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ALTERNATIVE RETROFIT STRATEGIES FOR PRE'70 R.C. BUILDINGS: VULNERABILITY MODELS AND DAMAGE SCENARIOS

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SUMMARY

The inherent seismic vulnerability of existing R.C. buildings, designed prior to the introduction of adequate seismic code provisions in the early/mid-1970s, has been dramatically confirmed by the catastrophic socio-economical consequences of earthquake events occurred worldwide in the past decade. Several alternative seismic retrofit/rehabilitation solutions have been studied in the past, few of which have been successfully implemented in practical applications on single buildings. However, due to the typical one-off peculiarity of a retrofit intervention, issues of costs and invasiveness and practical implementation still remain the most challenging aspects for their wide adoption. Recent developments and numerical/experimental validation of viable and low-cost retrofit solutions for pre-1970 buildings within a multi-level retrofit strategy approach, suggest the possible implementation of "standardized" solutions at a urban or territorial scale. In this contribution, the efficiency of such a structural mitigation strategy is investigated, within the framework of a seismic risk analysis approach. To this aim refinements of the models typically adopted for territorial scale vulnerability assessment have been suggested to better represent pre-1970 R.C. building behaviour before and after retrofit intervention, as recently observed in numerical and experimental investigations. Comparative evaluation of the reduced level of the expected damage after alternative retrofit solutions have been carried out and described in terms of fragility curves. Further exemplification of the effects of retrofit strategies planned at a territorial scale have been provided via damage scenarios referred to a case study region in Italy.

1. INTRODUCTION

Recent catastrophic seismic events (e.g. Turkey 1999, 2003 and Taiwan 1999) have highlighted the need for the development of advanced but reliable retrofit solutions for under-designed structures. Latest developments and numerical/experimental validation of viable and low-cost retrofit solutions for pre-1970 buildings within a multi-level retrofit strategy approach, suggest the possible implementation of "standardized" solutions at a urban or territorial scale. Nevertheless, the decision to set a large scale retrofit strategy is still neither straightforward nor obvious as, apart from the implications of a certain solution scheme on seismic demand and supply, there are issues of costs and practical implementation to be accounted for.

Damage scenarios and seismic risk analysis, devoted to the evaluation of the expected losses for a specific earthquake event or the possible losses in a time period, and the representations of their results in a GIS environment could be used as an helpful tools to support decision-making as the planning and prioritization of retrofit or seismic intervention programs at large scale. Comprehensive frameworks for damage scenarios and seismic risk analysis, including GIS-based evaluation tools for end-users, have been developed and proposed as part of major international programs, e.g. HAZUS (1999); RADIUS (1999), Risk-UE (2004), in addition to private implementations carried out by insurance/reinsurance/risk management companies. Regardless of the common framework, based on the traditionally accepted definition of seismic risk, i.e. convolution of hazard, exposure, vulnerability analyses and cost evaluation, alternative methods have been adopted for the seismic

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vulnerability assessment of buildings at territorial scale based on: a) actual damage observation b) expert judgment, c) simplified-mechanical and analytical models.
In this paper a comparative evaluation of the reduced level of the expected damage prior and after the implementation of alternative and recently proposed retrofit strategies, is carried out and presented in terms of fragility curves, by using two vulnerability methods, based respectively on an observational and a mechanical approach, implemented within the Risk-EU project (2004). Refinements of the proposed vulnerability methods are then proposed to better represent the seismic response of pre-1970 R.C. buildings, as recently observed in numerical and experimental investigations. In conclusion, using the refined vulnerability and capacity curves, the effects at territorial scale of the adoption of standardized multi-level retrofit strategies are investigated by means of damage scenario analysis carried out on a case-study region in Italy (Western Liguria).

