A methodology for the seismic risk mitigation based on mechanical models: the case of reinforced concrete schools in Genoa (Italy)

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15 WCEE
LISBOA 2012

SUMMARY:
Recent earthquakes have highlighted the potential seismic vulnerability of existing reinforced concrete buildings; not only in case of residential buildings but also strategic structures, such as schools or hospitals. Thus, a reliable vulnerability assessment at large scale becomes crucial in particular for public institutions to optimize the criteria to identify the priorities of intervention (priority lists) in order to better allocate the limited economic resources. Among the different approaches proposed in literature, the use of mechanical models seems particularly suitable to this aim. These models, being based on a limited number of geometrical and mechanical parameters, allow to describe the inelastic response of buildings by capacity curves. In the paper the attention is focused on the DBV-*concrete* model proposed in Lagomarsino *et al.* (2010) based on the displacement-based approach. It has been applied on the case of 55 schools of Genoa Province (North Italy). Results are compared to those provided by on observational vulnerability model in order to combine these different approaches for defining proper priority list.

Keywords: r.c. buildings, mechanical models, vulnerability assessment, mitigation policy

1. INTRODUCTION

Recent earthquakes (as an example in Italy, the 2009 L'Aquila earthquake) have highlighted the potential seismic vulnerability of existing reinforced concrete buildings, generally due to the lack of ductility rather than inadequate lateral strength. The ductility deficit is a consequence of two major failings in the original design process: poor detailing of reinforcement and the lack of a capacity design philosophy. This condition regards not only residential buildings but also strategic structures, such as schools or hospitals. Thus a reliable vulnerability assessment of existing assets becomes crucial. As known, main aims of a vulnerability analysis are: a) to be aware of the impact of an earthquake to the buildings; b) to plan preventive interventions for the seismic risk mitigation; c) to help the management of the emergency after a big earthquake. In particular, the paper deals to aims a) and b) by presenting an application to the case of reinforced concrete school buildings in Genoa Province (Italy) based on the combined use of different vulnerability models.

As known, vulnerability models are the tool to establish a correlation between hazard and structural damage. As a function of the model adopted, hazard may be represented in term of the macroseismic intensity, peak ground acceleration (PGA) or response spectrum. Structural damages are usually classified in various levels as a function of the heaviness and the spread in the building; thus, building Perfomance Levels (i.e. immediate occupancy, damage control, life safety, collapse prevention) may be associated to selected Damage States, on the basis of the consequences related to the advisability of post-earthquake occupancy, the risk to the life safety or the ability of building to resume its normal function. Several methods for the vulnerability assessment have been developed and proposed in recent years, which are implemented with different kind of data (from poor statistical data about the building type, the number of floor etc., to data specifically surveyed for seismic vulnerability assessment). They are based on various approaches, which may be basically classified according to the following two classes: the observational (or macroseismic) models and the mechanical ones.

Observational models are derived and, consequently, calibrated from damage assessment data, collected after earthquakes in areas that suffered different intensities. Considering set of buildings with a homogeneous behavior, the damage is described by damage probability matrices (DPM). DPM traditionally are associated to a discrete number of building classes. In order to pass from discrete to continuous vulnerability evaluation, different approaches are proposed in literature (as that illustrated in Giovinazzi and Lagomarsino 2004).

On the contrary, mechanical models describe the structure response by mean of a force – displacement curve which aims to describe the overall inelastic response of the structure, providing essential information to idealize its actual behaviour in terms of stiffness, overall strength and ultimate displacement capacity. Each point of this curve is associated to an exact pattern and level of damage. According to perfomance- based assessment, by defining on the capacity curve proper damage states (corresponding to predefined displacement values) it is possible to evaluate the distribution of damage levels. Differently from the case of macroseismic models, which are calibrated on basis of earthquake damage survey, the validation of the mechanical ones represents an issue much more complex since this direct comparison lacks. A possible alternative is to compare the results of the mechanical models to those provided by the macroseismic ones; however, this comparison needs the introduction of suitable correlation laws between the significant parameters which define the seismic input employed in mechanical approaches (usually the Peak Ground Acceleration) and those employed in macroseismic ones (that is the Intensity).

In case of reinforced concrete structures, mainly built following an "engineering approach" based on principles and rules of design, the use of mechanical approach seems particularly effective. In fact, it presents the following main advantages: to employ the results of sophisticated hazard analyses (by using the seismic input in the spectral form); to keep explicitly into account the different parameters which concur to define the structural response. This latter advantage seems even more significant in case of vulnerability assessments oriented to plan preventive intervention for the seismic risk mitigation. Actually, in this case, the realibility of the assessment is mainly related to the capability of the model to evaluate the relative seismic capacity of a building within a group in order to identify the more vulnerable structures and thus to define a list of priority.

