SEISMIC DESIGN OF MASONRY BUILDINGS: CURRENT PROCEDURES AND NEW PERSPECTIVES

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ABSTRACT :

The application of parametric linear and nonlinear analyses on regular structural configurations of masonry buildings has showed how the procedures of seismic safety checks based on linear analyses can lead to inconsistent results if the q-factor values recommended in some seismic codes (i.e. EC8) are used. The need for the consideration of an appropriate overstrength ratio (OSR) is demonstrated for masonry buildings. However, the choice of a single conservative value of OSR for a given building typology does not completely overcome the intrinsic limitations of a linear analysis, given the high variability of OSR from structure to structure. On the basis of comparison between the force distribution in linear and nonlinear analysis two simplified design linear procedures are proposed, solving the high variability of OSR. The first method is based on a traditional linear elastic analysis followed by a suitable redistribution of the internal forces (shear and bending moments). The second method is characterized by a preliminary estimation of the distribution of the internal forces based on the strength of the masonry piers. This estimation is evaluated trough the results of an elastic analysis. The safety checks are then carried out comparing the total base shear of the structure with the base shear strength. Both methods have shown to give good results in comparison with full nonlinear static procedures, when applied to regular building configurations. The second method can be very useful also as a simplified method to check the results of the nonlinear static analyses in terms of base shear-top displacement capacity curve.

KEYWORDS: Simplified seismic design, masonry buildings, linear analysis, q factor, overstrength ratio.

1. INTRODUCTION

In the technical and in the scientific community, there is a rather common opinion that the use of structural masonry for new buildings in seismic areas is "unsafe", since most collapses in recent earthquakes occurred on those structures. Nevertheless, the large majority of those collapses occurred to very badly detailed and badly designed (or not designed) masonry buildings (usually very low quality old dwellings).
However, the construction of new masonry buildings in European countries is far from being marginal, even in countries with considerable seismic hazard (Magenes, 2006). It is therefore clear that research and developments on new masonry buildings in seismic areas are very important and necessary.
The development of the new Italian seismic code, OPCM 3274 (2003), thought as a transition towards the final adoption of Eurocodes, has been an occasion to reconsider the current criteria for seismic design of masonry buildings and their consequences on practice, with particular reference to the problem of defining rationally based values of the behaviour factor q to be used in linear analyses. In particular there is a need to provide simple design procedures based on linear analyses which produce results consistent with the past experience in earthquakes, with the results of experimental tests and of the more refined nonlinear analyses.
The masonry buildings taken into account in this paper are typical structural typologies currently being constructed in Italy constituted by low-rise (up to 3 storeys) unreinforced masonry buildings with continuous r.c. ring beams at the level of in-plane rigid floors constructed in moderate seismic areas. Nevertheless, many of the principles developed could be suitably adapted to other typologies, by following a similar rational framework.
The attention will be focused on methods of global analysis, since in the design of new structures the structural conception and details should guarantee that the in-plane strength of walls could be exploited without out-of-plane collapse.

2. ISSUES ON SEISMIC DESIGN OF MASONRY BUILDINGS WITH LINEAR ELASTIC ANALYSIS

2.1. The current approach for seismic design of masonry buildings

For seismic design of buildings, modern codes (ASCE-FEMA 356, EC8, OPCM 3274, NTC) consider four main methods of structural analysis: linear static (or simplified modal), linear dynamic (typically multimodal with response spectrum), nonlinear static ("pushover"), nonlinear dynamic. In the design of modern structures the structural details (e.g. slenderness limits to the walls, connections) should prevent out-of-plane collapse and the in-plane response of walls should be checked through methods of global analysis. Methods of global analysis that are used in common practice are essentially elastic linear (static or dynamic) or nonlinear static methods based on storey mechanism (Tomazevic, 1999) or on equivalent frame or macro-element idealizations (Magenes et al. 2006, Lagomarsino et al., 2006).