2. ALTERNATIVE RETROFIT STRATEGIES AND SOLUTIONS FOR PRE 1970s R.C. BUILDINGS

Several alternative seismic retrofit solutions have been studied in the past and adopted in practical applications, ranging from conventional techniques, which utilize braces, jacketing or infills, to more recent approaches, including supplemental damping devices or advanced materials (e.g. Fiber Reinforced Polymers, FRP, or Shape Memory Alloys, SMA). In general, considerations on cost-effectiveness, invasiveness, architectural aesthetic, along with issues related to the socio-economical consequences of an excessive damage and related downtime due to a limited or interrupted functionality of the structures after the seismic event, come into the full picture of such a complex decision-making process. More recently, a low-invasive and cost-effective retrofit solution for pre-1970s frame systems, which relies on diagonal metallic haunches installed locally at the beam-column joints to protect the panel zone and to enforce a more desirable hierarchy of strength, has been presented, after numerical and experimental validations, by Pampanin and Christopoulos (2003) and Pampanin et al. (2006), as a valuable solution for wide application at large territorial scale with particular interest for under-developed countries. Alternative advanced retrofit strategies have been also recently proposed in literature, providing clear and correct distinction between the concepts of "retrofit" and "strengthening", too often, and sometimes improperly, associated. Selective upgrading techniques, proposed by Elnashai and Pinho (1998), aim for example to independently upgrades stiffness, strength or ductility-only of a single member (Fig. 1a). More recently, following the developments of high-seismic-performance systems based on a controlled rocking mechanism, a selective weakening approach has been proposed by Pampanin (2006) as a counter intuitive but efficient retrofit intervention for either frame, walls or floor systems. Preliminary applications of a partial or total selective weakening intervention on a wall system have been presented in Ireland et al. 2006 (Fig. 1b): the intervention aims to develop a more appropriate flexure-type rocking/dissipating mechanism by a) vertical splitting an existing shear-dominated wall, b) disconnecting the longitudinal reinforcement at the base and c) re-enhancing strength and dissipation capacity by adding vertical post-tensioned tendons and external energy dissipation devices (e.g. viscous, friction, Shape Memory Alloys dampers).

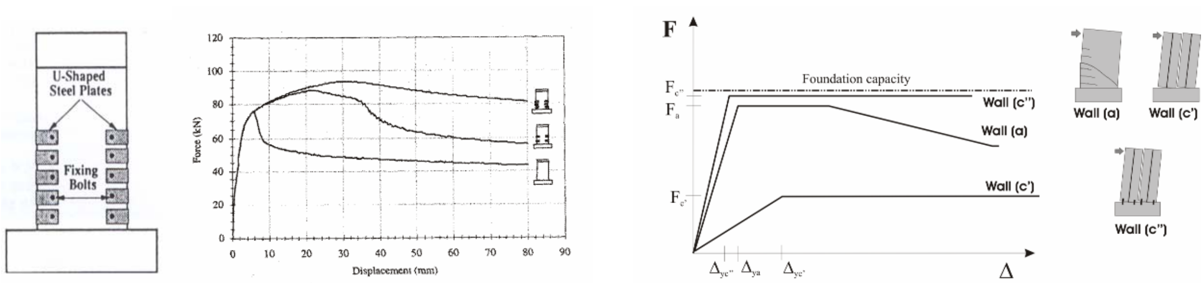

Figure 1. Alternative advanced retrofit strategies: a) ductility-only selective retrofit approach (Elnashai and Pinho, 1998), b) selective weakening approach: vertical + horizontal cuts with post-tensioning (c') + energy dissipation (c'') (Ireland et al. 2006)

2.1 Vulnerability and damage assessment for R.C. buildings within territorial scale analysis

As mentioned, two different vulnerability methods, developed as part of the Risk-UE project (2004) are herein adopted for the assessment of the building vulnerability and for the estimation of the expected damage before and after retrofit: an observed vulnerability approach to be used for an hazard description in terms of macroseimic intensity and a force-based procedure to be implemented when the hazard is described in terms of PGA or response spectra (spectral shape or discrete spectral ordinates).

The observed vulnerability approach, referred to as "macroseismic method" has been originally derived by Giovinazzi and Lagomarsino (2004) from the definitions provided by the EMS-98 macroseismic scale (Grunthal 1998). According to the "macroseismic method", the building vulnerability is measured in terms of a vulnerability damage index V and of a ductility-based index Q. Vulnerability curves allow to obtain the mean damage grade μ_D (defined as the mean value of the expected probability to have damage on respect to the EMS-98 discrete damage level scale D_k k=1÷5, D_1=Slight, D_2=Moderate, D_3=Heavy, D_4=Very Heavy, D_5=Destruction) as a function of the macroseismic intensity, I_{EMS-98} (Fig. 2a). Damage distributions, corresponding to the assessed mean damage grade μ_D, are obtained assuming a binomial function. Fragility curves, defining the probability of reaching or exceeding each damage grade $P[D_k|I,(V,Q)]$ can then be directly derived (Fig. 2b).