According to this latter aim, the paper focuses on the application of the DBV-*concrete* (Displacement Based Vulnerability) method originally adopted in Lagomarsino *et al.* (2010) on the case of 55 school buildings of Genoa Province (North Italy). This method refers to a mechanical model based on displacement approach. Thus, the quantitative evaluations based on this approach are compared to those obtained through the observational model proposed by Lagomarsino and Giovinazzi (2004) for the same building stock (as discussed in detail in Raineri *et al.* 2012). Finally, a procedure aimed to define a priority list based on the combined use of these two approaches is proposed: it aims to combine the abovementioned advantages of two models.

2. THE MECHANICAL MODEL ADOPTED

The mechanical model adopted is based on the DBV-*concrete* method proposed by Lagomarsino *et al.* (2010). This model basically starts from the one originally proposed by Crowley *et al.* (2004 and 2008), with some modifications mainly related to the definition of the yielding period (by the introduction of ψ coefficient) and the SDOF (by the introduction of κ' coefficient). Starting from the original proposal adotped in the DBV-*concrete* method, some slight modifications, defined on basis of the results of an extensive sensitivity analisys described in Cattari *et al.* (2012), have been introduced.

Assuming a bilinear curve without hardening, three quantities basically need to be defined to fully describe the capacity curve, in particualr they are: the vibration period (T_{LS2}) and the displacement capacity at yielding (D_{LS2}) and the ultimate capacity (D_{LS4}). Once defined D_{DS2} and T_{DS2}, the ultimate strength of the capacity curve (A_y) is obtained through their intersection ($A_y= D_{DS2}(2\pi/T_{DS2})^2$). Expressions of T_{LS2} , D_{LS2} and D_{LS4} are differentiated as a function of various structural types and two main global failure modes (if beam-sway or colum- sway).

Concerning the evalution of T_{LS2}, the formula adopted in the model is based on: the building height; two coefficients (C',β') defined as a function of different structural system (if bare, infilled frame or dual system, depending on systems designed or not to design capacity); a coefficient (ψ) defined as a function of the height section of column and beam (h_s and h_{st} respectively), inter-storey height (h_i) and the compressive strength of concrete (f_c). Compared to other expressions proposed in the literature (Crowley and Pinho 2010, Chopra and Goel 1997), the introduction of ψ coefficient seems particularly useful to the vulnerability assessment addressed to define a list of priority since is more capable to take into account – also for the T_{LS2} evaluation - the specific characteristic of a building within a group. The evaluation of displacement capacity at the limit state 2 and 4 (D_{LS2} and D_{LS4}) is basically related to chord rotation θ_{LSi} of the main structural element, column or beam, depending on the global failure mode (at first only the yielding contribution is computed for D_{LS2}, then, progressing the response in non linear range, the plastic one is also added for D_{LS4}). The formulations of chord rotation are based on two main approaches: analytical and empirical.

In particular, the model adopted in this paper, compared to the original DBV-*concrete* method: favours the empirical approach for the computation of θ_{LS4} (altough in §4 also the results obtained by the analitical approach – as discussed in Cattari *et al.* 2012 - are discussed); introduces the parameter L_T, beam length, in the formula of coefficient (ψ) for the computation of T_{LS2}. This latter factor, has been calibrated on basis some parametric modal analyses.

Table 2.1 summarize the expressions useful to define the abovementioned entities for the proposed application. For further details see Cattari *et al.* 2012, where is presented both a wider discussion about the different approaches for the computation of chord rotation and a sensitivity analysis performed on the parameters which the adopted model is based on.

