

- compressive strength of the masonry (f),

- shear (f_v), or tensile strength of the masonry (f_t),
- modulus of elasticity (secant modulus E),
- shear modulus (G), and
- ductility factor (μ),

should be determined experimentally by testing procedures which are compatible with the experiments on the basis of which the equations for calculations have been developed. Otherwise, errors can be made in predicting the seismic resistance of the building. In order to determine the seismic resistance of masonry walls, tests should be carried out by subjecting the specimens to similar loading conditions as they are subjected to in a building. In order to simulate the observed failure mechanism, the specimens should be of similar geometry and should be supported in a similar way as in the building's structural system.

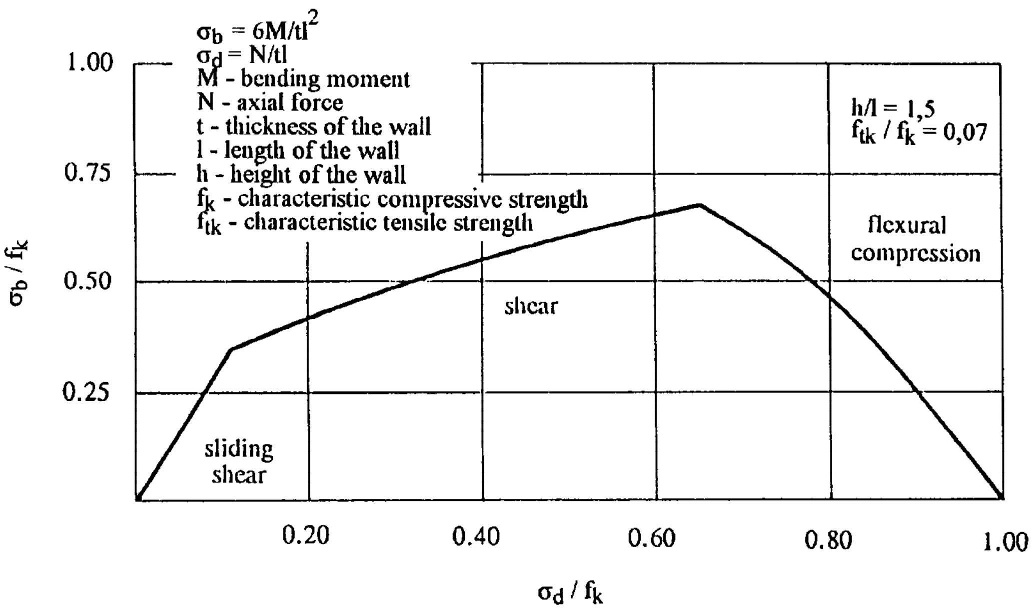

Fig. 6. Non-dimensional axial-load - bending moment interaction diagram, showing the dependence of failure modes of masonry walls on the level of compressive stresses in the walls

According to design philosophy of Eurocodes, characteristic values of material mechanical properties should be taken into account in the calculations. In the ultimate limit state verification of seismic resistance of masonry buildings, the following partial safety factors γ_M for masonry properties, depending on the categories of manufacturing control and of execution, should be used:

Table 2. Partial safety factors γ_M for material properties (Eurocode 8)

γ_M		Category of execution A	B	C
Category of manufacturing control	A	1,2	1,5	1,8
	B	1,4	1,7	2,0

Shear Resistance

Because of mechanical properties of masonry and geometry of structural walls, shear failure is the most common type of failure of a structural wall, subjected to seismic loads. It has been generally accepted that shear resistance depends on vertical compression stresses in the walls. However, two basically different hypotheses, which lead to virtually same results, have been developed in order to physically model the shear failure mechanism of a plain masonry wall.

