Sozen (25) proposed to model a highly non linear response using a 'substitute structure' where a linear response with appropriate equivalent damping and equivalent stiffness are assumed . In general the values of equivalent damping and stiffness depend on the level of the excitation, i.e. on the dynamic structural displacement or on the ductility demand.

Since the equivalent damping is therefore a measure of how 'fat' (i.e. how dissipative) are the force-displacement hysteresis cycles, its experimental value can be efficiently used to evaluate the appropriateness of a refined model, for which the equivalent damping can also be evaluated (14). The energy dissipation capacity of masonry walls is discussed in the following essentially in terms of equivalent damping.

Flexural response

The hysteretic energy dissipated in a typical rocking mode response is generally small. Typically, the dissipated energy in each cycle evolves with the increase of damage and with the increase of displacement demand. The values of equivalent damping per cycle obtained from a wall showing a rocking response (19) gave a mean value slightly above 5 %. In dynamic conditions, however, when rocking occurs, an additional energy dissipation mechanism is given by impact and radiation damping. If the viscous damping equivalent to the energy dissipated by perfect inelastic impact on the soil is evaluated using the simplified approach of Housner (15, 24), some useful indications can be obtained. If the mass of the wall can be neglected with respect to the carried mass, and all the mass is supposed concentrated at the top of the wall, it is possible to derive the following expression for the equivalent damping:

$$\xi_e = 48\ [1 - (\cos 2\alpha)^2] \qquad (7)$$

where α is the angle between the vertical axis and the line connecting the central point of the top of the wall and the edge point of the base. This equation allows an estimate of the equivalent damping as a function of geometry of the wall only. It results that the equivalent radiation damping can be significant, and increases rapidly as the wall gets squatter. However, rocking is likely to occur for slender walls. If a wall with aspect ratio H/D= 3.0 in single bending is considered ($\tan\alpha = 1/6$) , a value of ξ_e =5.4 % would result. This value can be considered as a good low bound reference, since the slenderness ratio of 3.0 can be considered as an upper limit for walls which can be taken into consideration for seismic resistance. Although this quantification is very rough, it is believed that an additional 5% damping is a minimum which can be safely used in additional to viscous and hysteretic damping for an equivalent linear dynamic analysis. It appears therefore that for limit cases of rocking mode, approximate values of the global equivalent viscous damping associated to hysteresis and radiation damping can be 10%, in addition to the default viscous damping (typically 5%) used for linear response analysis.

Diagonal shear cracking response

The evaluation of the hysteretic energy dissipation capacity of a diagonal shear failure (19) showed that there is a clear trend to an increase in dissipation after the first diagonal cracking is reported. Furthermore, in cyclic tests, the hysteretic energy content of each cycle is approximately constant after cracking, not depending on displacement or ductility. It was also reported that before diagonal cracking the equivalent damping is very close to the value reported for rocking response. However, being the dissipated hysteretic energy per cycle dependent on degradation of internal friction mechanisms, the loading history plays a significant role in the calculated values of equivalent damping. This was reported in (19), on the basis of pseudodynamic test results (2). However, given the results of the cyclic and pseudodynamic test, a lower bound value of ξ_e (10%) could be envisaged as appropriate for post-cracking behaviour of diagonal shear failure modes. When mixed shear-flexural mechanism take place, lower amount of hysteretic energy is dissipated, due to the pinching effect given by the superposition of the rocking-flexural deformation to the shear deformation. Such a case is typical of situations when the calculated flexural strength of the wall is very close to the shear strength. In such cases the equivalent damping typical of flexural response appears to be a conservative reference.

Shear sliding response

Being the pure shear sliding cyclic response very close to an indefinitely elastic perfectly plastic response, equivalent damping evaluation would lead to very high values of ξ_e, with an asymptotic value of 64% for indefinitely large displacements. None of the quasi static experiments carried out by the writers showed a pure sliding failure mechanism, nor the writers are aware of tests on walls where sliding on horizontal bedjoint was the exclusive failure mechanism . Also in the dynamic experiments described in (17) sliding was accompanied or followed either by rocking or diagonal cracking. It would be therefore very unconservative, in an assessment procedure, to assume such a high energy disspiation capacity, also because our ability to predict the failure mechanism is based on approximated strength evaluations. At present no clear experimental information is available on the contribution of sliding to energy dissipation in these mixed-mode or transition situations. In absence of such a reference, special caution should be used for the definition of equivalent damping values in excess of what can be assumed for rocking or diagonal shear cracking.