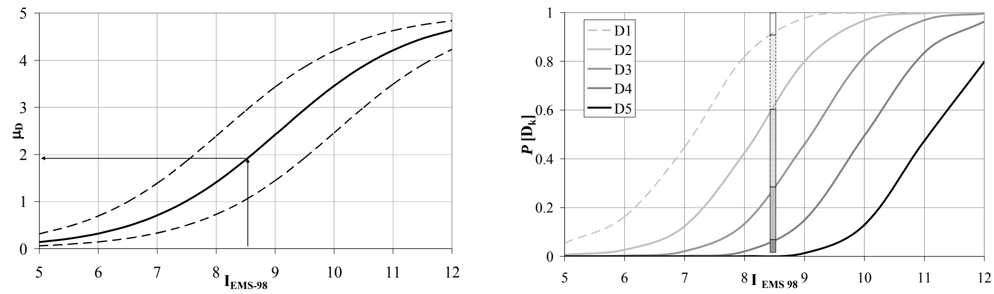

Figure 2. Macroseismic method for medium–rise pre-code R.C. moment frame: a) medium, (V=0.64, Q=2.3) upper and lower vulnerability curves; b) fragility curves $P[D_k]$ as a function of macroseismic intensity I_{EMS-98} and damage probabilities p_k for I_{EMS-98}=8.5

The mechanical method is essentially a capacity spectrum-based method, similar to that adopted by HAZUS (1999). Simplified bilinear elastic-perfectly plastic capacity curves are assumed for the vulnerability representation and described in terms of three parameters: the yielding acceleration a_y, the fundamental period T and the structural ductility capacity μ. Constant-ductility inelastic response spectra are derived from a 5% damped elastic response spectrum $S_{ae}(T)$ by means of a ductility-based reduction factor, R_μ. The displacement corresponding to the performance point S_d^* can thus be directly evaluated, without the need of any further iteration. A four damage limit state scale (D_{Sk} k=1÷4, D_{S1}=Slight, D_2=Moderate, D_3=Extensive, D_4=Complete) related to performance levels S_{dk} (assuming tentative limit states $S_{d1}=0.7d_y$; $S_{d2}=1.5d_y$, $S_{d3}=0.5(d_y+d_u)$, $S_{d4}=d_u$) has been adopted for the damage description. The probability of exceeding each damage state threshold S_{dk} is evaluated from the performance displacement S_d^* by using of a lognormal cumulative function.
Although the proposed macroseismic and mechanical approaches are, in principle, different for derivation and conception, their closed-form formulations allow for a quantitative comparison and reciprocal calibration. As a useful result, refinements in the definition of the mechanical model based on numerical/experimental analysis results can be directly implemented ("translated") into an equivalent macroseismic approach. Dually, the reliability of assumed force- or displacement-based capacity curves can be cross-validated on the basis of real observed damage data. The calibration was performed assuming a similitude in the damage scales ($D_{Sk} = D_k$ k=1÷3 and $D_{S4} = D_4 + D_5$) and evaluating the seismic input providing equivalent level of damage from the two approaches. The correlation between intensity I_{EMS-98} and the peak ground acceleration a_g was, after that, set in the form of $a_g=c_1c_2a_g^{(I-5)}$. The relationships between the capacity curves parameters (a_y and μ, after assuming T) and the macroseismic method indexes V and Q are given as:

$$\begin{cases} V = \dfrac{1}{6.25}\left[8.1 - 0.95Q - \ln_{C_2}\left(\dfrac{a_y}{1.43sc_1}\right)\right] \\ Q = \dfrac{1}{1.35}\ln_{C_2}\left[\left(\mu - 1 + \dfrac{T_C}{T}\right)\dfrac{T}{0.7T_C}\right] \end{cases} if\ T < T_C \quad \begin{cases} V = \dfrac{1}{6.25}\left[8.1 - 0.95Q - \ln_{C_2}\left(\dfrac{a_y}{1.43sc_1}\dfrac{T}{T_C}\right)\right] \\ Q = \dfrac{1}{1.35}\ln_{C_2}\left(\dfrac{\mu}{0.7}\right) \end{cases} if\ T \geq T_C \tag{1}$$

where c_1 and c_2 are the I-a_g correlation parameters and s is a soil factor (e.g. as per EC8-spectra).
Fig. 3a shows the capacity curve for a typical medium–rise pre-code R.C. moment frame derived, according to Eq. (1) from the macroseismic method vulnerability curve (Fig. 2a), when the I-PGA correlation c_1=0.043 c_2=1.6, proposed by Margottini et al. (1992), is assumed and the period T is evaluated as a function of the height (inter-story height h=3m) according to Chopra and Goel (2000). In Fig. 3b it can be observed how the damage distribution associated to a_g=0.25g (equivalent to I_{EMS-98}=8.5 when c_1=0.043 and c_2=1.6), is comparable to the assessment provided by the macroseismic method (Fig. 2b).

