A strategy for the seismic vulnerability assess of heritage architecture

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SUMMARY:
Most of the Italian architectural heritage is located in zones that are characterized by a moderate or high seismic hazard and often public activities and services are located in these buildings. So that it is important to undertake the evaluation of the seismic capacity of these structures and, where necessary, make strengthening interventions. The huge number of buildings that have to be analyzed implies that it is not possible to assess all of these at once because it requires a strong financial commitment. In this paper is presented a simplified strategy that allows to assess the seismic capacity of buildings on the basis of very few data and through quick analysis. In this way it is possible to obtain a rough classification of buildings on the basis of their seismic capacity, so to allow the Government to destine the limited resources available firstly to the buildings with lower seismic capacity. A worked example shows the described methodology allows a rapid application and requires only few information about the analyzed building. Thus, the necessary data for the assessment may be collected during a rapid survey.

Keywords: Seismic capacity, masonry buildings, assessment procedure .

1. INTRODUCTION

Most ancient buildings located in seismic zones were constructed without a seismic code. So it is necessary to check their effectiveness to resist earthquake motions and to design the interventions needed to reduce the risk for human lives. Because of the large number of buildings that need to be analyzed, it is necessary to use simplified methods to assess the seismic vulnerability.

Some methods, as FEMA154 (2002) and GNDT (2007), provide a set of tables that summarize some structural characteristics of the building and the potential weaknesses, which allow to provide a score as a vulnerability index; it is not quantified the seismic resistance. Calvi (1999) developed a system to evaluate the vulnerability of masonry buildings subjected to in-plane failure mechanisms based on a comparison between the displacement capacity for given limit states and the displacement demand. Restrepo-Velez and Magenes (2004) extended this methodology taking into account the out-of-plane failure mechanism. Lutman (2010), on the basis of numerous seismic analyses performed on existing masonry buildings and collected by the Slovenian National Building and Civil Engineering Institute developed a parametric methodology called PO-ZID. This method requests an important building inventory collection and gives the seismic vulnerability in terms of expected damage.
Bernardini et al. (1990) proposed the Vulnus procedure, which is based on the evaluation of some geometric and mechanical characteristics of each building in combination with some qualitative judgments that control the response of the structure.

There are also some mechanical methods that quantify the seismic resistance without using any subjective judgment, but usually they need a detailed survey of the relevant geometrical parameters. D'Ayala and Speranza (2002) developed a procedure called FaMive. This procedure studies the mechanisms that may occur in a masonry structure and associates a load factor or collapse multiplier to each mechanism by a static equivalent procedure. "VM" is another mechanical method proposed by

Dolce and Moroni (2007), which provides the peak ground resisting acceleration, related to the global collapse of the building (reaching of ultimate capacity of all the walls).

A new potential procedure is presented in this paper that allows to quantify the seismic resistance of the structure from the data collected during a speedy inspection on site of the building and it considers all the relevant characteristics of masonry buildings. The procedure is named FIRSTEP, that is the acronym of FIRst STep Evaluation Program, and it takes into account both global and local collapse of masonry buildings. The procedure was developed for the ASSESS project (Assessment of Seismic Scenarios about Strategical and School Buildings) which is a study, financed by the Italian Region Friuli Venezia Giulia, aimed to construct a prioritization ranking for seismic strengthening of public school buildings. The project, started in 2008, includes the study of approximately one thousand buildings, half of them with masonry structure.

2. THE SEISMIC VULNERABILITY ASSESSMENT

The purpose of this strategy is to reduce the number of buildings to check with a complete survey using desk study and data collection. The first phase of the strategy concerns the examination of the available technical documentation that, through simple formulas, allows to determine the seismic safety index of all the building designed as seismically resistant and also to reduce the number of buildings to be subjected to inspection on site. In the second phase both a brief inspection of the building and a simplified analysis using FIRSTEP procedure are carried out. In the third phase some more deepen studies are conducted to improve the results obtained in the second phase. In Figure 2.1 is clearly evidenced the flowchart of the strategy (Gattesco et al., 2009).