Experimental estimation of modelling parameters

The expressions used to describe strength, deformation and energy dissipation capacity, give an immediate indication of the parameters which must be evaluated in order to make a quantitative assessment of the seismic lateral load capacity of URM piers. This evaluation must be supported by testing, and it is possible mainly through non destructive in-situ testing, since only in rare cases can undisturbed samples of masonry be brought to the laboratory for destructive testing and it is in practice impossible to manufacture exact replicas of the original material. However, testing need not be excessive if it is well planned with specific consideration of URM mechanisms of resistance. A critical review of experimental techniques for the seismic assessment of masonry structures was presented recently in (5). Some important issues can be however summarized here.

Among the failure mechanisms associated with seismic actions, shear failure modes are the most dangerous and most difficult to assess. Priority should be given to the collection of experimental information on shear strength and deformability. Laboratory tests (in particular shear-compression tests on wall panels) performed and collected in the literature constitute a fundamental source of information. In-situ joint shear testing (shove test) is very useful for safety checks, but proper correction and safety factors should be applied to compensate its tendency to overestimate the strength (cohesion) of the joint, associated to local dilation effects.

Laboratory tests on triplets combined with shear tests on walls are a means to verify correlation of local and global behavior.

Flexural or rocking failure modes of piers are not very sensitive to masonry strength, which in most cases of practice can therefore be assumed. An accurate experimental evaluation of the behavior in compression of masonry is secondary to shear behavior, unless vertical stresses are particularly high or the materials are severely deteriorated.

Vertical stresses play a key role in all of the failure modes cited above. In most cases, load evaluation and simplified analysis are sufficient to ensure an accurate assessment. Single flatjack tests can be used in special cases of complex structures and heterogeneous materials.

For research oriented testing, it should be recognized that recent extensive studies have increased the comprehension of several local failure modes, and of the relationship between the element response and the global response. This general knowledge needs to be extended to additional masonry materials to define the limits separating different modes of failure. Extensive, well-planned parametric experimental studies should therefore be encouraged to allow the refinement of numerical analysis tools.

Simplified methods for seismic assessment of buildings

As discussed in a previous chapter, prevention of out-of-plane collapse by means of ties or ring beams is the most important step in the retrofit of a building. Once these mechanisms are inhibited, the in-plane resistance of the walls is the main mechanism for seismic resistance. The ability to predict the failure mechanisms of a multistory URM wall subjected to in-plane horizontal loads is therefore a fundamental tool for assessment and for the design of strengthening. It is now accepted that simplified assessment methods should account for nonlinear structural behaviour, since existing buildings often do not fulfill the requirements which allow the use of elastic analysis for ultimate state safety checks. However, there is an understandable tendency to refine equivalent static or 'pushover' analysis methods as a more practical alternative to nonlinear dynamic analysis.

The main requirements for simplified equivalent static methods of assessment can be summarized as:

- one or more sets of appropriate equivalent static forces should be defined; these sets must be justified on a rational basis from the consideration of the anticipated dynamic response;
- the structural model should be a rational idealization of the real structure, with clear and justified assumptions regarding e.g. the role of out-of-plane response of walls, the stiffness of the floor slabs, the strength and stiffness of piers and spandrels, and of other structural elements such as ring beams or reinforced lintels;
- the maximum lateral load capacity should be predicted with sufficient accuracy, together with the associated failure mechanisms (i.e. identification of the critical elements and of their failure mode);
- the displacements generated by the equivalent static forces (in particular interstorey drifts) should be a reliable prediction of the maximum displacements generated by the design earthquake;
- criteria to model the effects of the possible strengthening techniques should be available;
- the analysis and the interpretation of the results should require a low computational and pre- and post-processing effort.

Significant progress has been made in the recent past in Italy to provide simplified static nonlinear analysis tools to evaluate the capacity and failure mechanisms of multistorey walls subjected to a prescribed distribution of horizontal forces at the floors (11,20), and rather satisfactory results have been obtained. Still, it appears that there is a fair amount of research needed before all the abovementioned requirements are fulfilled. In particular, uncertainty is left on how to extend the available analytical tools to the analysis of full three-dimensional systems, where the interaction between orthogonal walls may be significant, and on the proper definition of suitable sets of equivalent static forces, which appear to be the topics with a stronger need for further research. Some recent progress in the analysis of multistorey walls is here reported.