In the first case, which has been also accepted for shear resistance evaluation in EC 6: *Design of masonry structures*, shear strength of masonry f_v is defined according to friction theory by expression:

$$f_v = f_{vo} + \mu\, \sigma_d, \tag{3}$$

where f_{vo} is the shear strength under zero compressive stress, μ is the constant defining the contribution of compressive stresses, and σ_d is design compressive stress, perpendicular to shear. Values of f_{vo} and μ should be determined by tests. Accordingly, design shear resistance V_{Rd} of a plain masonry wall is given by:

$$V_{Rd} = \frac{f_{vk}\, t\, l_c}{\gamma_M}, \tag{4}$$

where t is the thickness of the wall, and l_c is the length of the compressed part of the wall. As can be seen, characteristic value of shear strength f_{vk} is considered in (4).

In the second case, it is assumed that diagonal cracks at shear failure are caused by the principal tensile stresses, which develop in the wall when subjected to a combination of vertical and lateral load. By considering the masonry wall as an elastic, homogeneous and isotropic structural element, the basic equation for the evaluation of the shear resistance of plain masonry walls has been proposed by Turnšek and Čačovič (1971). The equation was later modified (Turnšek and Sheppard, 1981) to take into account the influence of geometry of the wall and distribution of actions (ratio between vertical and lateral load) at ultimate state. Principal tensile stresses σ_T developed in the wall at the attained maximum resistance H_{max} are given by:

$$\sigma_T = f_t = 0{,}5\sigma_d + \sqrt{(0{,}5\sigma_d)^2 + (b\tau_{H\,max})^2}, \tag{5}$$

where f_t is the tensile strength of the masonry, τ_{Hmax} is the average shear stress in the wall at the attained maximum resistance H_{max}, and b is the shear stress distribution factor, depending on geometry of the wall and ratio between lateral and vertical load at the attained maximum resistance H_{max}. Accordingly, design shear resistance of the wall V_{Rd} is calculated by:

$$V_{Rd} = \frac{1}{\gamma_M}\, 0{,}9\, A_w\, \frac{f_{tk}}{b} \sqrt{\frac{\sigma_d \gamma_M}{f_{tk}} + 1}, \tag{6}$$

where A_w is the horizontal cross-sectional area of the wall. Again, characteristic value of tensile strength f_{tk} is considered in the calculation. Coefficient 0,9 in (6) is a consequence of definition of tensile strength (maximum principal tensile stress in a hypothetical elastic, homogeneous and isotropic wall at the attained maximum resistance) and the idealization of the wall's hysteresis envelope. The value of 0,9 resulted from an extensive series of tests of more than 60 walls (Tomaževič and Žarnić, 1986).

Evidently, τ_{Hmax} may be considered as equivalent to shear strength obtained by (3):

$$\tau_{H\,max} = f_v = \frac{f_t}{b}\sqrt{\frac{\sigma_d}{f_t} + 1}, \tag{7}$$

When failing in shear, plain masonry walls behave as brittle structural elements with little energy dissipation capacity, especially when subjected to high compressive stresses. In order to improve lateral resistance and ductility, masonry walls are reinforced with steel reinforcement in one of the existing systems. Generally, the shear resistance of reinforced masonry walls depends on different mechanisms, such as dowel action of vertical steel, the combination of truss and arch-beam action of vertical and horizontal reinforcement and masonry, as well as interlocking between the parts of the walls separated by diagonal cracks. Few attempts have been made to theoretically model this mechanism (Tassios, 1984; Wakabayashi and Nakamura, 1984; Shing *et al.*, 1993). However, because of the complexity of mechanisms, the proposed equations are not all suitable for practical design, and the validity of theoretical solutions is limited to specific cases.

In practical calculations, shear resistance of reinforced masonry walls is calculated as a sum of contributions of masonry and reinforcement. For example, the following equation is given in EC 6:

$$V_{Rd} = V_{Rd1} + V_{Rd2}, \tag{8}$$

where V_{Rd1} is given by (4) and V_{Rd2} is calculated by:

$$V_{Rd2} = 0{,}9\ d\ \frac{A_{sw}}{s}\frac{f_{yk}}{\gamma_s}, \tag{9}$$

where d is the effective depth of the wall, A_{sw} is the area of shear reinforcement, s is the spacing of shear reinforcement, f_{yk} is the characteristic yield strength of steel, and γ_s is the partial safety factor for steel (γ_s = 1,0).