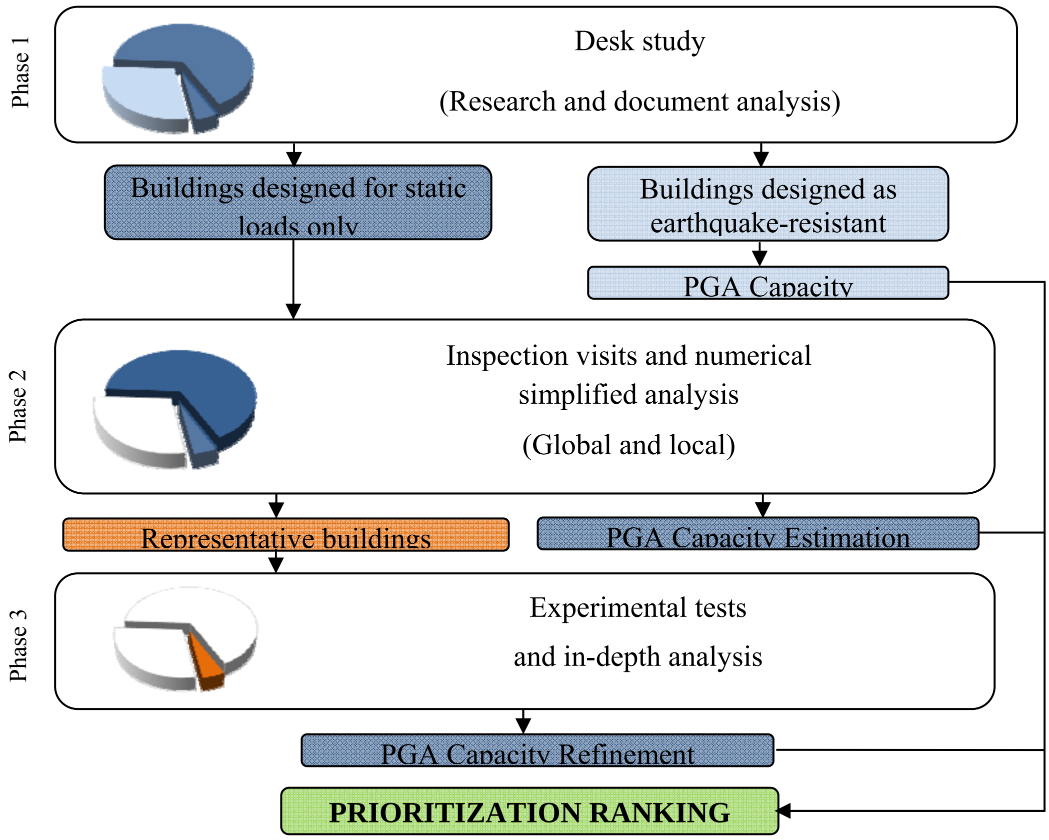


Figure 2.1. Assess Procedure

It is important to notice that every phase produces an assessment of seismic capacity of buildings with different accuracy level, but with reference to the same quantitative parameter: peak ground acceleration.

By calculating the ratio between capacity and demand, the seismic safety index (I_s), used to classify every building in the priority list, can be obtained:

$$I_s = \frac{a_u}{a_{code}}, \tag{2.1}$$

where a_u is the soil acceleration resisted by the building, a_{code} is the peak ground acceleration of the site required by the code.

3. PHASE 1: RESEARCH AND DOCUMENT ANALYSIS

If the date of construction of the building (or possible expansions and renovations), its constructive technology and its location are known, it is possible to associate the code used in the design and the intensity of the seismic action considered. By overlaying this information to the maps that show the changes of classification of seismic areas, it is possible to define a subdivision among buildings with a seismic safety index not lower than one (buildings that can be considered safe), buildings with a seismic safety index lower than one (buildings designed considering an acceleration lower than that required by current code) and buildings designed without considering seismic actions.