Table 2.1. Summary of expressions useful to define the entities which mechanical model adopted is based on

<table>
<tr><td rowspan="3">Definition of the capacity curve parameter</td><td rowspan="3">Entities directly defined on mechanical basis</td><td>T_{LS2}</td><td>$T_{LS2}=\psi C' H_T^{\beta'}$
Where: $\psi=\left(\frac{\overline{h_s}}{h_{si}}\right)^{0.5}\left(\frac{\overline{h_{sT}}}{h_{sTi}}\right)^{\beta_3}\left(\frac{H_{Ti}}{\overline{\overline{H_T}}}\right)^{0.75}\left(\frac{\overline{f_c}}{f_{ci}}\right)^{0.125}\left(\frac{L_T}{\overline{L_T}}\right)^{0.75}$</td></tr>
<tr><td>D_{LS2}</td><td>$D_{LS2}=\kappa\theta_{LS2}H_T$
$\theta_{LS2}=\Phi_y\frac{L_v}{3}+0.0013\left(1+1.5\frac{h_{s(t)}}{L_v}\right)+0.13\Phi_y\frac{d_b f_y}{\sqrt{f_c}}$
Where: κ = κ' κ₁ ;k_1 is defined as the ratio between the height to the centre mass of the SDOF substitute structure and the total height of the original structure (see Glaister and Pinho 2003) and κ' is equal to κ' = (2N+1)/N. The yielding curvature at the end section for the beam sway mechanism and column sway mechanism stand respectively for:
$\Phi_y=1.7\frac{\varepsilon_y}{h_{st}}\quad \Phi_y=2.4\frac{\varepsilon_y}{h_s}$</td></tr>
<tr><td>D_{LS4}</td><td>Beam sway: $D_{LS4}=\kappa_1\theta_{LS2}H_T+(\theta_{LS4}-\theta_{LS2})\kappa_1 H_T$
Column sway: $D_{LS4}=\kappa_1\theta_{LS2}H_T+(\theta_{LS4}-\theta_{LS2})h_1$
Where: $\theta_u=\frac{1}{\gamma_{el}}0.016(0.3^{\nu})\left[\frac{\max(0.01;w')}{\max(0.01;w)}f_c\right]^{0.225}\left(\frac{L_v}{h}\right)^{0.35}25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}\left(1.25^{100\rho_d}\right)$</td></tr>
</table>

Table 2.2 summarizes the parameters set the mechanical model is based on.

Table 2.2. Building parameters for the mechanical model definition

Geometrical features of the member	N (storey number); H_T (total height); h (inter-storey height); h_1 (inter-story height at ground floor); L_t (beam length)
Geometrical features of the section	h_s (height section of the main structural element ruling the global response, that is the r.c. beam or the r.c. column); d_b (longitudinal bar diameter); A_s (column longitudinal reinforcement), A_{st} (beam tension longitudinal reinforcement), A'_{st} (beam compression longitudinal reinforcement), A_{sw} (transversal reinforcement), p (stirrup spacing), b_c and d_c (width and depth of the confined core of the section)
Mechanical parameters and loads	ε_{cu} (ultimate concrete strain); ε_y (yielding steel strain); ε_{su} (ultimate steel strain); f_y (yielding steel strength); f_c (concrete resistance) ; L_V (shear span); f_{yw} (yielding transverse steel strength; ν (axial load ratio).
Corrective factors	β_3 (equal to 0.25 in case of column sway mechanism and 0.5 in case of the beam sway one); C' and β'(are defined as function of the structural type, for example in case of moment resistant frames designed only for vertical load without significant seismic details stands for 0.089 and 1 as proposed in Crowley and Pinho 2006); γ_{el} (is equal to 1.5 for primary structural elements).
Note: For the evaluation of coefficient ψ, the reference mean values of $\bar{h}_s, \bar{h}_{sT}, \bar{h}_i, \bar{f}_c$ should be specify.	

Once defined the capacity curve through the mechanical model, the vulnerability assessment may be provided by evaluating the target displacement (D_{PP}) according to non linear static procedures proposed in the literature and codes (such as the Capacity Spectrum Method or the N2 Method).

3. DESCRIPTION OF THE EXAMINED SAMPLE

The examined sample (owned and managed by the Provincial Authority of Genoa) is composed of 55 reinforced concrete buildings constisting of typical beam-column RC frames with no shear walls. The vast majority of them were designed and built between 1920 and 1971 (age from which structural design has became compulsory); thus, almost all (except one) were designed only for vertically load without significant seismic details. They generally present the resistant r.c. frames only in one direction; in the orthogonal one, the frame effect is guaranteed only by the floors slabs (usually constitued by light blocks separated by RC ribs, and then a reinforced slab cast on top): thus this latter direction may result quite vulnerable. It has to be pointed out that pre-code, or gravity load designed, represent the majority of the building stock in many areas that have been recently classified as seismic: this stresses their potential high vulnerability. It is worth noting that only the 6% of the schools examined have pilotis on the ground floor; the 60% of the sample has a storey number varying from 1 to 4. Additional information about characteristics of the building stock are discussed in Raineri et al. (2012).