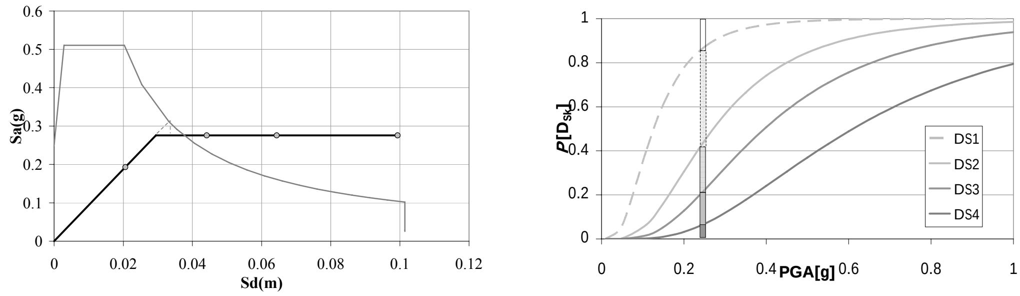

Figure 3. Mechanical approach for medium–rise pre-code R.C. moment frame: a) EC8 elastic response spectrum for a_g=0.25g and Soil A, evaluation of the performance point through capacity spectrum method; b) fragility curves $P[D_k]$ as a function of PGA and damage probabilities p_k for PGA =0.25g

2.2 Evaluation and representation of the effects of alternative seismic retrofit and strengthening

As anticipated, damage scenario analysis can be a fundamental tool to assess the impact of alternative retrofit solutions at territorial scale. At an intermediate step of the full procedure, the effects and efficiency of alternative retrofit strategies can be appreciated by comparing fragility curves corresponding to pre-defined levels of damage (D_k). Simulated retrofit interventions can be easily represented within the macroseismic and the mechanical models by properly specifying the increment (or decrements) of the defining parameters due to the seismic retrofit or strengthening. Fig. 4 shows, as an example, the effects of three alternative interventions, namely, two selective upgrading (strength only and ductility only) and one selective weakening solution, on a low-rise (three storey) pre-1970 fame building according to the mechanical method (§ 2.1).

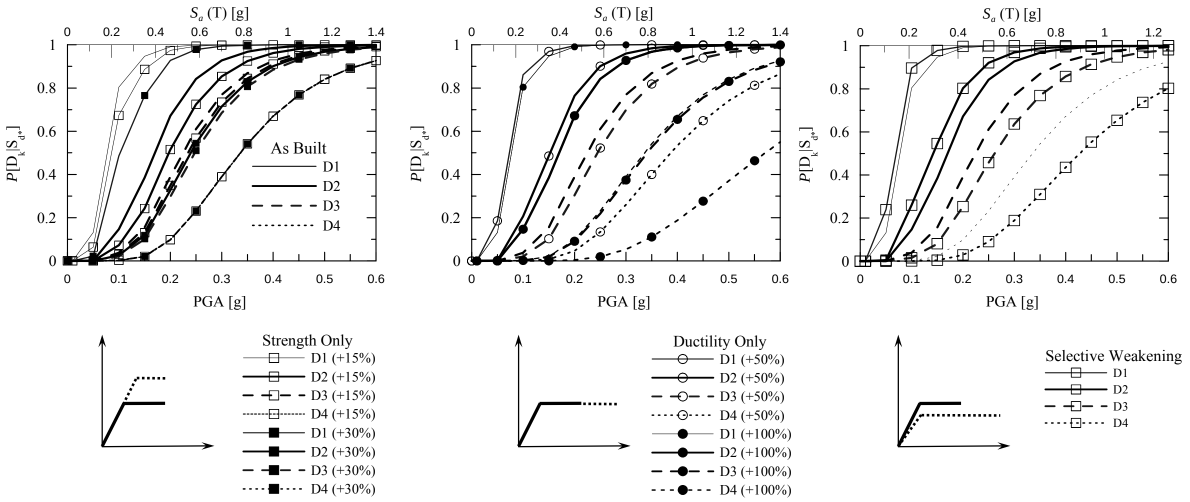

Figure 4. Efficiency of alternative retrofit solutions (strength-only, ductility-only, selective weakening) in terms of fragility curves

It can be noted that each retrofit solution shows a different degree of efficiency at different damage level D_k. The selective weakening solution, herein consisting of reducing the strength by 15% and increasing the ultimate displacement by 1.5 times, would be un-effective at lower level of damage (D_1 and D_2) but more effective at higher levels (D_3 and D_4).