In the first phase these last buildings are considered without any seismic capacity because they are located in areas that were not classified as seismic at time of construction. These buildings will be the first that have to be surveyed and analyzed in the second level. It is noted that this assessment of seismic resistance in many cases differ significantly from the real strength, but it is a first quantitative estimation that can be accepted for the purposes of its use. For the analyzed building stock, in Italy, and with regard to masonry structures, the comparison between the old and the current acceleration required is only possible starting from 1977, the year of enactment of the Technical Document DT2 (1980). This document was introduced to regulate the renovation of buildings damaged by earthquake in 1976 that affected the Region Friuli Venezia Giulia.

4. PHASE 2: INSPECTION VISITS AND NUMERICAL SIMPLIFIED ANALYSIS

In the second phase the focus is on masonry building constructed without seismic criteria. The methodology evaluates the seismic capacity through the data acquired during an inspection on site of the building. The inspection consists in a visual screening of the building that takes approximately two hours per building (average volume of about 5000 m^3). To gather all the information, it is necessary to perform an exterior survey and also to go inside the building. As the rapid visual screening RVS procedure proposed in FEMA 154 (2002), the scope of the inspection is to identify the primary structural-load-resisting system, the structural material of the building and to identify the building attributes that may characterize the seismic performance. This phase of the work allows to identify all the building peculiarities needed in FIRSTEP algorithm.

4.1 Global analysis

Masonry buildings are characterized by a wide range of construction techniques, the structures can have different materials and also different slabs, rigid or deformable, all this characteristics influence the structural behaviour. Moreover, certain portions of the structure may have different characteristics because the construction of buildings often took many years for the completion or it was built in multiple times. The procedure proposed attempts to take into account all of the possible variants that can be found in masonry buildings.

The FIRSTEP program for the global seismic evaluation was prepared with an Excel Macro, compiled on Visual Basic, and uses the geometrical data extracted from a Cad file. Using AutoCAD software it is quick and simple the process to define the geometry of the building. After importing geometry data in Excel, through an automatic procedure, some other data, such as mechanical properties of masonry materials, slabs material and behaviour, inter-storey height, behaviour factor and the soil type are imputed in the Excel program.

The program evaluates the elastic distribution of a unitary force among vertical elements (f_{ei}). If the structure has flexible floors, the seismic action is divided among walls in proportion to their influence area. In case of rigid diaphragms, the seismic action is divided among the walls in function of their position and stiffness. If the centroid is eccentric with respect to the centre of stiffness, a twisting effect is taken into account. The ultimate shear resistance of masonry walls (F_{ui}) is estimated by the relationship of Turnsek-Cacovic (1971).

The peak ground resisting acceleration (PGA) is calculated with the relation:

$$a_{ux} = \frac{q}{S \cdot F_0} \cdot \left(\frac{F_{Rx} \cdot g}{W} \right), \tag{4.1}$$

where q is the behaviour factor, S is the soil parameter and F_0 is the building amplifier factor (NTC 2008, EN 1998, 2004). For deformable diaphragms, $(F_{Rx} \cdot g)/W$ coincides with the lower ratio between F_{Rxi}, that is the seismic resistance capacity of the i-th vertical element and W_i/g, that is the mass that pertains to the i-th vertical element. For rigid diaphragms, W/g is the total mass of the structure and F_{Rx} is equal to the minimum ratio between the ultimate resisting force (F_{ui}) and the force due to the unit horizontal determined for each resisting wall (f_{ei}).

To improve the assessment of the resistance capacity of the structure, only in the cases with rigid diaphragms, the Firstep procedure permits to do a linear elastic analysis with redistribution of shear force among the piers (NTC 2008, 2008), (EN 1998, 2004). The evaluated resisting acceleration for each principal direction is used for the estimation of the seismic safety index.

4.2 Local analysis

Masonry buildings may also have local weaknesses with the possible partial collapse of them. For this reason, the building resistance assessed in terms of peak ground acceleration has to consider also the possible occurrence of local mechanisms.