In the case of linear elastic approaches, the safety check procedure is usually based on at least two-level performance requirements (no collapse and damage control); at ultimate (ULS) the safety check consists of a strength verification, whereas for damage control (DLS) the check is made on deformation (drift) demands. The design seismic action is obtained from an elastic acceleration response spectrum scaled by a seismic force reduction factor or behaviour factor (called, hereafter, q-factor) that accounts, in an approximate way, for inelastic response at ultimate. The action is applied to a linear elastic model of the structure and the resulting internal force and displacements are calculated. In general, for masonry structures the ULS verification is governing with respect to DLS. The ULS verification is carried out by checking that in each structural element the design resistance is not exceeded according to the strength criteria defined in codes. In other words, the ULS safety requirement is not met if the shear strength or the flexural strength of even just one element is exceeded.

The nonlinear static analysis (sometimes called "pushover" analysis) consists of the application to the structural model of the building of vertical gravitational loads and of a horizontal force distribution that, keeping constant the relative ratio between the acting forces, is scaled in order to monotonically increase the horizontal displacement of a control point of the structure (e.g. the centre of the mass of the roof), up to the achievement of the ultimate conditions. The result of the nonlinear analysis is a curve (usually called "capacity curve") where the displacement of the "control point" is placed on the abscissa and the base shear is placed on the ordinates. Possible models have been mentioned above, in which masonry buildings are modelled by a three-dimensional equivalent frame with walls, ring beams and masonry spandrels modelled as beam-column elements placed in the centroid of the structural elements. The walls and the horizontal elements are supposed to have an elastic-plastic behaviour with limited deformation expressed in terms of chord rotation or drift. The elements have a linear elastic behaviour until one of the possible failure criteria (flexure or shear) is met. This idealization approximates the experimental resistance envelope under cyclic actions.

2.2. Inconsistencies in linear analyses of masonry buildings

In a study carried out in Pavia (Morandi, 2006) it was possible to observe that from the results of the linear methods of analysis applied to common real structural configurations of masonry buildings following the standard force-based procedures, some important inconsistencies arose.

For example it was evident that, using a q-factor equal to 1.5-2.0 as suggested by some seismic codes (e.g. EC8) it was practically impossible to satisfy strength safety checks for any configuration of unreinforced 2 or 3 storey masonry buildings for peak ground acceleration a_gS greater than 0.1g. In many cases the strength safety checks would not be satisfied even for a_gS greater than 0.05g.

The results of the safety checks after the elastic analyses were in contradiction with the experimental evidence and with nonlinear analyses and were thus found to be overly conservative whereas the results of nonlinear static analyses provided results more in line with the experience. Also, the results of the safety checks through elastic analyses were generally in contradiction with the provisions for the so called "simple building" (i.e. deemed-to-comply regular buildings that satisfy simple geometrical and construction details and for which the explicit safety verifications is not required). On the contrary, when nonlinear procedures were applied more consistent results with the provisions for simple buildings were found.

It was therefore inferred that the main cause of such inconsistencies is not the definition of the level of the seismic action defined as the expected ground shaking, or the necessity of finding particular reserves of deformation or dissipation capacity in the nonlinear range, but the design seismic action for elastic analysis and, therefore, the behavior factor q.

2.3. The role of the overstrength ratio in the definition of the q-factor

It was thus considered necessary to reconsider the criteria for the definition of the behaviour factor q.
The first important consideration was that, according to the linear elastic analyses, the "ultimate" limit state is attained when even one wall of the building has reached its flexural or shear strength. In Figure 1 this state is defined as F_{el} and does not correspond to the ultimate capacity of the buildings. In fact, the unreinforced masonry elements provide a limited deformation capacity in the nonlinear range that allows the building to sustain an increasing seismic load, beyond this "elastic" limit F_{el}, by increasing the forces on the other structural elements. The ultimate strength capacity (defined as F_y in Figure 1 in the case of bi-linear idealization) is reached for higher values of base shear in comparison with F_{el}.
Therefore, for masonry buildings, it appears evident that for the definition of the behaviour factor q it is necessary to introduce an overstrength ratio (OSR) as done for other structural typologies (r.c., steel structural systems). The behaviour factor q can be therefore defined as follows:

$$q = \frac{F_{el,max}}{F_{el}} = \frac{F_{el,max}}{F_y} \cdot \frac{F_y}{F_{el}} = q_0 \cdot \frac{F_y}{F_{el}} = q_0 \cdot OSR \tag{2.1}$$

where q_0 is the basic value that takes into account the dissipative capacity of the structure multiplied by the overstrength ratio $OSR=F_y/F_{el}$.
The ultimate capacity of the buildings is reached when the structural system has attained its displacement capacity. The ultimate base shear corresponding to this point can be much higher than the base shear corresponding to the attainment of the strength capacity in the first wall in the building. In fact, when the "first failure" occurs, the forces can be redistributed through the other structural elements without the structure losing any significant global resistance. Instead, the ultimate capacity of a masonry building is usually reached when any wall first attains its ultimate displacement capacity.
The evaluation of the overstrength ratio can be made through nonlinear numerical simulations and through experiments on models of buildings. It is expected that the OSR should depend, with a varying degree of sensitivity, on many factors, several of these related to modelling hypotheses. Therefore, a consistent evaluation of the OSR cannot rely on interpretation of experimental results alone. A numerical evaluation of the OSR was carried out on different structural configurations on unreinforced masonry buildings as indicated in Morandi, 2006. Such values were used as a reference for the introduction of new values of q-factors published in the recent Italian seismic codes (OPCM 3274 and NTC 2008).
The OSR was found to be very variable (from 1.2 to 3.8) as shown in Figure 2 for two and three-storey unreinforced masonry buildings with r.c. ring beams.

2.4. Drawbacks of linear methods based on q-factor

The recognition of the necessity of the OSR in masonry design, which has been introduced in the revised version of OPCM 3274 (OPCM 3431, 2005) was undoubtedly an important step to rationally explain and find a solution for the inconsistencies found in the application of the code.
Nevertheless, the choice of a specific value of OSR, even for the same homogeneous typology of masonry buildings, does not overcome completely the intrinsic problems of the linear methods of analysis. Considering for example the homogeneous class of two and three storey buildings whose OSR are given in Figure 2, the choice of a single conservative value, be it the minimum or a "sufficiently conservative" percentile (e.g. 1.8 as proposed in OPCM 3431), has the consequence that in the wide majority of the cases, in which the OSR is much higher (e.g. 2.5 or 3), the design seismic action will be much higher than it should. For such configurations, the use of a default conservative OSR could be so penalizing that the strength safety checks can never be satisfied, even if the quality of materials, the structural configuration and details, the total amount of shear walls clearly show that the design should be safe.
A possible solution to these issues stays in the application of a redistribution of the internal forces after the linear analysis that is deemed to be very effective especially in those cases which only few walls are not verified. Nevertheless, it is important to provide which limits can be used in the redistribution of the forces and comprehend which relationship occurs between the overstrength ratio and the redistribution of the forces.
In order to better estimate those issues a detailed comparison between the distribution of the internal forces at the elastic limit and at the ultimate limit state was carried out.

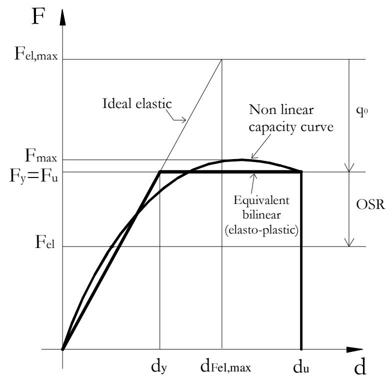


Figure 1 Definition of the q-factor and of the overstrength ratio (OSR)