3 IMPROVEMENT OF VULNERABILITY METHODS FOR THE REPRESENTATION OF PRE'70 BUILDINGS BEHAVIOUR

An increased number of experimental and numerical investigations on the seismic performance of pre-1970s R.C. buildings have provided valuable quantitative evaluation of their inherent vulnerability (Hakuto et al., 2000, Park, 2002; Pampanin et al., 2002), as well as favoured the calibration and further development of simplified analytical methods and assessment procedures (i.e. Pampanin et al. 2003). Due to the poor reinforcing details (including lack of transverse reinforcement in the joint region), the absence of capacity design principle and the use of plain round reinforcing bars, undesirable brittle failure mechanisms can occur. In particular, shear damage and failures in the beam-column joint panel zone can lead to peculiar effects on the overall response (Calvi et al. 2002), activating more complex inelastic mechanisms, given by the combination of flexural plastic hinges and joint shear hinges in addition to traditional beam-sway and column-sway mechanism. Moreover, the presence of infills (e.g. typically un-reinforced masonry) can lead to undesirable, yet controversial, effects due to the interaction with the bare frame (Crisafulli et al., 1997, Dolsek and Fajfar, 2001, Magenes and Pampanin, 2004). On one hand, the presence of infills can in fact guarantee higher stiffness and strength, reducing the inter-storey drift demand, thus delaying the formation of a soft-storey mechanism, when compared to the response of a bare frame. On the other hand, the interaction between un-reinforced masonry infills and the bare frame can result in local failures (e.g. short column effects, damage to the joint region) as well as into unexpected soft-storey mechanisms, even in presence of uniformly distributed infills and not necessarily at the first storey. Deformation- or drift -based limit states associated to the joint and infills panel damage, has been proposed by Pampanin et al. (2003) and Magenes and Pampanin (2004).

Fundamental refinements of currently adopted seismic assessment procedure, either directed to a single building or to a class of buildings within a territorial scale vulnerability analysis, could be obtained by properly accounting for the aforementioned damage and collapse mechanisms in the definition of both capacity and demand curves. Moreover, specific improvements of mechanical vulnerability methods for pre-1970 buildings could include the derivation of more realistic capacity curves to account for the actual strength and stiffness degradation due to joint- or infill- related damage mechanisms. Given the high flexibility of existing frame, P-Δ effects should also be considered.

In the following, an attempt to include the aforementioned aspects within the existing vulnerability approaches, while maintaining the desired original simplicity, is given. To this aim, additional parameters are suggested to be introduced to describe the equivalent bilinear capacity curve required by the mechanical method (§ 2.1). More specifically, a post-yielding stiffness parameter r is introduced to represent either the global effects of strength degradation as well as the aforementioned P-Δ effects. The additional strength provided by the presence of uniformly infill panels is then accounted in terms of the maximum infill strength a_{pan} and corresponding displacement d_{pan}).

Moreover, the proposed refinements have been implemented into the macroseismic approach (§ 2.1), by "translating" or cross-calibrating the results obtained of the mechanical method according to Eq. 1.

3.1 Presence of masonry infills

The effects of infills on the response are herein considered with reference to the response of a case-study six-storey three-bays frame system, designed according to Italian code provisions available between the 1950s and 1970s, presented in Magenes and Pampanin (2004). Fig. 5a shows the results from non-linear pushover for two different configurations: 1) uniformly infilled frame (uniform distribution of the infills along the elevation); 2) non-uniformly infilled frame (no infills at the first floor). In both configurations, a double panel arrangements for the infills (typical of external frame) was considered.

The base-shear-top displacement curve obtained from the 2D pushover analysis, have been replaced with an idealized bilinear relationship representative of the SDOF substitute structure. The SDOF system effective height has been evaluated recognizing the likely formation of a soft storey mechanism at the first or second storey for the partially infilled or uniformly infills configurations, respectively. Similarly, the effective initial stiffness K_E, the effective yield strength a_y, and the post-yield stiffness ratio r were derived, The ultimate displacement d_u of the system has been evaluated considering the plastic hinge rotation capacity of the columns, subjected to a soft storey mechanism, thus as a function of the geometric and materials proprieties of the reinforced concrete section (Fig. 5a). With regards to the force-displacement curve obtained for the uniformly infill frame, the secant stiffness to the maximum infill strength a_{pan} has been assumed as the effective lateral stiffness K_E=K_{pan}. As observed by previous studies report in literature (Dolsek and Fajfar, 2001, Magenes and Pampanin, 2004), after extensive cracking of the masonry infills occurs at one floor level, a soft storey mechanism is likely to develop and the overall force-displacement response tend to converge to that expected for a partial infill frame. The ultimate displacement d_u, identify on the capacity curve the ultimate acceleration a_u and thus the effective yield strength a_y, assuming a zero post-yielding stiffness r=0 (Fig. 6a).

Once defined according to the previous considerations, the capacity curves have been translated in terms of equivalent vulnerability curves, to be adopted within a macroseismic approach, by using the closed form relation (Eq. 1) between the capacity curves parameters and the macroseismic method indexes V and Q.