A survey sheet was constructed to simplify the identification of possible mechanisms in masonry structures. Furthermore a quick system to assess, during the visit, the local capacity was created. This simplified procedure consists in a series of nomograms, one for each mechanism. In Fig. 4.1 is illustrated the nomogram relative to the overturning mechanism of the wall. Through a nomogram, fixing the width, the height and the thickness of the wall it is possible to quantify the value of the acceleration at the base of the building that causes the local collapse of the element due to a possible mechanism. This simplified system is necessary because of the large number of mechanisms that may activate and it allows to choose the most critical that may be evaluated with the analytical procedure on desk, after the visit.

The strategy used for the calculation of local collapse mechanisms is presented in the Italian seismic code (NTC 2008, OPCM 3431, 2005). The method uses the limit analysis (kinematic method) to determine the multiplier of the horizontal loads (α_0), which leads to the activation of the mechanism. The peak ground acceleration a_g, required to activate the mechanism, is for simplicity obtained as follows (OPCM 3431, 2005):

$$a_g = a_0 \cdot \frac{q}{S} \cdot \xi(z,N), \tag{4.2}$$

where a_0 is the spectral acceleration that activates the kinematic mechanism obtained from the relationship

$$a_0 = \frac{\alpha_0 \cdot g}{e^*}, \tag{4.3}$$

g is the gravity acceleration, e^* is the fraction of the participating mass to the mechanism (NTC 2008), S is the soil factor, q is the behavior factor, z is the height of the center of gravity of the weight forces of the mechanism elements, from the base of the building, H is the total height of the structure.

The evaluated resisting acceleration for each mechanism analyzed is used for the assessment of the seismic safety index. The kinematic analysis considers the elements as mono-dimensional. In order to consider that the walls may be restrained in all boundaries, not only at the top and at the bottom, it is necessary to carry out a study so to determine an equivalent height of the mono-dimensional model that represents the actual wall.

Limit analysis, by using the upper bound theorem, was used for calculating the collapse load of panels with real restraints and subjected to a uniformly distributed load (Hendry, 1986). The collapse mechanisms for panels restrained on 3 or 4 sides are shown schematically in figure 4.2.
The collapse load is applied to an ideal equivalent wall that has restraints only to the foot or at the top. This equivalent wall has the same width of the real wall, and the height he is modified to give to the wall the same collapse load of the real wall. The effective height was used to construct the survey nomograms and is also used in the kinematic calculation.