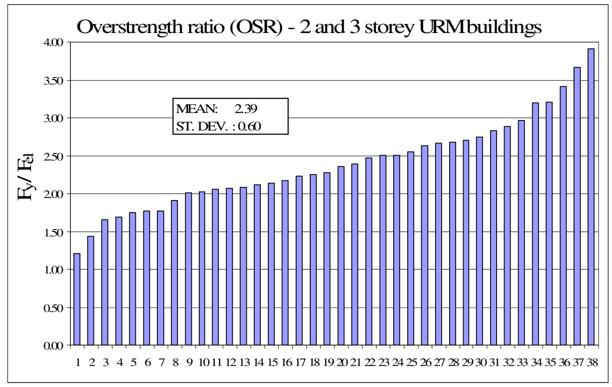


Figure 2 Calculated OSR values for 2 and 3 storey URM buildings with flexurally coupled model.

3. COMPARISON OF THE INTERNAL FORCES AT THE ELASTIC LIMIT AND AT ULTIMATE LIMIT STATE

The attention was focused on the results of plane structural walls and on simple buildings modelled either with the cantilever (pier) and with the equivalent frame method (Morandi, 2006). The results are presented in terms of internal forces of every wall of the structure. In particular, the comparison was made on the shear force distribution, on the axial load distribution and on the moment distribution.
The comparison of the internal forces was made on the linear elastic model at the elastic limit state F_{el} (Figure 4) and at the level of the ultimate base shear F_y (Figure 4) and on the nonlinear static model at the ultimate limit state. It is useful to remind that the elastic limit state occurs when the first wall of a building in the direction of the seismic action fails in its plane for shear or for flexure. The ultimate limit state occurs, instead, when the capacity curve of the structure reaches a force degradation of the 20% of the maximum. This limit state occurs very often in proximity of the attainment of the ultimate displacement capacity in shear or flexure for the first wall of a building. Finally, the ultimate base shear is the value of shear on the capacity curve corresponding to the ultimate displacement. The definition of the limit states introduced above is explained in Figure 4, where the seismic response curve of an actual structure (also called capacity curve) is idealized as a bi-linear elastic-perfectly plastic envelope.
The results of the comparison showed (as reported in Figure 5 for the case of the 1, 2 and 3 storey structural configurations of Figure 3 in the case of equivalent frame modelling) that the shear distribution in the walls using linear elastic models is very different from the shear distribution throughout the walls at the ultimate limit state found carrying out a nonlinear static analysis. In particular, the base shear forces in the walls are not distributed in proportion to the stiffness but in proportion to the strength of the walls.
The distribution of the axial loads on the walls of the buildings is strongly influenced by the boundary conditions which are determined by the coupling offered by r.c. ring beams or floor slabs or spandrels. In the case of equivalent frame models the axial force distribution on the walls changes during the analysis with the increasing of the total lateral force applied on the structure due to the coupling effects of the horizontal elements. However, it has been found that the distribution of the axial forces through the walls at ultimate is very similar to the distribution of the axial forces in a linear elastic analysis as long as the value of the applied lateral force F_h lies between the elastic limit F_{el} and the ultimate limit F_y.
The moment distribution on the walls at the elastic limit and at ultimate limit is totally different. However, from the comparison of the parameter θ that represents the height of the point of contra-flexure from the base of the wall divided for the height of the wall ($\theta = M/(V \cdot h)$), where M is the moment, V the shear and h the height of the wall) at the elastic and the ultimate limit, it is possible to state that the differences are low. Therefore, the point of contra-flexure does not change significantly from the elastic to the ultimate limit.
Starting from these remarks, two proposals for simplified procedures for seismic design of masonry buildings have been developed and are described in the following paragraphs.