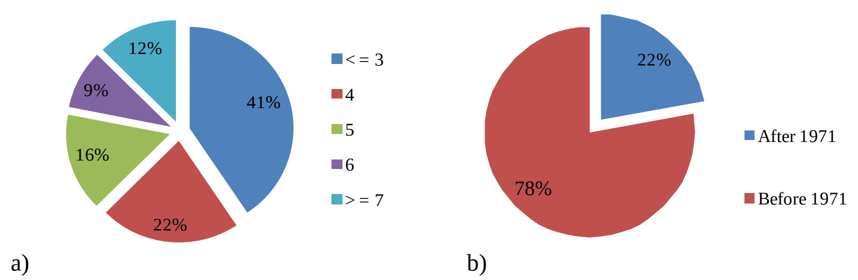


Figure 3.1. a) Number storeys of buildings; b) Building age

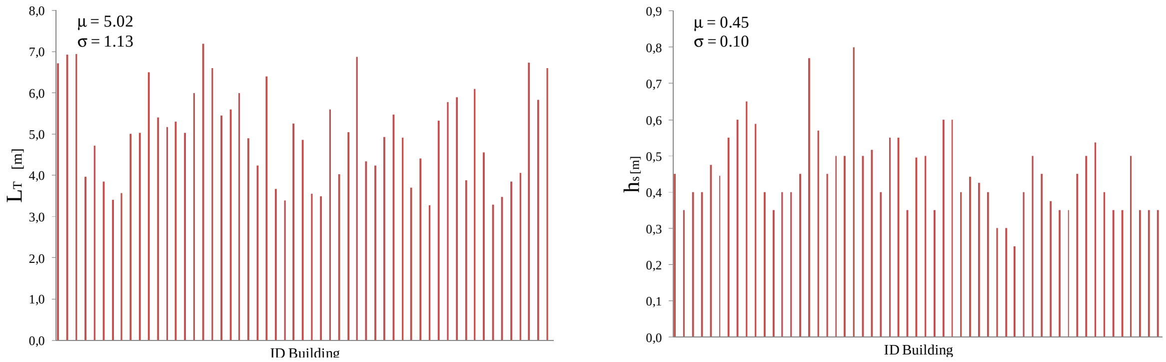
3.1. Definition of the parameters which the model is based on for the examined sample

In order to apply the mechanical model described in the §2, mechanical and geometrical features have to be defined for each building. Tables 2.2. summarize the meaning of parameters adopted. It is worth noting that the definition of these factors has followed different strategies, as a function of the data which were available for each building. If, for the building analyzed, the structural design was available (10% of the sample), these parameters have been directly deduced by it, on the contrary a proper integration of the missing data has been obtained by combining in situ survey, analysis of architerctural plan available and simulated design. Some parameters have been hypothized unvaried for all buildings as a function of: the age (if designed “before1971” or “after 1971”, according to specific recommendation of in force codes); the failure mode (beam-sway or column-sway mechanism); the structural type (moment resistant frames).

Table 3.1. Data adopted in the model

As a function of the Age			As a function of the Failure mode		
Parameter	**“Before 1971”**	**“Post 1971”**	**Parameter**	**Column-sway**	**Beam sway**
f_c [Mpa]	20	26	β_3	0.25	0.5
f_y - f_{yw} [Mpa]	300	430	$\bar{h}_s\,[m]$	0.33	0.33
ε_{cu}	0.0075	0.015	$\bar{h}_{sT}\,[m]$	0.60	0.30
ε_{su}	0.036	0.036			
h_1 [m]	3.20	3.20	**As a function of the Age and Structural Type (Moment resistant frame)**		
L_T [m]	4	4	**Parameter**	**“Before 1971”**	**“Post 1971”**
A_{sw} - ϕ8 [cm^2]	1.01	1.01	C_1	0.089	0.089
p [cm]	20	20	β_1	1	0.9

Figure 3.2 shows the variations of some geometrical features, like as the height section of columns and the beam lenghts, within the sample.



a) b)

Figure 3.2. a) Beam length; b) Height section of column

It is worth pointing out that the protection level of school buildings, set by rules, is higher than that of ordinary buildings. Thus, the expected seismic hazard corresponding to limit state 3 (*LS3*), for example, is evaluated for a return period (T_R) of 949 years; in the studied sites of Genoa Province, the related PGA value on stiff soil ranges from 0.073 g and 0.138 g (as provided by a detailed seismic hazard map proposed in the Italian Code for Structural Design issued in 2008, named in the following as NTC 2008).