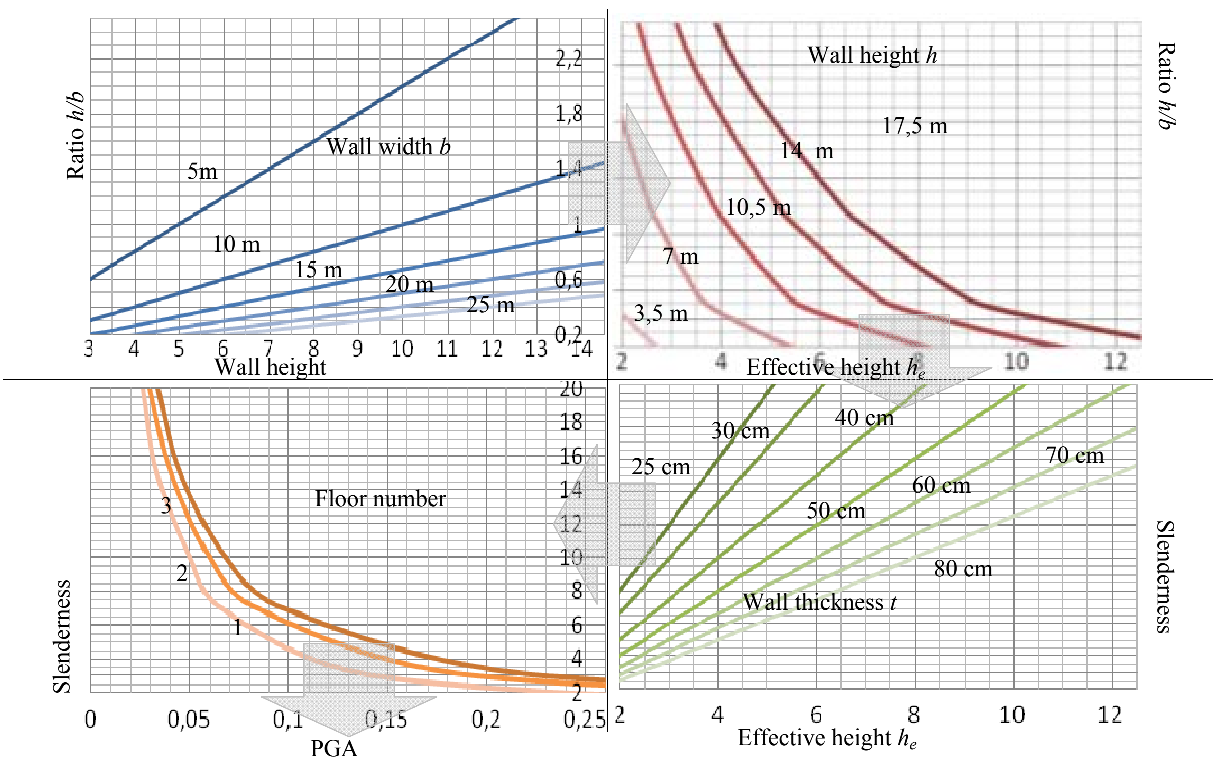


Figura 4.1. Nomogram for the overturning mechanisms of a wall.

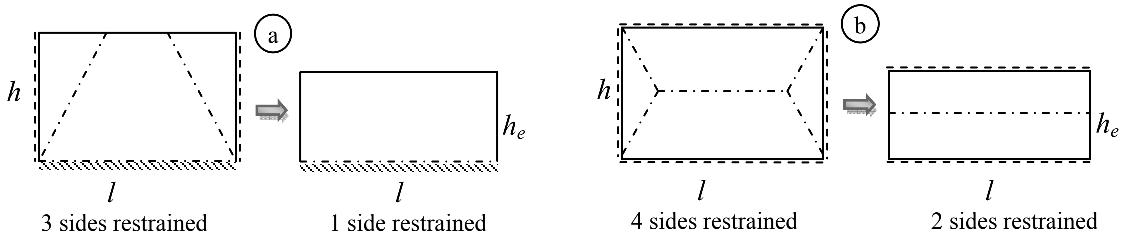


Figure 4.2. Collapse mechanisms for real and equivalent walls: a) real wall with 3 sides restrained equivalent to a wall with fixed end at the bottom, b) real wall with 4 sides restrained equivalent to a wall with hinges at both ends.

To take account of the openings influence, numerous cases of perforated walls were studied so to define all possible kinematically admissible mechanisms. The study evidenced that in most cases it is more conservative to calculate the height of the equivalent wall (he) referring to walls without openings.

4.3 Worked example

A masonry building of the nineteenth century was considered as worked example. This building was constructed in pre-seismic code periods, without application of seismic criteria and details. During the construction only some practical rules were adopted. So an inspection of the building was performed to carry out all necessary data.
The building has a rectangular shape (7.9 x 14.2 m) and it has three storeys. The first two interstoreys are 3 m high and the remaining one is 2.2 m high (Figure 4.3). The walls, 0.55 m thick, are made with almost rounded cobblestones coming from morainic deposits and bound together with quite thick joints of lime mortar. The wooden floors are made with parallel joists of spruce. The mechanical properties of the masonry are: specific weight γ_c = 19 kN/m^3, shear resistance f_{vo} = 0.033 MPa and the modulus of elasticity E = 1500 MPa. In this study a safety factor γ_M = 1 was used.
The length, thickness and position of the various vertical resistant elements are the data needed in the program FIRSTEP. The vertical load acting on each wall is calculated automatically sharing the total weight of the floor (Fig. 4.4).