In the case where masonry walls are reinforced with vertical steel, part of the shear capacity of the wall can be also attributed to vertical reinforcement acting in bending (dowel action). However, as the post-earthquake damage observations and experiments indicate, vertical steel alone is not capable to contribute to the shear resistance of masonry. Walls reinforced only with vertical reinforcement will fail in shear, despite their predicted flexural behaviour.

Also, as the test results indicate, in the case of masonry with mortar bed-joint reinforcement, the tension capacity of horizontal steel, as assumed in (9) cannot be fully utilized because of the bond failure between mortar and steel. The results of an extensive series of tests of 64 reinforced masonry walls with horizontal mortar bed-joint reinforcement and with or without vertical edge reinforcement (Table 3) indicate that horizontal steel acts in tension when subjected to lateral loads, as expected theoretically (Fig. 9a).

Table 3. Configuration and reinforcement of tested walls

Series	Type of blocks	Mortar grade (MPa)	Geometry of wall (h/l)	Horizontal steel ρ (%)	Vertical steel ρ (%)
A	concrete	10-15	1.5	0.14-0.50	none
B	ceramic	10-15	1.5	0.14-0.50	none
C	concrete	10	1.25-2.3	0.14-0.50	0.26
D	concrete	10	1.25-2.3	0.14-0.50	0.52

Accumulation of permanent strain in horizontal reinforcement after unloading has been observed before yielding, indicating that horizontal steel keeps the cracked parts of the wall, splitting because of vertical load, together by horizontal prestressing (Tomaževič and Žarnić, 1986; Tomaževič and Lutman, 1989). Consequently, many uniformly distributed diagonal cracks develop in a horizontally reinforced wall at shear failure. However, as can be seen in Table 4, where the results of tests are summarized in terms of effectiveness of steel, expressed as a ratio between the actual maximum average tension, measured in horizontal bars H_{rh}, and yielding capacity of reinforcement $H_{rh,y}$, the loss of bond between mortar and steel in the bed-joints, as well as crushing of blocks prevented yielding of horizontal steel in most case.

It might not be possible to predict the effectiveness of horizontal reinforcement by calculation, since it depends on the shape and quality of masonry units and mortar, as well as amount and way of anchoring the reinforcing bars at the ends of the wall. A tendency of decreasing in effectiveness of horizontal reinforcement with increased reinforcement ratio has been observed in all series of tests, the reinforcement being best utilized in the case of stirrups, bent around the vertical steel (walls of series C and D).

Table 4. Effectiveness of horizontal steel H_{rh} / $H_{rh,y}$ in dependence on reinforcement ratio ρ

ρ (%)	Series A	Series B	ρ (%)	Series C	Series D
0.18	0.84	0.89	0.14	0.83	0.79
0.32	0.58	0.63	0.28	0.65	0.78
0.38	0.39	0.46	0.50	0.65	0.61