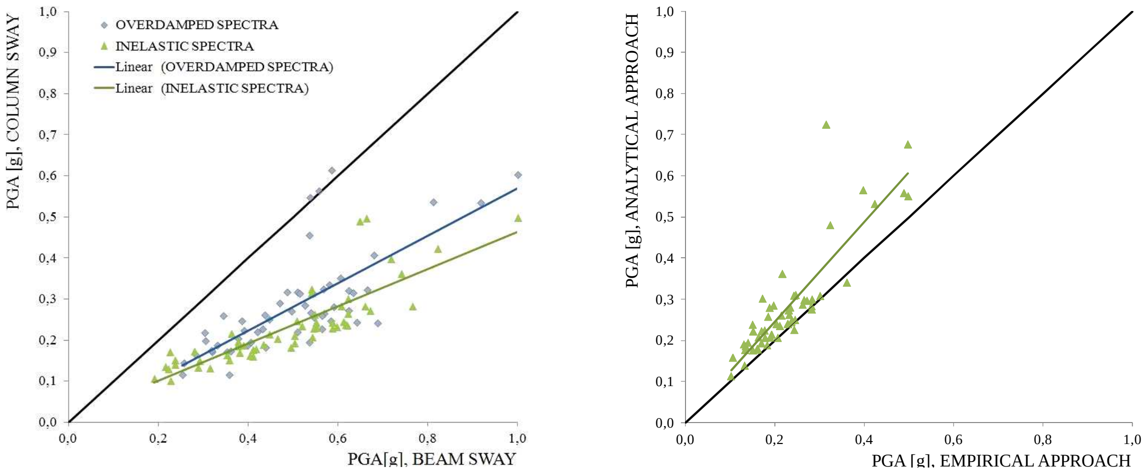
4. APPLICATION OF THE MECHANICAL MODEL TO THE EXAMINED SAMPLE

This chapter focuses on the application of mechanical model to examined sample, as described in §3. In particular the vulnerability assessment is obtained by considering:

- both analytical and empirical approaches for the evaluation of chord rotation at the LS4, θ_{LS4};
- two failure modes hypothesized: beam-sway and colum-sway mechamism;
- both overdamped (according to *Capacity Spectrum Method*, proposed originally by Freeman *et al.* 1975) and inelastic (according to *N2 Method*, proposed originally from Fajfar 2000 and used in both Eurocode 8- Part 1 2005 and NTC 2008) approaches, in order to properly reduce elastic spectra and define the target displacement D_{PP}.

Resultant capacity curves of each building may significantly differ in terms of strength, stiffness and ductility. Since all these three aspects play a fundamental role in seismic assessment, thus in the following, a comparison in terms of maximum acceleration ($PGA_{LS3},_{max}$) compatible to the fulfilment of LS3 was done in order to define their relative vulnerability; LS3 is assumed consistent to the Life Satefy LS. In particular the value of $PGA_{LS3},_{max}$ has been obtained by imposing the target displacement (D_{PP}) of the structure with the displacement capacity associated to LS3 obtained as 60% of the displacement capacity at the ultimate state D_{LS4} (computed from expressions summarized in Table 2.1). The target displacement has been computed according to expressions proposed in Fajfar (2000) and adopted in Eurocode 8 (2005), in case of inelastic spectra, and by those of the CSM in case of the overdamped ones.

Figure 4.1 highlights the comparisons between the value of $PGA_{LS3},_{max}$ obtained for each structure. As evident, the estimation more on the safe side is achieved by adopting: the column-sway failure mode, inelastic spectra and empirical approach. Thus this set is assumed as reference in the following.



a) b)

Figure 4.1. a) Comparison between both failure mode (column-sway and beam-sway) and reduced spectra (inelastic and overdamped); b) Comparison between empirical and analytical approach, hypnotizing the column-sway mechanism and using inelastic spectra

The more conservative prediction in case of column-sway is certainly coherent with the fact that the existing structures do not satisfy the principles of capacity design. In addition, from the simulated design (according to very simplified design rules suggested in oldest codes adopted as reference), strong beams and weak columns have been obtained: consequently the column-sway mechanism is favoured.

In the following, the above discussed evaluations based on the DBV-*concrete* model are compared to those obtained through the observational model proposed by Lagomarsino and Giovinazzi (2004) for the same building stock (as discussed in detail in Raineri *et al.* 2012). Finally, a procedure aimed to define a priority list based on the combined use of these two approaches is proposed.