The global earthquake resistance of the building in two perpendicular directions, in terms of maximum peak ground acceleration, is obtained by using the procedure above described. The values of peak ground acceleration are 0.337g, for longitudinal direction, and 0.209g, for transversal direction.

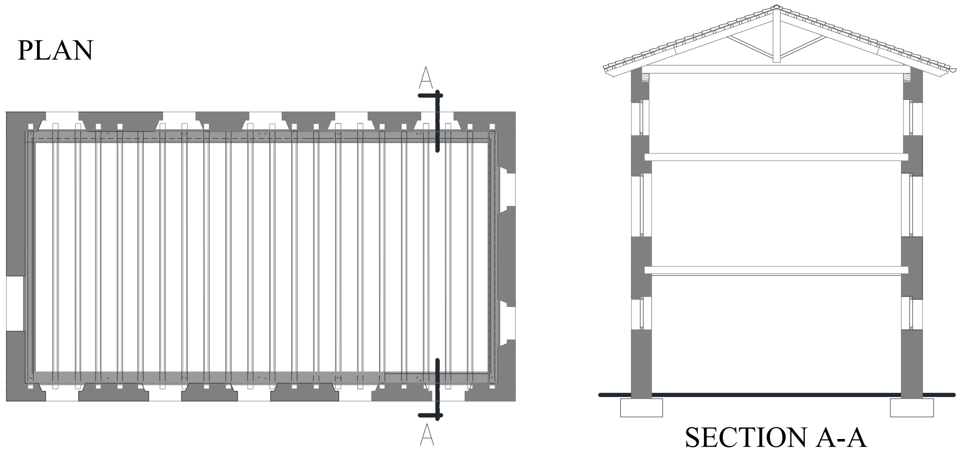


Figure 4.3. Relevant plan and section

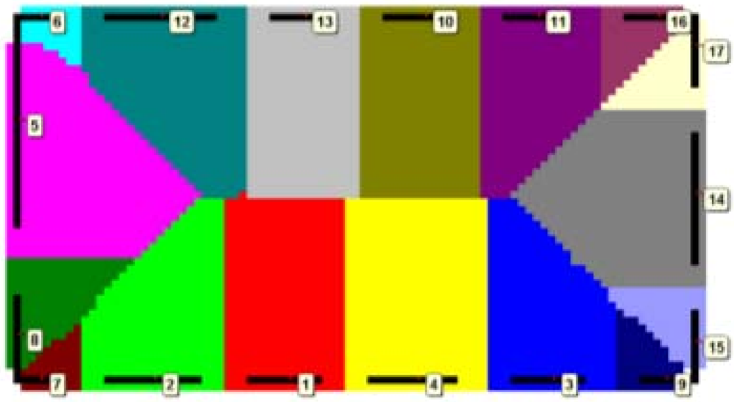


Figure 4.4. Automatically created influence areas

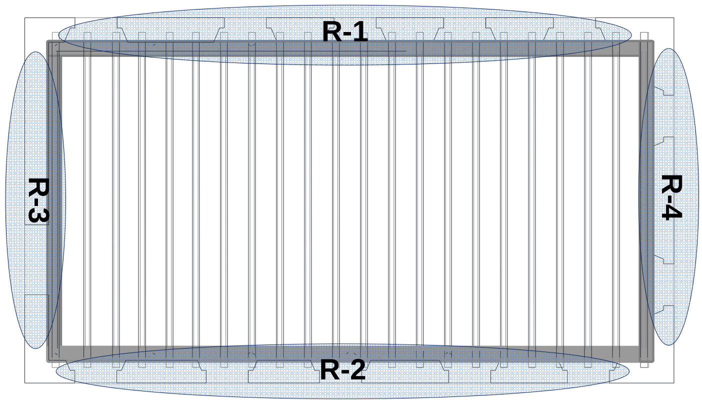


Figure 4.5. Local mechanical overturning wall.