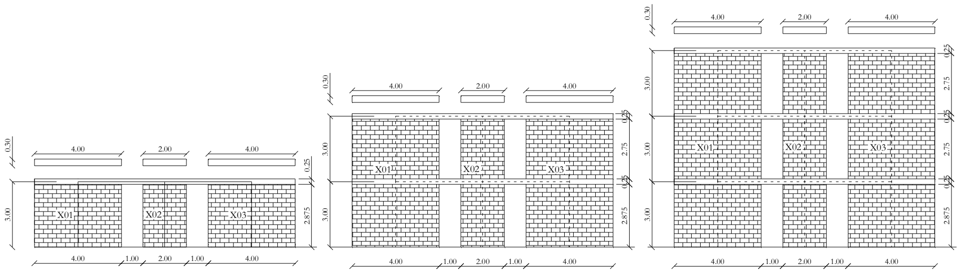


Figure 3 Some of the structural configurations analyzed:
1, 2 and 3 storey plane masonry walls

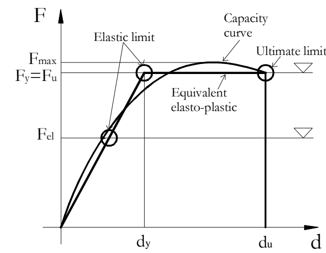
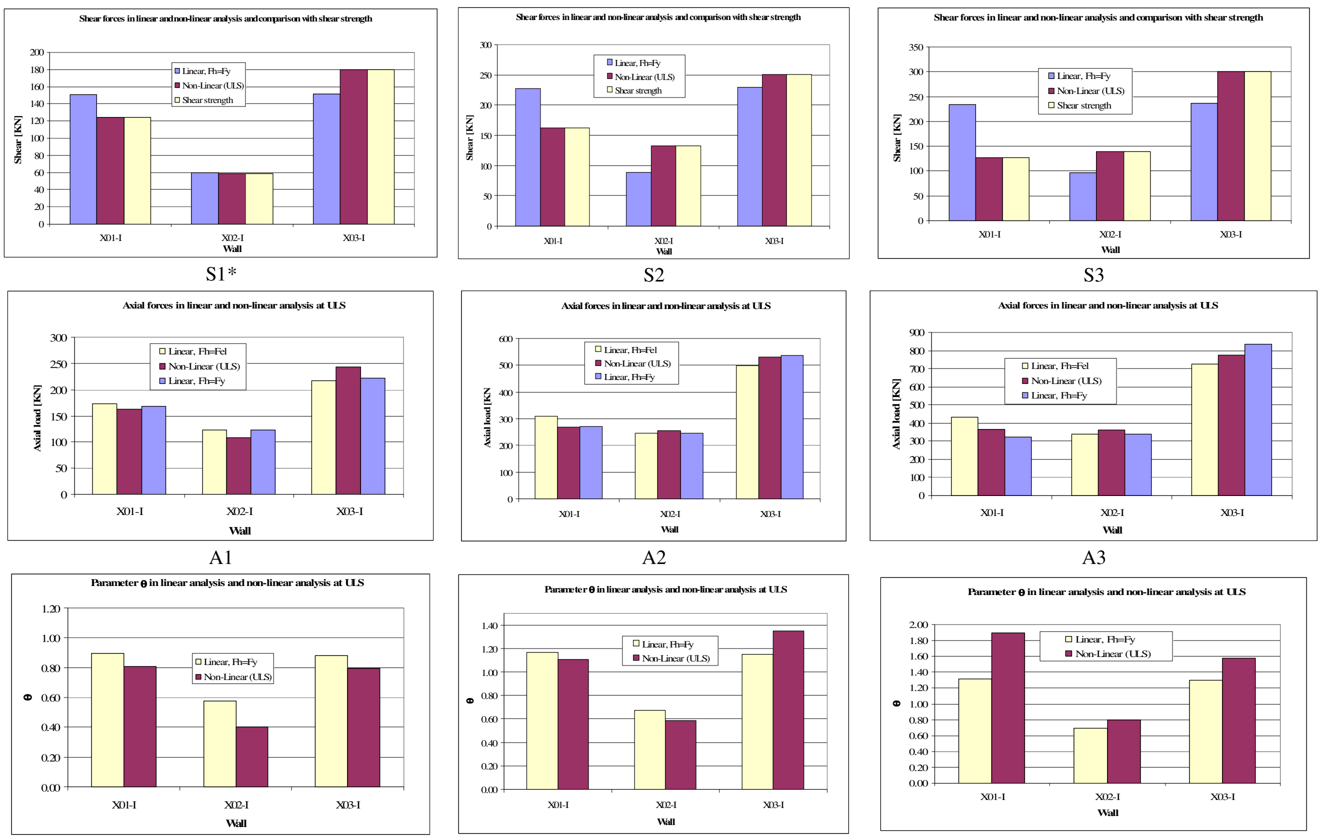