4.1. Comparison between the observational and mechanical approaches

As abovementioned, several methods for the vulnerability assessment have been developed and proposed in recent years, which are implemented with different kind of data. They are based on various approaches, which may be basically classified according to the following two classes: the macroseismic models and the mechanical ones. The first ones are derived and, consequently, calibrated from damage assessment data, collected after earthquakes in areas that suffered different intensities. The second ones employ the results of sophisticated hazard analyses and keep explicitly into account the different mechanical and geometrical parameters which define the structural response.

Among the different models proposed in the literature refereable to the observational class, that proposed in Giovinazzi and Lagomarsino (2004) is adopted. According to this model, proper vulnerability curves may be introduced to correlate the macroseismic intensity (I) to the mean damage grade μ_D (a continuous parameter, $0<\mu_D<5$), and a histogram of damage grades is evaluated by a proper discrete probabilistic distribution (binomial). The vulnerability curve is defined by two parameters, the vulnerability index (V_I) and a toughness coefficient, which should be evaluated from the information about the building. The V_I index is defined on the basis of structure typology of building (masonry or r.c. building,…) and refined through behaviour modifier scores, related to qualitative parameters for example maintenance state, structural regularity, etc. These modifiers allow to pass from the fragility curve of the building class to the curve of specific building.

It is worth pointing out that, on the one hand, some qualitative modifiers present on the macroseismic model (e.g. state of maintenance or irregularity factors) are not considered in the mechanical one and, on the other hand, mechanical parameters included in mechanical approach are neglected at all in the macroseismic one: thus, it seems particularly interesting the comparison between two models.

Since the hazard is represented in terms of Intensity and Peak Ground Acceleration (PGA, a synthetic parameter aimed to summarize the spectrum) in case of macroseismic and mechanical approach, respectively, a proper correlation law between these entities have to be introduced. Figure 4.2 shows the comparison between the PGA values – as obtained from the mechanical model - and the I values – as obtained fron the macroseismic one - with respect to some I^* - PGA relationships proposed in literature. In particular, for each building, the evaluation corresponding to LS3 is represented; in case of the observational model, it has been obtained by imposing μ_D equal to 3 (as assumed in Raineri *et al.* 2012).

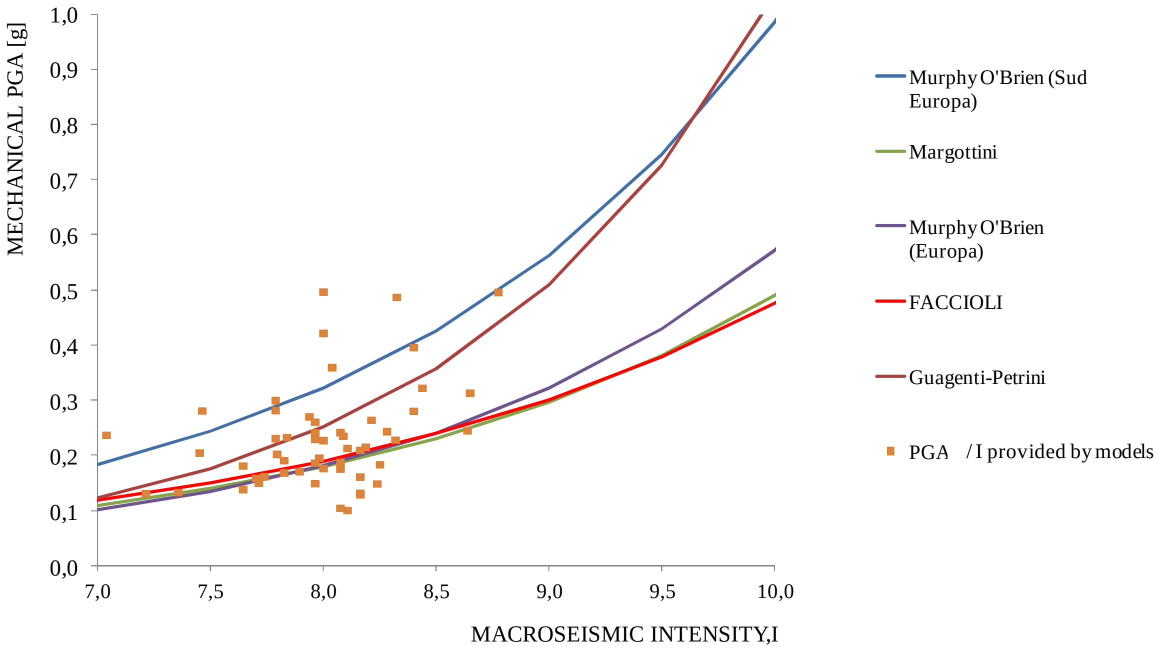
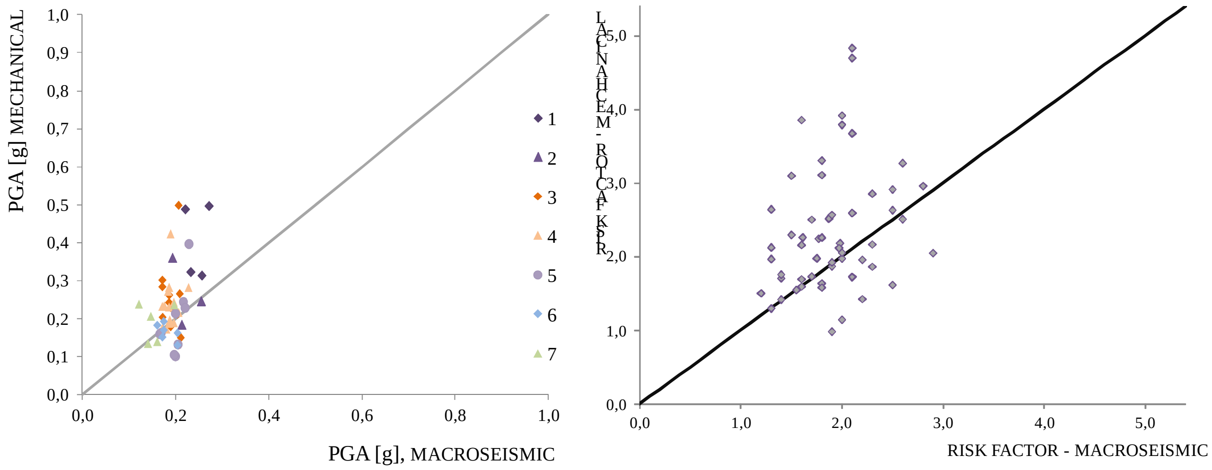