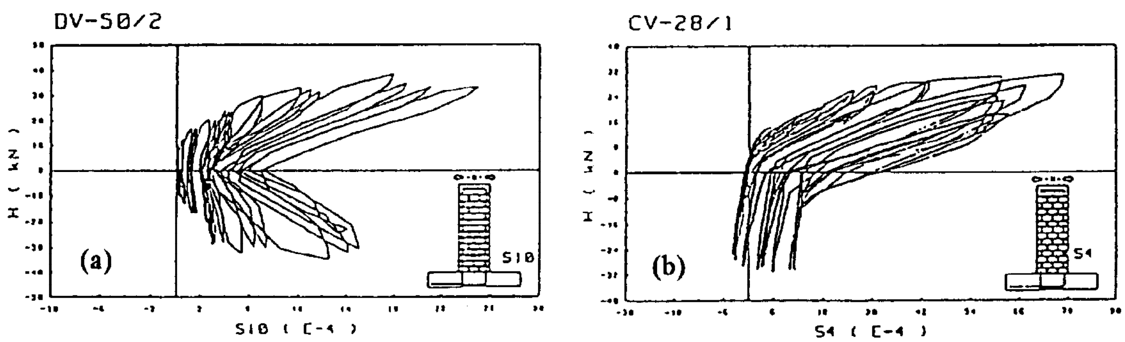

Fig. 9. Typical relationship between the lateral load
and strain in horizontal (a) and vertical steel (b)

The experiments confirmed the idea of assessing the shear resistance of reinforced masonry walls as a sum of contributions of masonry and reinforcement. By analyzing the test results it can be seen that masonry itself is carrying an important part of lateral load, acting on the wall, until the wall's final collapse. However, the contribution of horizontal steel is limited because of premature bond failure. It should be reduced by an experimentally determined capacity reduction factor ϕ_h:

$$\Phi_h = \frac{H^r_{max} - H_{max}}{H_{rh,y}} , \tag{10}$$

where $H_{max}{}^r$ is the experimental maximum resistance of a horizontally reinforced wall, and H_{max} is the experimental maximum resistance of referential plain masonry wall. A modification of (8) is therefore proposed, where, in the presence of shear reinforcement, dowel action of vertical steel may be also taken into account:

$$V_{Rd} = V_{Rd1} + \phi_h V_{Rd2} + V_{Rd3}, \tag{11}$$

where ϕ_h is the vertical reinforcement capacity reduction factor, and V_{Rd3} is the contribution of vertical reinforcement (dowel action).

According to the requirements of EC 6, no account should be taken of the strength of the reinforced concrete or reinforced masonry members, which confine masonry walls, in the design of confined masonry for seismic loads. This is in good agreement with recent experiments, where improved energy dissipation capacity, but little increase in strength has been reported (Alcocer and Meli, 1995).

Flexural Resistance

Flexural failure of a plain masonry wall is sometimes defined by the initiation of horizontal cracking at the tensioned side of the most stressed section of the wall. However, the lateral resistance of the wall is not yet attained at that time. As indicated by experiments, the stresses in the compressed part of the cross-section of the wall, reduced because of the horizontal tensile cracks at the opposite side, increase until the compressive strength of masonry is attained.

Taking into consideration fact that the behaviour of masonry subjected to uniaxial compression is similar to that of concrete, in determining the flexural, or as it is sometimes called, flexural-compression resistance capacity of a section, a rectangular stress distribution as in the case of concrete may be assumed. The shape of the compressive stress block proposed by EC 6 is shown in Fig. 10.

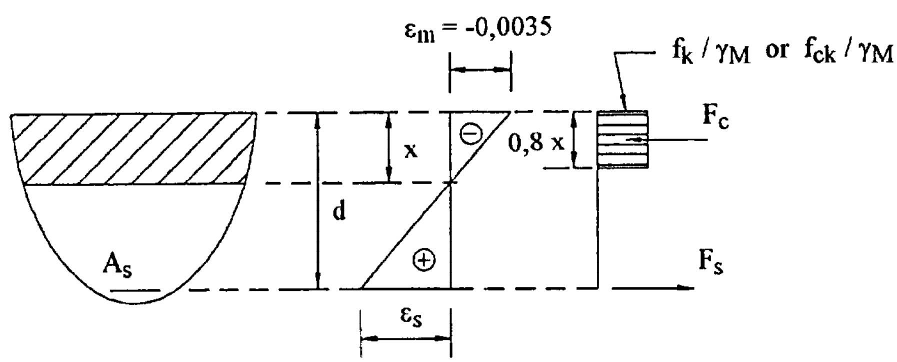

Fig. 10. Simplified rectangular stress block, proposed by Eurocode 6.