The equivalent vulnerability curves for the partially infilled frame configuration (Fig. 6b), have been immediately obtained as a function of the yielding acceleration a_y, of the fundamental period T and of the ultimate ductility μ provided in Table 1. With regard to uniformly infill frame, the defining parameters V_{pan} and Q_{pan} of the vulnerability curve equivalent to the response of the frame before the peak strength due to the infills, a_{pan}, is exceeded have been obtained from Eq. 1 by imposing $\mu = a_{pan} / a_y$, $a_y = a_{pan}$, $Q = Q_{pan}$. The vulnerability curve equivalent to the building behavior after the infills failure is obtained as a function of the yielding acceleration a_y, of the fundamental period T and of the ultimate ductility μ. The resulting vulnerability curve, representative of the behavior of a uniformly infilled frame, is obtained connecting the two curves in their intersection points (Fig. 5b).

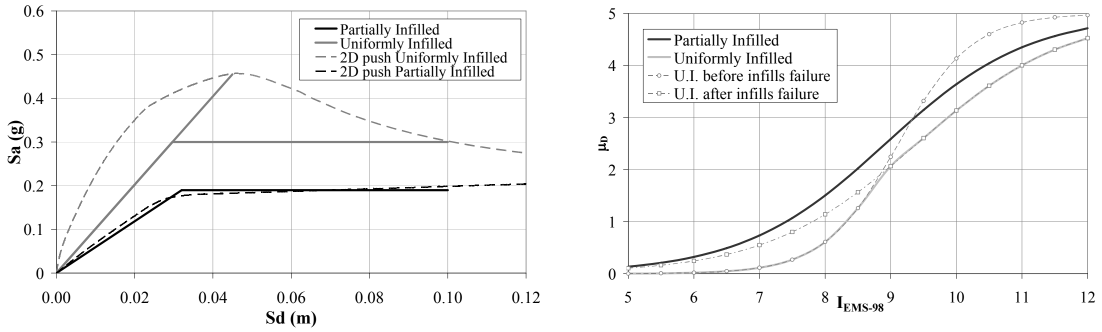

Figure 5. Six storey frame: a) capacity curves from 2D pushover curves; b) corresponding vulnerability curves according to the macroseismic method

Table 1. Six storey frame: defining parameters for the capacity curves and the vulnerability curves

	a_{pan} [g]	d_{pan} [m]	a_y [g]	d_y [m]	T_E [s]	d_u [m]	a_u [g]	μ	V_{pan}	V	Q_{pan}	Q
Partially Infilled	-	-	0.2	0.0320	0.807	0.1	0.2	3.1		0.65		2.2
Uniformly Infilled	0.455	0.045	0.30	0.0293	0.631	0.1000	0.300	3.4	0.64	0.59	1.1	2.3

3.2 Strength reduction and P-Δ effects

P-Δ effects, primarily due to gravity loads acting on the deformed configuration of the structure, can lead to remarkable increase in lateral displacements, particularly when dealing with poorly designed reinforced concrete buildings and should thus be properly accounted for within a vulnerability assessment procedure. According to FEMA356 (2000), the degree by which dynamic P-Δ effects increase displacements depends on different factors: 1) the ratio α of the negative post-yield stiffness to the effective elastic stiffness; 2) the fundamental period of the building, 3) the strength ratio, 4) the hysteretic load-deformation relations for each story, 5) the frequency characteristics of the ground motion and 6) the duration of the strong ground motion. In the framework a "Coefficient Method" for the assessment of the target displacement of the nonlinear MDOF system, FEMA356 introduces a coefficient C_3 in order to account for the P-Δ effects, being function of : the negative post-yield stiffness r, the ratio between the response spectrum at the effective fundamental period and damping ratio of the building Sa(T,ξ) and the yielding acceleration a_y.