Figure 4.2. Comparison between $PGA_{LS3,max}$ obtained by mechanical model and macroseismic intensity $I_{,SL3}$ obtained by macroseismic model

In the following, the Faccioli and Cauzzi (2006) I-PGA correlation law, which based on data from earthquakes of the Mediterranean area, is adopted as reference. Figure 4.3 compare the resultant values obtained from two approaches. In particular, it is worth noting that: the two models shows a quite good agreement;in both cases, for increasing the storey number the PGA values decrease; the outsider cases are related to one or two storey buildings. In general, macroseismic model seems to be more punitive than mechanical one. Figure 4.3b shows the comparison in terms of Risk Factors computed as the ratio between structural capacity (expressed by means of PGA value) and the earthquake demand.



a) b)

Figure 4.3 a) Comparison between $PGA_{LS3,max}$ obtained by mechanical model and PGA associated to LS3, obtained by macroseismic model, the values are divided according to the storey number. b) Comparison between risk factors achieved by both models

5. CRITERIA TO DEFINE PRIORITY LIST BASED ON THE COMBINED USE OF OBSERVATIONAL AND MECHANICAL APPROACHES

The main goal of the application above discussed is to evaluate the relative seismic capacity of a strategic structure within a group to identify the more vulnerable buildings and thus to define a list of priority (that is in which buildings are listed with increasing values of the Risk Factor). Starting from this list, it is possible to select buildings where more detailed analyses may be focused on.

Since both approaches examined (either the mechanical and the observational one) present interesting and in some way complementary advantages, it seems particularly interesting to combine their use.
Thus, in the following a procedure aimed to outline a priority list based on both these two approaches is proposed. In particular, it is based on the following three criteria:

a) to adopt the priority list as obtained from the model considered more reliable in terms of results achieved: it may be either the mechanical or the observational one;
b) to adopt the priority list as obtained by selecting the first 10 (for example) buildings that are present in both lists as resultant from the mechanical and the observational one, separately;
c) to select the ten buildings associated to the smaller Risk Factor by far (as resultant from either the mechanical or the observational one).

These criteria may be used in separate or combined way. In particular, it is proposed to use them in an integrate way.