Static testing of a full scale two-storey prototype in Pavia (6,18) allowed the detailed observation of the main phenomena in the in-plane collapse of a URM multistorey wall with openings and weak mortar joints. These phenomena can be synthesized as follows.

a) Initially, as the horizontal loads increase, cracking is limited to the spandrels between the openings, in part because these regions are not subjected to any vertical stress due to dead loads, and the joint shear strength is therefore less than elsewhere in the structure (Figure 6).
b) As cracks develop in the spandrels, the coupling between the masonry piers decreases; but the horizontal loads can still increase; eventually, cracks in the spandrels cease to propagate further, and the failure mechanism becomes one dominated by failure (shear cracking or rocking) in the piers (Figure 7). The global overturning moment and the residual coupling given by the spandrels generate an increase in the compression in the piers on the side corresponding to the direction of the seismic forces, and a decrease in the compression on the opposite side. This was shown by the asymmetry in the crack patterns of the single piers.

Numerical simulations of the experiments made by means of nonlinear finite element analyses have reproduced rather closely these phenomena. In (11) a collection of numerical predictions of the prototype test is presented. In some of the methods adopted, a significant reduction in computational effort was obtained by means of techniques which recognize as the main source of nonlinearity the quasi no-tension behaviour of masonry, such as:
- use of macro-elements together with the concept of a multi-fan compression field (4)
- use of an adaptive finite element mesh to reproduce geometrically the evolution of the effective section of piers and spandrels subjected to tensile cracking (9).

A simplified method was developed in Pavia (20), with the aim of implementing the concepts on simple wall behaviour described in the previous section into a nonlinear static structural analysis of a multistorey wall with openings. The method is based partly on original ideas, partly on ideas which can be found in other simplified methods (23, 27). The method was conceived for walls with a sufficiently regular geometry and with vertical alignment of the openings, so that a 'frame' idealization is possible. In such an idealization (Figure 8) it is possible to define 'pier' elements and 'spandrel' elements which can crack, and 'joint' elements which are assumed infinitely resistant. It is supposed that the wall is subjected to an assigned distribution of horizontal forces at the floor levels, which increase proportionally to a scalar multiplier λ.

The deformational behavior of the piers is idealized with bilinear elastic-perfectly plastic shear-displacement relationship, as in Figure 4. The ultimate shear V_u is given by the lowest strength among all possible failure mechanism (a reference is given by the strength equations

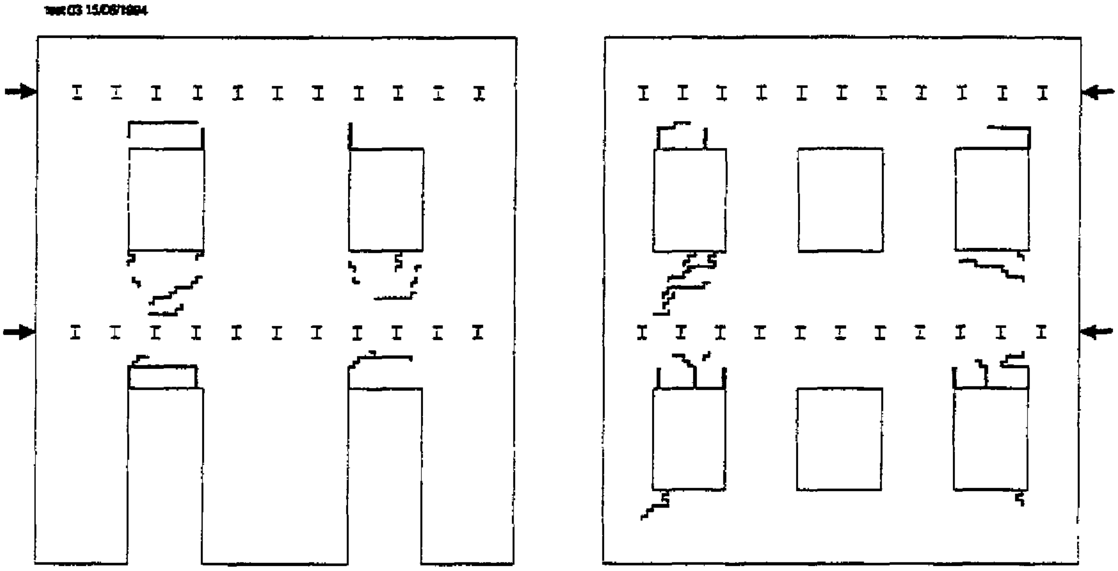


Figure 6. Static test on an URM prototype (18). Crack pattern at moderate damage level (max. drift 0.075%).