On the basis of this assumption, the following equation for the calculation of the design flexural capacity M_{Rd} of a plain masonry wall's section can be derived:

$$M_{Rd} = 0{,}5\, \sigma_d\, t\, l^2 \left(1 - \frac{\sigma_d\, \gamma_M}{f_k}\right), \qquad (12)$$

where l is the length of the wall and f_k is the characteristic compressive strength of masonry. In the case of a plain masonry wall, however, the basic condition of equilibrium of forces acting on the wall as a rigid body should be also fulfilled.

In the case of masonry walls, reinforced at the edges with concentrated vertical reinforcement, yielding of vertical steel takes place at the tensioned side of the most stressed section of the wall, with simultaneous crushing of masonry units and grout at the compressed side (Fig.9b). Taking into account similar assumptions regarding stress and strain distribtution, the following equation is obtained in the case of symmetrically reinforced masonry wall section:

$$M_{Rd} = 0{,}5\,\sigma_d\,t\,l^2\left(1 - \frac{\sigma_d\,\gamma_M}{f_k}\right) + (l - l')\,A_s\,\frac{f_{vk}}{\gamma_s}, \tag{12}$$

where l' is the distance of the centre of vertical steel from wall's edge, and A_s is the area of symmetrical vertical steel at the edges of wall. As can be seen, the contribution of vertical reinforcement is simply added to the flexural-compression capacity of a plain masonry section. Similar equations can be derived for walls with distributed vertical reinforcement.

Since the elements used for the confinement of masonry wall are not load-bearing elements, but are aimed only at improving ductility and energy dissipation capacity of masonry structures, they are, according to requirements of EC 6, not considered in assessing the flexural resistance of walls.

Deformability. In order to make the distribution of design seismic shear onto the walls in the story under verification possible, the stiffness of individual walls should be known. In the general form, effective stiffness of the wall K_e is given by:

$$K_e = \frac{G\,A_w}{1{,}2\,h\left(1 + \alpha'\frac{G}{E}\left(\frac{h}{l}\right)^2\right)}, \tag{13}$$

where:

α' - factor, determined by the position of the bending moment inflection point (α' varies from 0.83 to 3.33 for the cases of fixed-ended and cantilever walls, respectively). If (13) is used for the determination of the value of masonry's shear modulus from the results of lateral resistance tests, modulus of elasticity E should be previously known from vertical compression tests.

SEISMIC RESISTANCE ANALYSIS

No specific procedures or algorithms for seismic resistance analysis are required by the new EC 8. According to EC 8, the structural model of the building should adequately represent the stiffness properties of the entire system. The stiffness of structural walls should be evaluated by considering both their flexural and shear deformability, but also axial deformability, if relevant. Floor diaphragms may be considered as rigid in their own plane, and frame analysis may be used, provided that coupling beams have been included in the structural model. In the case where linear analysis is used for the distribution of the total base shear among structural walls, the redistribution of shear to the walls is possible, provided that global equilibrium is satisfied and the shear in any wall is neither reduced more than 30 % nor increased more than 50 %.

In the case of masonry buildings, which fulfil the basic requirements for rigid horizontal diaphragm floor action, concentration of damage in the bottom-most story at ultimate state has been observed by both, post-earthquake observations and experiments. This indicates that the seismic resistance of masonry buildings generally depends on the lateral load-bearing capacity of the walls in the bottom-most story, where the maximum seismic loads (base shear and bending moments) usually develop. If such is the case, the seismic resistance of the building can be determined simply by the curve which defines the relationship between the lateral resistance and corresponding story drift of the critical story - story resistance envelope, represented by full-line curve in Fig. 5a.