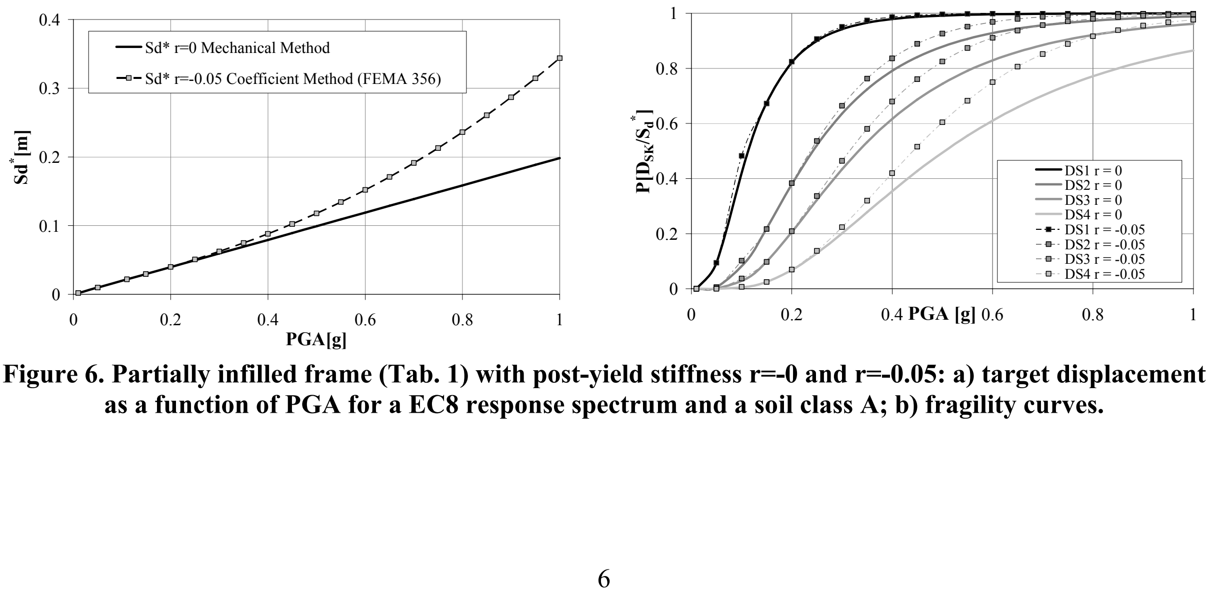

Figure 6. Partially infilled frame (Tab. 1) with post-yield stiffness r=-0 and r=-0.05: a) target displacement as a function of PGA for a EC8 response spectrum and a soil class A; b) fragility curves.

The closed form solution proposed by FEMA 356 for the evaluation of the target displacement is based on the equal displacement rule as for the mechanical method herein adopted (§ 2.1) and can thus directly implemented in the overall framework for a territorial scale seismic risk assessment. This assumption allows a considerable simplification of the procedure and of the computational time for the performance point assessment when compared to a standard iterative procedure, particularly when a negative post-yielding stiffness has to be considered. In Fig. 6a the performance point S_d^* resulting for the partially infilled configuration (Tab. 1) is represented as a function of the PGA[g]. It is worth noting the increase of displacement demand when a negative post-yield stiffness value r=-0.05 is accounted for. The corresponding increment in the expect damage can be observed by means of the fragility curves shown in Fig. 6b

4. IMPACT ASSESSMENT OF TERRITORIAL SCALE ALTERNATIVE RETROFIT STRATEGIES ON PRE-1970S REINFORCED COCRETE BULDINGS

In this session, exemplification of the effects of retrofit strategies planned at a territorial scale are provided via damage scenarios referred to a case study identified with Western Liguria Region in Italy. The building vulnerability and the expected consequences of an earthquake in this area have been investigated as part of an Italian National research project for the definition of Earthquake scenario and strategies for the preservation of historic centers funded by the INGV-GNDT (2004).

The inventorying of the buildings stock including number and characteristics has been carried out processing census statistical data. The total number of current buildings in the selected region is 49372, with RC and URM typologies representing the 36% and 64% of the total, respectively, In spite of the higher number of URM buildings, the majority of population lives in RC buildings (60% out of the total 211349 inhabitants living in RC buildings, and 40% in URM buildings), mostly designed prior to 1981, date of adoption of seismic code provisions in that area (56% pre 1971, 33%, between 1971 and 1981, 12% after the 1981). Focusing on the characteristics of pre'71 buildings, it is worth noting that the majority consists of low rise buildings (59% 1÷2 floors, 33% 3÷5 floors, only 9% have more than 5 floors). Moreover, according to census data, a not negligible part of pre'71 buildings (18%) are of pilotis typology, i.e. infills present only in the upper storeys. The highest concentration of pre'71 buildings is in the costal area where the soil resulting from the geology-based microzoning belongs generally to class C (T_C=0,6). Soil amplification effects can be therefore expected for the pre '71 buildings whose period values range from T≈0.6÷≈0.8 (Table 1).

For the damage scenario analysis, the maximum historical event in the region, corresponding to the Western Liguria Feb 23, 1887 earthquake (M=6.3, I_0 = X, Long=8°,1430, Lat = 43°,7480), which caused over 509 victims, severe destruction in costal towns and villages (Fig. 9a), has been considered.

The expected consequences in terms of damage to buildings and consequences to people have been evaluated for either the as built (AB) and the retrofitted configuration, after the implementation of two different retrofit strategies: a partial retrofit (PR) and a total retrofit (TR). According to a multi-level retrofit strategy approach (Pampanin and Christopoulos, 2003), a partial retrofit, aiming to achieve an intermediate performance objective, could be targeted if a full upgrade (total retrofit) is not achievable or impractical from a cost and invasiveness point of view. The effects of retrofit interventions have been represented in terms of increased strength, stiffness and ultimate displacement when compared to the as built configuration (Table 1).