Table 4.1. Local PGA results

Mechanisms	R-1	R-2	R-3	R-4
Nomogram method a_{gu}	0.10g	0.10g	0.120g	0.120g
FIRSTEP method a_{gu}	0.148g	0.148g	0.182g	0.182g

All the potential local mechanisms were considered and, during the supervision in situ, were checked by using the survey sheet described in Section 4.2. The mechanism that evidenced a value of the resisting acceleration lower than that required by codes were then analyzed by using the analytical procedure described in Section 4.2. For the example, the overturning mechanisms evidenced in Figure 4.5 resulted to be the most critical to be subjected to analytical assess. The resisting acceleration of these mechanisms, evaluated both with the graphic and the analytical procedures are reported in Table 4.1. Appreciably greater resistances were obtained with the analytical procedure with respect to those derived with the nomogram. This is mainly due to the approximations that were made for the construction of the nomogram; the results of the nomogram, however, are always conservative.

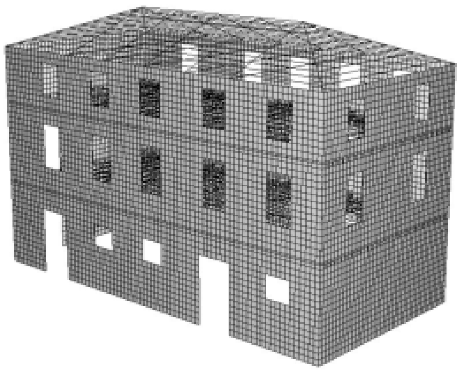
5. PHASE 3: EXPERIMENTAL TESTS AND IN-DEPTH ANALYSIS

In the third phase some more deepen studies were conducted to prove and improve the results obtained in the second phase. A comparison between the results of different types of analysis using the resisting peak ground acceleration as comparing parameter is presented, so to understand the accuracy level of the various models and methods.

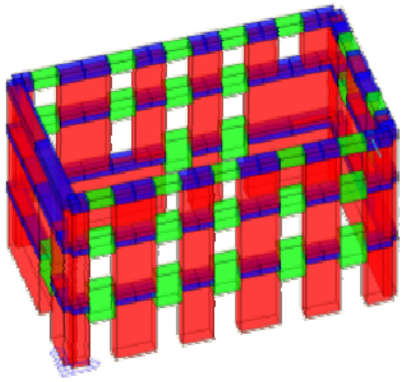
Firstly, the comparison concerns the out-of-plane collapse of external walls of the building in Fig. 4.3

in the hypothesis of in-plane deformable floors. The comparison is carried out between the results obtained with the simplified analysis FIRSTEP and with the 2D nonlinear analysis (Abaqus) considering the horizontal forces acting in Y-direction (transversal). In Abaqus program the resistant masonry walls were schematized using four-noded shell elements (S4R elements provided by Abaqus), having a characteristic dimension equal to 200 mm, while the Concrete Damage Plasticity Model (Hibbit et al., 2004) was employed to model the non-linear material properties of the masonry assumed as isotropic continuous material. A yield function of modified Drucked-Prager and Rankine type determines the state of failure or of damage. The plastic flow is governed by a potential function formulated according to the non-associated flow rule and depending on the effective stresses, the dilation angle ψ and on the masonry tensile strength f_{tcm}. In this example, an elastic modulus E_m =1500 MPa, a compressive strength f_{cm}=2 MPa, a tensile strength f_{tcm} = 0.05 MPa and a dilation angle ψ=35° were assumed.

The procedure FIRSTEP evidenced that the collapse is due to the overturning of the longitudinal walls in correspondence of a ground acceleration equal to 0.148g (Table 5.1). The 2D nonlinear analysis leads to the same type of mechanism, for a value of the ground acceleration equal to 0.17g (Table 5.1). As can be evidenced by these results the simplified procedure FIRSTEP provides a good estimate of the real capacity of the building/wall.



(a)



(b)

Figure 5.1. Numerical model for the masonry building: (a) 2D model (Abaqus), (b) equivalent frame model (MidasGen).