The idea of verifying the seismic resistance of masonry buildings by comparing the story resistance envelope to the design seismic action and global ductility requirements has been first used after the earthquake of Friuli of 1976, when a simple method for the seismic resistance analysis of plain masonry buildings has been developed in Ljubljana (Tomaževič, 1978; Tomaževič and Turnšek, 1982). This method, originally known as "the method POR", was based on the shear resistance of plain masonry walls, failing as symmetrically fixed piers, simulating story mechanism action of plain masonry buildings at ultimate state. In the today's form, the same idea of calculating the story resistance envelope can be applied to modern, reinforced-masonry buildings, where the coupled shear-wall mechanism of the walls prevails. The method complies with the basic requirements of EC 8 for structural analysis of masonry buildings.

Since all structural walls in the critical story contribute to the seismic resistance of the building, their individual contribution, which is a function of their resistance and inter-story drift, should be evaluated. The assumption of rigid horizontal diaphragm action of floors is usually taken into account and the redistribution of seismic loads from one wall to another is carried out by assuming that the walls as a whole, and not their critical sections, behave as ductile structural elements. In order to make the calculations simple, the actual hysteretic behaviour of a wall is represented by an idealized bi-linear hysteresis envelope, defined as shown in Fig. 11.

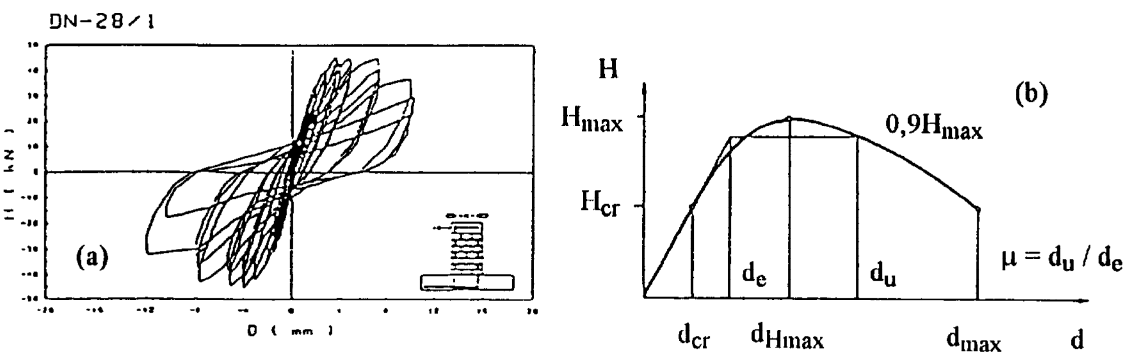

Fig. 11. Experimentally obtained lateral load - displacement hysteresis loops (a) and (b) idealized hysteresis envelope

By analyzing the observed damage patterns and dynamic response of plain and reinforced-masonry buildings, the following main conclusions can be drawn regarding the mechanism of seismic behaviour and subsequent mathematical modelling (see for example Paulay and Priestley, 1992; Tomaževič and Weiss, 1994):

- In the case of plain masonry walls and rigid floors, the flexural capacity of the wall's top and bottom sections is to low to activate the flexural capacity of horizontal elements, tie-beams and floor slabs. Horizontal cracks occur, indicating the pier action of walls at ultimate state and story mechanism.
- If reinforced, however, the structural walls behave as vertical cantilevers, coupled with horizontal elements. The distribution of bending moments induced in the walls by seismic loads depends on the rigidity of the walls and horizontal elements. The flexural capacity of spandrel and lintel beams, tie-beams and floors improves the lateral resistance and energy dissipation capacity of the structural system. It should be therefore taken into account when designing the reinforced-masonry walls for seismic actions.
- When subjected to earthquakes, the first mode of vibration is predominant in the case of both, plain and reinforced-masonry buildings. Therefore, as good approximation, the triangular distribution of seismic forces and displacements along the height of the building can be assumed.