Figure 4 Limit states on the capacity curve



S1* S2 S3

A1 A2 A3

T1 T2 T3

Figure 5 Histograms of the distribution of the ground storey of the shear forces in the walls (S1, S2, S3)
Histograms of the distribution of the ground storey of the axial forces in the walls (A1, A2, A3)
Histograms of the values of the parameter θ in the walls of the ground storey (T1, T2, T3)
* The number after S, A, T means that the results are related to the structural configuration with 1, 2 or 3 storeys

4. SIMPLIFIED PROCEDURES

4.1. Procedure A: linear elastic seismic design with redistribution

The procedure proposed is based on a traditional equivalent linear elastic static analysis. The analysis consists of the application of the lateral forces at the level of each storey assuming a linear first mode displacement shape. The lateral seismic force is computed from the acceleration design response spectrum reducing the elastic response spectrum by the behaviour factor q. The behaviour factor q is assumed equal to the factor q_0, that takes into account only the dissipation capacity of the structure and should not be multiplied by the overstrength ratio OSR that, as proved, can vary within a wide range and that can be higher than the conservative estimate

suggested for the given typology.
Once the lateral seismic force is computed, a structural analysis can be carried out using a cantilever model or an equivalent frame model. The internal forces are thus computed on all the walls. At this point the safety verifications can be carried out. The verifications are based on the comparison between the internal forces and the strengths of every wall.
At this stage it is very likely that one or more walls do not satisfy some of the safety checks even in the case of moderate seismic actions. In this case it is possible to redistribute the internal forces throughout the walls.
The redistribution of the internal forces should be carried out maintaining both the translational and the rotational equilibrium. This means that the total force at each storey and the position of the point of application of the force should be the same before and after the redistribution.
No limitation on the amount of increasing or decreasing of shear (and consequently on bending moment) on the walls is needed provided that the redistribution is compatible with the strength capacity of the walls. This is explained by the fact that at the ultimate limit state all walls of the building in the main direction of the applied seismic force attain their resistance. Therefore, it is possible to add or to subtract amount of shear on walls up to their value of strength, provided that both the translational and the rotational equilibrium are satisfied.

4.2. Procedure B: capacity based seismic design

From the results of the analyses carried out on the building configurations proposed, it is evident that at the ultimate limit state the distribution of the shear in the walls is very different from the distribution of the shear at the elastic limit state. It is clear that at the ultimate limit state the shears are distributed according to the strength and not in proportion to the elastic stiffness of the walls. This is always true for every wall of the critical storey. The distribution of the axial force at ultimate depends, instead, by the coupling between the walls due to the r.c. ring beams. If the degree of coupling is very low and the beams can be considered very flexible in shear and in flexure, the structural models can be realistically represented by the “cantilever” model. Carrying out structural analyses with this model, the axial forces on the walls remain constant and, therefore, are the same at the elastic limit and at the ultimate limit state. If, instead, the r.c. beam is stiffer, the grade of coupling should be considered in the structural analysis using for instance an equivalent frame model. In this case the axial loads do not remain constant throughout the analysis but change. It has been shown that the distribution of the axial forces through the walls at ultimate is very similar to the distribution of the axial forces carrying out a linear elastic analysis as long as the value of the applied lateral force is included between the elastic limit and the ultimate limit (see Figure 5 A1 to A3). Such value of lateral force can assumed as:

$$F_h = \min(0.3 \cdot W_{TOT}; W_{TOT} \cdot \frac{S_e(T)}{q_0}) \tag{4.1}$$

$$F_i = F_h \frac{s_i W_i}{\sum_{j=1}^{ns} s_j W_j} \tag{4.2}$$

where W_{TOT} is the weight of the building, $0.3 \cdot W_{TOT}$ is a simplified estimate of the strength of the building, $S_e(T)$ is the ordinates of the elastic response spectrum, q_0 is the behaviour factor without considering the overstrength, F_h is the total base shear, F_i is the force to apply to the i-th storey of the building (ns is the total number of the storeys), s_i is the displacement of the i-th floor in the first mode of vibration.
From the considerations made above, by applying a linear elastic analysis with the lateral force included between the elastic and the ultimate limit, it is possible to estimate the axial force distribution and the point of contra-flexure of the moment diagrams for the walls of the unreinforced masonry building, assuming it close to the value that would occur at the ultimate limit state.
Finally, knowing the axial force and the point of contra-flexure of the moment diagrams for each wall, it is possible to compute its shear and the flexural strength. Dividing the flexural strength for the distance from the end of the structural element to the point of contra-flexure it is possible to calculate the shear strength corresponding to the flexural failure. The minimum of the strengths calculated in this way represents the effective shear strength of the wall. Applying the same procedure for the walls of the building is possible to compute the resisting shear at every storey.
It is then necessary evaluate the critical storey. The critical storey is the first storey that reaches a failure mechanism in relation with the distribution of the lateral forces applied to the structure. Therefore, it is the

storey that provides the minimum base shear resistance V_{R,TOT_Base} of the structure with the assumed distribution of the lateral forces:

$$V_{R,TOT\_Base} = \min\left\{ V_{R,TOT\_1}; V_{R,TOT\_2} \cdot \left(\frac{\sum_{i=1}^{ns} s_i \cdot W_i}{\sum_{j=2}^{ns} s_j \cdot W_j} \right); \ldots; V_{R,TOT\_ns} \cdot \left(\frac{\sum_{i=1}^{ns} s_i \cdot W_i}{\sum_{j=ns}^{ns} s_j \cdot W_j} \right) \right\} \tag{4.3}$$

Where $V_{R,TOT,k}$ is the shear resistance of the k-th storey.
The resisting shear thus calculated at every storey results in a rather good estimate of the storey shear resistance calculated with a nonlinear analysis above all in the case of buildings modelled as “cantilever” (Morandi, 2006).
The safety check is based on the comparison between the base shear resistance and the seismic base shear computed as the product of the total seismic weight (W_{TOT}) for the spectral ordinates of the acceleration elastic spectrum corresponding to the period of the first mode of vibration ($S_e(T)$), divided for q_0 that represents the behaviour factor without considering the overstrength ($W_{TOT} \cdot S_e(T)/q_0$). If the shear resistance is lower than the seismic force, the building is not verified. If the shear resistance is higher than the seismic force in both the main directions of the building, it is necessary to check the torsional equilibrium according to the procedure defined in the next paragraph.

4.3. Torsional effects

The new constructed buildings are usually torsionally restrained systems which have at least one couple of walls not on the same plane along both main directions which are able to withstand the lateral seismic actions (see Figure 6a). In this case one of the walls parallel to the seismic action will attain its resistance for shear or for flexure first. The stiffness and the location of the longitudinal walls perpendicular to the considered seismic direction will ensure that the lateral force can continue to rise until every wall fails. The total building force-displacement response is like that presented in Figure 6b.
It should be noted that once every wall have attained their resistance, the effective stiffness in the direction considered is zero which means that, increasing the displacement in the direction parallel to the seismic action, an increase of the total storey shear does not occur. Moreover, when every wall has failed, the center of the resisting force has moved from the center of the elastic stiffness (C.R.) to the center of shear strength (C.V.).
The torsional moment M_T at the ultimate limit state when every wall has attained their resistance can be computed by the following expression assuming the main direction of the seismic action along the Y axis (see Figure 6a):