Table 5.1. Results of models with deformable floors

Method	Result [PGA]
Simplified analysis (FIRSTEP)	0.148g
2D nonlinear analysis (Abaqus)	0.170g

The second comparison concerns the behavior of the same building but considering rigid the two first floors. The comparison is carried out among the results obtained with the simplified model FIRSTEP, the nonlinear push-over analysis based on an equivalent frame model (MidasGen) and the 2D nonlinear analysis (Abaqus) considering the horizontal forces in the transversal direction (Fig. 5.1).
The equivalent frame method used in MidasGen program utilize monodimensional elements (beam type) with rigid ends to take into account the real dimension of the elements. Each element has two flexural hinges and one shear hinge. The behaviour of pier hinge, both for shear and bending moment, was modeled by a bilinear elastic-plastic curve defined by resistance, stiffness and ultimate displacement. The shear resistance is obtained by Turnsek and Cacovic (1971) relationship, considering the diagonal cracking mechanisms. The ultimate bending capacity is obtained by equation:

$$M_{Rd} = \frac{\sigma_0 b^2 t}{2}\left(1 - \frac{\sigma_0}{0.85 \cdot f_m} \cdot \gamma_m\right), \tag{5.1}$$

where f_m is the compressive strength of the masonry, b and t are the width and the thickness of the wall ($A=b\ t$), γ_m is the material safety factor (unitary in nonlinear analysis) and σ_0 is the average compressive stress due to axial force N ($\sigma_o=N/A$).

The ultimate displacement of the pier is δ_u = 0.004 h_{eff} for shear and δ_u = 0.006 h_{eff} for bending moment (NTC, 2008), where h_{eff} is the effective height of the pier evaluated according to Dolce (1991). The stiffness is calculated taking in considerations both shear and flexural deformability.
The behavior of spandrel hinges was modelled by a bilinear elastic-brittle curve, both for shear and bending moment. The ultimate displacements may be assumed equal as in spandrels. The shear V_t and flexural M_u resistances are calculated using the following equations:

$$V_t = h \cdot t \cdot \frac{f_{v0k}}{\gamma_m}, \tag{5.2}$$

$$M_u = \frac{H_p \cdot h}{2} \cdot \left[1 - \frac{H_p \cdot \gamma_m}{0.85 f_h \cdot h \cdot t}\right], \tag{5.3}$$

where f_{v0k} is the shear strength of the element in absence of vertical load, h and t are height and thickness of the spandrel, H_p is taken as the minimum value between $0.4 f_h \cdot h \cdot t$, and the tensile strength of any horizontal tie element, f_h is the horizontal compressive strength of the element.

The push-over procedure (EN 1998, 2004) can calculate the capacity curve, that displays the base shear against the displacement of a control node. The peak ground acceleration in the push-over analysis is obtained dividing the maximum base shear by the total mass of the building. In Abaqus program are valid the previous explained consideration but in this case diaphragm behavior is rigid. The comparison is evidenced in Table 5.2. The results obtained with the three programs are not so far one another; as awaited, the FIRSTEP program provides an underestimation of the capacity with respect to other programs. The FIRSTEP program is based on a linear elastic analysis with redistribution of the shear force among piers and provides results very close to those obtained with a push over analysis.

Table 5.2. Results of models with rigid floors

Method	Result [PGA]
Simplified analysis (FIRSTEP)	0.209g
Push-over analysis (MidasGen)	0.221g
2D nonlinear analysis (Abaqus)	0.250g

6. CONCLUSIONS

A strategy for seismic vulnerability assess of existing masonry building was presented. This strategy is finalized to construct a prioritization ranking for the strengthening interventions needed to plan the reduction of seismic risk of existing buildings. Inside this strategy was presented a procedure for quantifying in a quick and simple way the seismic capacity of masonry structures taking into account both the global structural collapse and the eventual activation of local mechanisms.

As shown by the examples, the procedure is effective and provides very good results requiring a rather limited number of data.

The strategy was used in the Assess Project (Region Friuli Venezia Giulia, Italy) aimed to evaluate the seismic capacity of masonry school buildings. Up to now, more than one-hundred masonry school buildings were analyzed with success.

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