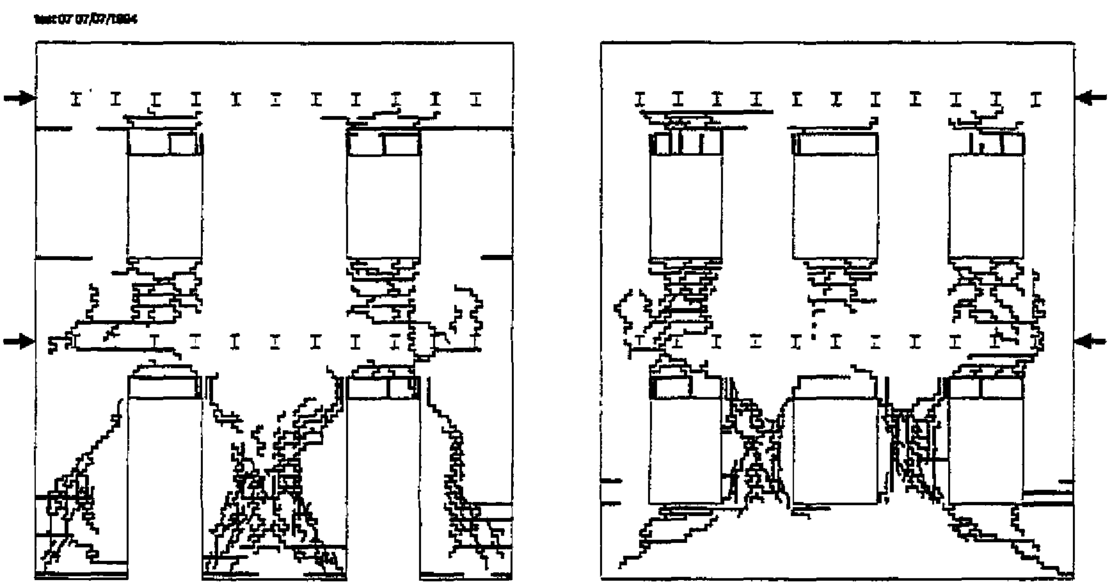


Figure 7. Static test on an URM prototype (18). Crack pattern at ultimate (max. applied drift 0.43 %).

discussed in the previous sections of this paper). The ultimate displacement d_u is given either on the base of prescribed ductility limits, or, as discussed in this paper, on the basis of a maximum drift. It is also supposed that the spandrels can crack in shear, the maximum shear being defined by strength formulas similar as those used for piers.

The analysis is incremental, in the sense that the horizontal forces are increased in finite steps. At the end of each increment the strength criteria are checked, and the internal forces are redistributed in case one or more elements (spandrels or piers) violate the strength criteria (i.e. crack). The redistribution is made in such a way as to keep global and local equilibrium (within an acceptable tolerance). Therefore, as the horizontal loads increase, the variation of axial load in the piers, due to global overturning and to spandrel coupling, can be reproduced. Also, the evolution of the static boundary conditions of the piers is reproduced.

The method was conceived to give a fair prediction of the wall behaviour at ultimate. Being final collapse usually associated to the failure of all piers of one storey, the definition of the initial elastic stiffness of the piers is not as crucial as it is the adoption of adequate strength criteria for the structural components (piers and spandrels).

The method can be defined as a simplified limit analysis where the solution must satisfy equilibrium for each structural element (translational and rotational). The concept of an equivalent frame idealization is suitable for implementation in a nonlinear finite element code, however, the algorithm used in (20) can be in principle applied 'by hand', since compatibility requirements are enforced only in the horizontal displacement at the storey level. This simplification is payed in terms of a lower accuracy in the determination of the correct deformed shape of the building, especially when the coupling given by the spandrels is weak.