On the basis of relevant mechanism, story resistance - displacement envelope of the critical story can be evaluated as a superposition of lateral resistance and deformability characteristics of all contributing walls in the story. The assumption of rigid horizontal diaphragm action of floors makes possible the distribution of seismic actions onto the walls according to their stiffnesses. A step-by-step (push-over) procedure is used. Assuming triangular distribution of displacements along the height of the building, the structure is displaced by a small value. The walls are deformed correspondingly and the reaction forces of all walls which resist to the imposed displacements, are calculated. Since story mass-centers are displaced (the resultant of seismic inertia forces is acting in the mass-center of each story), torsional rotation takes place if the stiffness-center does not coincide with the mass-center. In such a case, the calculated displacements of the walls are corrected by taking into account the torsional effects. The calculation is repeated step-by-step by increasing the imposed displacements. Once the walls enter into the non-linear range, the structural system of the building and stiffness matrices are accordingly modified. As a result of calculation, relationship between the resistance of the critical story and interstory drift - resistance envelope is obtained.

The behaviour of each wall in the story can be followed throughout the calculation. Usually, the state of the walls at the characteristic points of the story resistance envelope, such as elastic limit (d_e), maximum resistance (d_{Hmax}) and ultimate displacement (d_u) is indicated, so that the critical walls in the story can be identified and redesigned, whenever necessary. In order to verify the validity of the proposed method, the calculation of the story resistance envelope has been carried out for the case of three-story masonry building models, tested in Ljubljana (Tomaževič and Weiss, 1994). As can be seen in Fig. 12, where the calculated first story resistance envelope of reinforced-masonry building model is compared to the experimentally obtained one, good correlation has been obtained in this particular case.

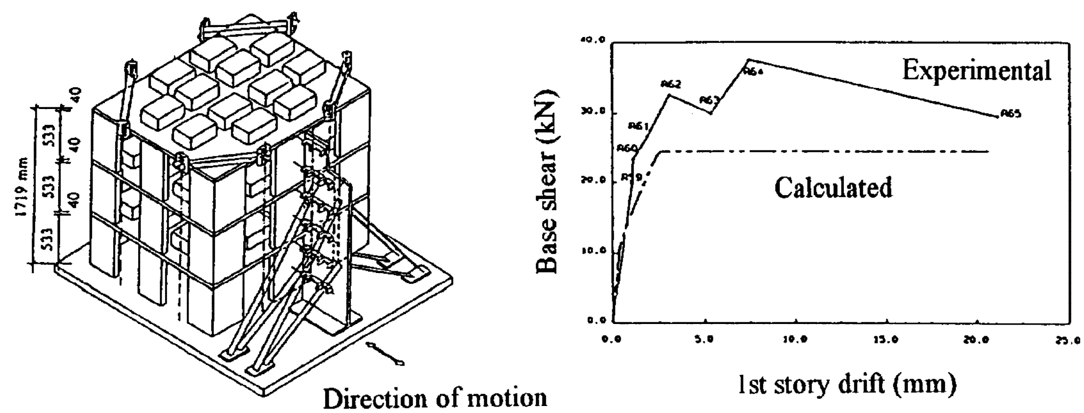

Fig. 12: Correlation between the experimentally obtained and calculated resistance envelope of a reinforced masonry building model

CONCLUSIONS

Extensive, even internationally coordinated research in seismic behaviour of masonry structures in the last decades made possible the improvement in technologies and development of up-to-date limit states methods for structural analysis and earthquake resistant design. Some of the recent advances have been discussed in this contribution. However, because of the great variety of forms of masonry construction systems, it is difficult to propose unified models. In the case of reinforced masonry, for example, balance has to be found between the load bearing capacity of masonry units on the one side, and reinforcement on the other, including bond between reinforcing steel and mortar, for each particular type of masonry. Since practically all parameters needed in numerical verifications need to be determined experimentally, different existing methods, which may yield to different results, have yet to be harmonized. Whereas the seismic behaviour of some of the advanced masonry construction systems, such as reinforced grouted masonry, has been systematically investigated, not enough is known about the others, such as confined and traditionally reinforced masonry. Hopefully, since masonry construction is getting momentum because of its ecological qualities, some of remaining problems will be resolved in the near future.

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