$$M_{T,y} = V_{Ry} \cdot e_x = \sum V_{Ry,i} \cdot e_x \tag{4.1}$$

Where V_{Ry} is the total shear strength along the main direction and e_x is the eccentricity between the center of the strength (C.V.) and the center of the mass (C.M.).
Since no walls in the considered seismic direction has incremental stiffness at ultimate, this torsional moment is resisted entirely by the walls perpendicular to the seismic direction, which, in general, remain elastic. Therefore, in order to compute the shear forces on the walls perpendicular to the seismic direction keeping the torsional equilibrium, it is possible to apply the torsional moment (for example $M_{T,y}$ in the case of the example of Figure 6a) on the structural model and run the structural analysis without the stiffness of the resisting walls
along the Y direction. This means that once the walls in one direction have reached their resistance, the moment due to torsional effects can be distributed in terms of internal forces along the perpendicular walls (in this case along the X walls) as if no walls along Y axis are present.

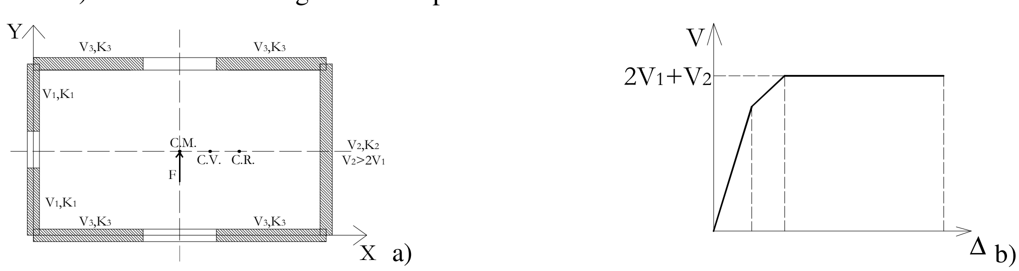


Figure 6 Response of a torsionally restrained building; a) plan view of building; b) force-displacement curve

5. CONCLUSIONS AND FUTURE DEVELOPMENTS

A part of the discussion has been dedicated to the problem of defining a consistent level of seismic design action to be used in the elastic analyses. In particular, the behavior factor q has been redefined as a function of the results of the nonlinear static analyses through the introduction of the overstrength ratio. Nevertheless, the choice of a specific value of OSR, even for the same homogeneous typology of masonry buildings, does not overcome completely the intrinsic problems of the linear methods of analysis.
Starting from the results of a comparison between the values of the internal forces at the elastic limit state and at the ultimate limit state, two proposals for simplified procedures for seismic analysis and safety check of masonry buildings have been developed.
The first one is based on a traditional linear elastic structural analysis with the introduction of a suitable redistribution of the internal forces throughout the walls. No limitation on the amount of increasing or decreasing of shear (and consequently on bending moment) on the walls is needed. Any redistribution of internal forces which satisfies equilibrium and strength capacity criteria in all walls should therefore be considered admissible.
The second procedure is based on a first estimation of the axial forces and of the location of the points of contra-flexure by a linear elastic analysis and it is followed by a safety verification in terms of total base shear of the structure comparing the seismic base shear with the base shear resistance.
Both the procedures, developed and applied for regular structural configurations, have produced results very similar with those of the static nonlinear analyses both in terms of total base shear and in terms of global safety checks (Morandi, 2006). It is believed that these methods provide a very useful procedure not only for the seismic design of masonry buildings but also as a tool to check the results of the capacity curve of nonlinear static analyses.
Further work should be carried out on in order to extend the proposed methods even for non-regular buildings.

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