Table 5.1 shows the lists as resultant from the abovementioned criteria (from a) to c)).
In order to combine these different criteria, then, a “final” priority list may be obtained in this way: at first, the buildings present in all three lists (the red one in Table 5.1) are selected; then, buildings present at least in two lists (the grey one in Table 5.1) are added; finally, if present, the buildings highligthed only by the model valued as the more reliable one are included in the list. The “final” list illustrated in Table 5.1 has been obtained by assuming the more reliable model – following criterion a) - as the mechanical one. In red are marked the building present in all three lists (derived by applying separately criteria a), b) and c)).

It is worth noting that more vulnerable buildings result: structures with plan and elevation irregularities and built in areas of high-seismicity in Genoa Province. In agreement with the experiences in past and recent earthquakes, most of the damages are related to architectonic and structural irregular configuration in plant and elevation.

Table 5.1. Application of proposed criteria to define a priority list

CRITERION a)		CRITERION b)	CRITERION c)	"FINAL" PRIORITY LIST
MECHANICAL MODEL	MACROSEISMIC MODEL			
70	114	114	70	**88**
59	74b	88	59	**85**
88	74c	85	114	**114**
85	68	(77-78)a	88	**70**
57	88	262	74b	**59**
114	85	74b	74c	**57**
262	(77-78)a	29	68	**74b**
157	74a	54	85	**(77-78)a**
29	72d	50	(77-78)a	**57**
3	262	27	57	**29**

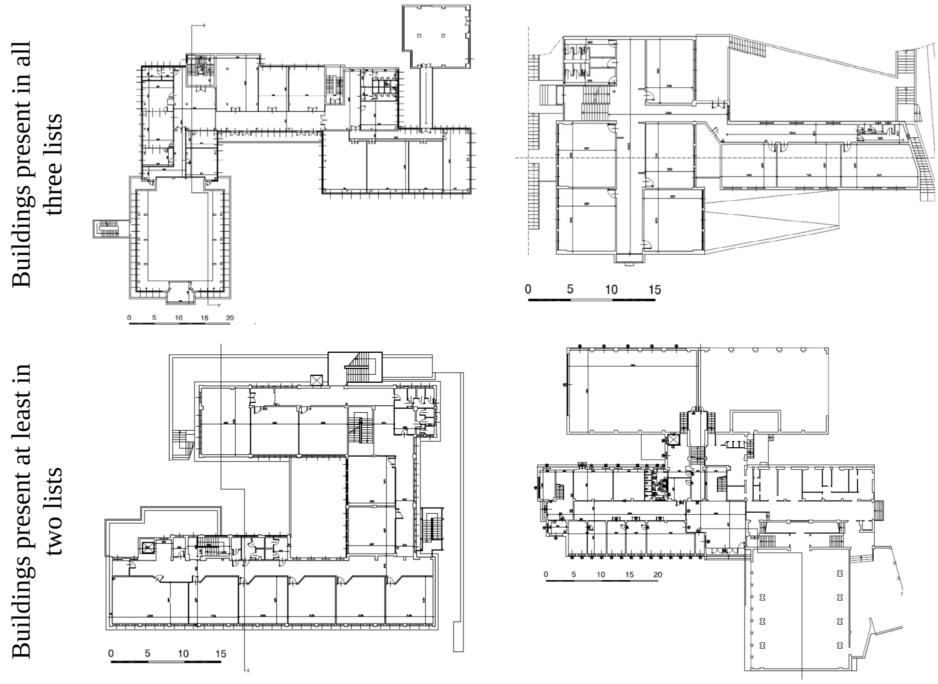


Figure 5.1. Some plan configurations of more vulnerable structures in the examined sample

6. FINAL REMARKS

In this paper, a vulnerability assessment at large scale on 55 school buildings of Genoa Province has been performed. It is mainly addressed to support mitigations strategies for public institutions: in fact, given the limited budget usually available, it is necessary to optimize the criteria to identify the priorities of intervention (priority lists) in order to better allocate the economic resources. The use of mechanical models seems particularly suitable to this aim, since based on a limited number of geometrical and mechanical parameters.

The quantitative evaluations based on this model are compared to those obtained through an observational model for the same building stock: although based on very different approaches, results are in a quite good agreement.
Thus a procedure to define a priority list, addressed to take advantages of both models, has been proposed. It aims to combine the specific peculiarities of two approches: on the one hand, in case of mechanical model, the possibility to take explicitly into account the dependence on specific

parameters (particularly useful in r.c. structures built according to an “engineering approach”); on the other one, in case of the observational one, the support on a direct calibration with the earthquake damage survey.

ACKNOWLEDGEMENT
The authors acknowledge the Provincial Authority of Genoa – Seismic Office for the support provided in the proposed application.

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