This method was checked against nonlinear finite element analyses where a specific constitutive model for brick masonry (12, 13) was used.

The results of the comparisons made so far on two- and three-story walls are rather satisfactory, especially as regards the ultimate load prediction, the failure mechanisms, and the distribution of internal forces at ultimate. In particular, a significant improvement was found with respect to a similar method, rather popular among practicing engineers in Italy, where cracking of the spandrels is not considered. Good performance of the method was reported also in presence of very weak spandrels (Figures 9 and 10). A larger number of structural configurations is being considered to have a better appreciation of the reliability of the method. Also, the problems related to the extension to the analysis of three-dimensional buildings are being tackled.

At present, the research coordinated by GNDT includes among its main subtasks the validation of simplified methods of assessment and their application to selected real buildings.

The choice of retrofitting techniques

The output of an assessment procedure is a judgment on whether the building needs retrofitting or not, according to the design seismic input. Within the formulation of most modern design codes, the inelastic behaviour of the structures can influence both the structural model and the design action (through the 'behaviour factor' or 'force reduction factor'). A structural system with high 'ductility' and ability to withstand repeated cycles of inelastic deformation without a significant degradation of strength and stiffness qualifies for the use of higher seismic force reduction factors. It is therefore rational to suggest that a similar philosophy should be accepted for the assessment of retrofitted structures. With this in mind, the choice of a retrofitting technique should carefully consider its effects on both strength and ductility of a structure, although emphasys is sometimes put more on strength enhancement. To this end, modern codes like Eurocode 8, part 1-4, (10) recognize the possibility of changing (i.e. increasing) the force reduction factor as a consequence of an increase in ductility of the retrofitted structure. Experimental resarch performed on structural elements and structural systems has shown that such approach could be in principle applied also to the retrofit of URM. This can be understood if the following considerations are made:

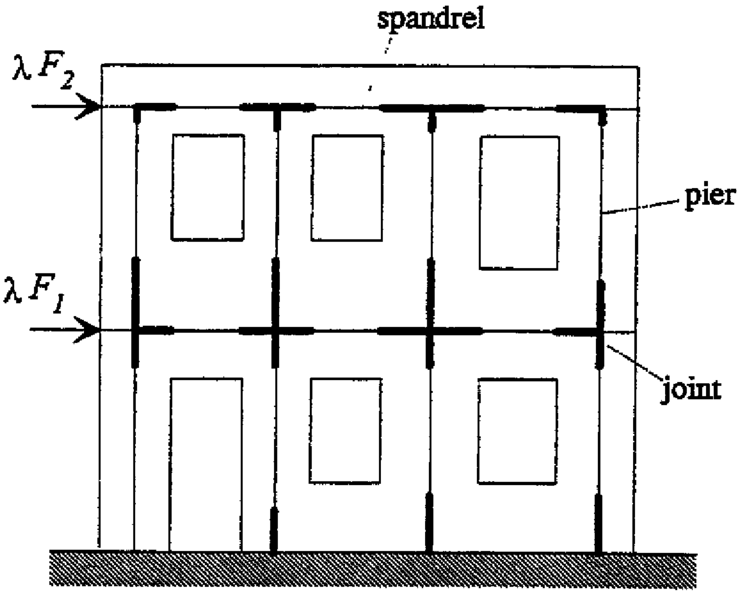
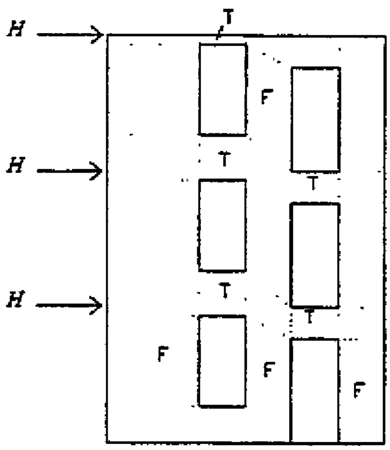


Figure 8. Equivalent frame idealization of a multistorey wall with opening



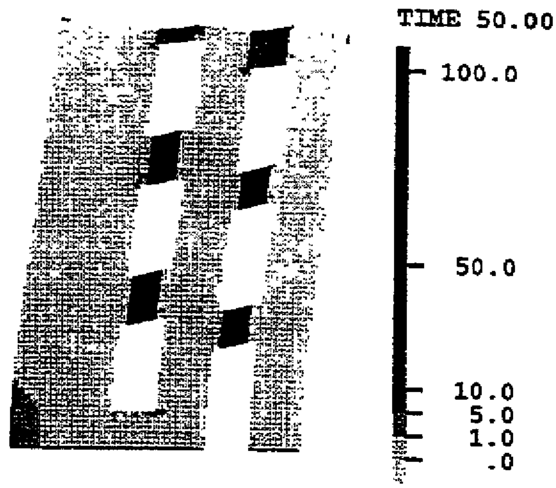


Figure 9. Simplified analysis (20) of a three storey wall with weak spandrels (total height 9 m, length 6 m). Mechanisms of failure: F = flexure, T = shear. Predicted ultimate load $3H$ = 132.3 kN.

Figure 10. Nonlinear finite element analysis of the wall in fig. 9. Distribution of the mortar joint damage parameter (model of Gambarotta and Lagomarsino, refs. 12,13). Predicted ultimate load $3H$ = 120.9 kN.

- masonry walls can show different different deformational behaviour and different energy dissipation according to the failure mechanism (rocking or sliding or diagonal shear cracking);
- a retrofitting tecnhique can be calibrated to inhibit mechanisms which are considered less favourable or more brittle, in favour of other more stable mechanisms.

Considering the present available experimental information (including both quasi static and dynamic tests), there is some strong indication in favour of rocking and sliding mechanisms as stable mechanisms of earthquake resistance (8, 17) as opposed to diagonal shear failure mechanisms. In particular, although diagonal shear cracking can provide significant energy dissipation and a rather stable behaviour in squat walls when mortar bed- and headjoint failure is dominant (22), catastrofic failures are reported when brick cracking is involved, or when diagonal cracks are closer to the vertical direction than to the horizontal (moderately slender piers with high axial load).

Strengthening techniques which have the clear effect of increasing the strength of the walls, such as vertical post tensioning with unbonded bars or strands (internal or external), have the merit of reducing damageability for moderate earthquake, but must be looked at with special care, since their effect may not be necessarily positive in terms of dynamic response under severe earthquakes, if the increase in strength and stiffness due to increase in vertical stresses leads to brittle diagonal cracking failure. For this reason, the solution of vertical post-tensioning with a moderate increase in the compression of masonry (0.1÷0.2 MPa) is going to be tested experimentally on a full scale prototype in Pavia.

It can be concluded that the seismic rehabilitation of a masonry building should be based on the appropriate combination of strength and deformation capacity of the resisting elements, that should aim to the best combination of strength and deformation capacity of the system for the governing limit state, which in turn depends on the local seismicity, expressed in terms of a complete hazard curve. It is believed that time is mature to develop a rational rehabilitation approach for masonry buildings based on these concepts.

Conclusions

Far from being a comprehensive summary of the state of the art on seismic assessment of URM buildings, this paper has focused on some issues that were recently investigated in the course of researches coordinated by GNDT. The methodology which was followed and is presently being followed in these researches is believed to be a rational approach to the problem, promoting interaction and synergy between experimental and numerical research.

The approach to the problem of assessment and rehabilitation presented in this paper includes a revision of the failure modes and deformation and energy dissipation capacity of resisting wall, identification of the relative importance of material parameters and relevance of the most common experimental techniques, development of simplified methods for assessment, indication of possible lines of thinking for a rational approach to rehabilitation. It is felt that a significant step ahead has been made towards a rational assessment and rehabilitation procedure for masonry buildings.

The results described herein, however, cover only part of the tools needed for a rational seismic assessment procedure. Further research is strongly needed before comprehensive and reliable methodologies can be available to designers. In particular, topics of interest are envisaged as:

- extension of the experimental and numerical techniques already calibrated on clay brick masonry to other forms of historical masonry;
- calibration of simplified nonlinear assessment procedures, with special regard for equivalent static methods;

- applicability of modem concepts, such as displacement-based assessment to URM assessment;
- rational criteria for the assessment of irregular buildings.

Acknowledgments

Funding for the research was provided by the Italian National Research Council (CNR - Gruppo Nazionale per la Difesa dai Terremoti).

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