In particular, for the partial retrofit (PR): +15% strength, +10% stiffness and +150% ultimate displacement. For the total retrofit (TR): +25% strength, +20% stiffness and +200% ultimate displacement. The two retrofit interventions have been applied to all the pre'71 buildings. Then the retrofit intervention has been restricted considering the R.C. pre'71 buildings supposed to be more vulnerable: high buildings or pilotis. Table 4 shows the number of buildings, the total area and the cost for either a PR and a TR intervention. The cost have been evaluated by multiplying the "typical structural cost" in Table 2 for the building area to be retrofitted. For simplicity, the structural cost of a typical seismic retrofit intervention for existing R.C. buildings has been herein evaluated according to the FEMA156 (1994) Option2 evaluation approach which require the knowledge of: the building area, the building location, the construction starting date, the number of buildings in the inventory and the targeted performance objective after retrofit. According to these guidelines, the typical structural cost, to seismically rehabilitate a building, C, (expressed in $/sq. ft.) is estimated as a product of five different factors: 1) C_1 building group mean cost (C_1=20.02 $/sq. ft for R.C. moment frame building), 2) C_2 building area adjustment factor (considered areas are: S-small <1000m^2, M-medium 1000 m^2÷4999 m^2, L-large 4999 m^2÷10000 m^2, VL-very large areas >10000 m^2), 3) C_3 seismicity/performance objective adjustment factor, 4) C_L location adjustment factor (C_L=1 is assumed for this application), 5) C_T time adjustment factor (C_T =1.621 assuming an inflation rate =4% for the 2006). Table 2 shows the direct cost of retrofit C for concrete moment frame and frame with infill walls, as a function of the performance level, the building area and for a moderate seismicity.

Table 2. Typical structural cost to seismically rehabilitate RC building

Performance Levels	Life Safety				Damage Control				Immediate Occupancy			
Area	S	M	L	VL	S	M	L	VL	S	M	L	VL
Moderate Seismicity	25.10	24.41	23.26	19.34	30.05	29.22	27.84	23.16	49.49	48.13	45.86	38.14

* The unit cost is expressed in terms of dollars per square foot \$ sq.ft. where m^2 =10.76 $sq.ft^2$.

In particular it has been assumed that a Total Retrofit solution, TR, would target a "Damage Control" performance level while a Partial Retrofit solution, PR could target a Life Safety one. The earthquake dependent costs have been roughly evaluated considering: 1) building damage and collapse, assuming a cost of 2500€/m^2 for the evaluation of the building value and the ratio between cost of repairing and cost of replacement set as: 0.01 for D_1, 0.1 for D_2, 0.35 for D_3, 0.75 for D_4, 1 for D_5; 2) injuries and fatalities, assuming according to Hopkins (2006) a cost of 6250€ for injury and a cost of 195000€ for life; 3) temporary shelter, assuming a cost of 6250€ for each temporary housing unit suitable, on average, for four people.

Table 3. Consequences to building and people for as built condition (AS) and for different hypothesis of retrofit interventions: partial retrofit (PR) and total retrofit (TR)

		<'71 all			<'71 high -rise			<'71 pilotis all			<'71 pilotis high-rise		
N° building		9855			1794			1792			290		
Surface (m^2)		4074774			2019650			685001			313583		
Inhabitants		80764			39791			13048			6020		
Retrofit interventions		AB	PR	TR	AB	PR	TR	AB	PR	TR	AB	PR	TR
Retrofit Cost (M€)		*-*	*852*	*1021*	*-*	*421*	*505*	*-*	*143*	*172*	*-*	*65*	*78*
Buildings	Uninhabitable	487	179	125	164	56	38	243	111	75	71	33	22
	Collapsed	36	6	2	15	2	1	25	5	2	10	2	1
People	Homeless	7665	2684	1790	4448	1523	1008	3292	1580	1053	1788	865	576
	Casualties	231	38	17	138	22	10	155	33	15	90	19	9
Earthquake Cost (M€)		*1676*	*816*	*658*	*555*	*237*	*180*	*717*	*383*	*287*	*213*	*109*	*79*

The results of the damage scenario simulation, shown in Table 3 in terms of consequences to buildings and people (mean values), confirm the efficiency of a partial retrofit intervention in drastically reducing the effects of the selected earthquake event. Conversely, the additional reduction provided by the implementation of a total retrofit solution seems not be justified, from a cost-benefit point of view, for a territorial scale implementation.

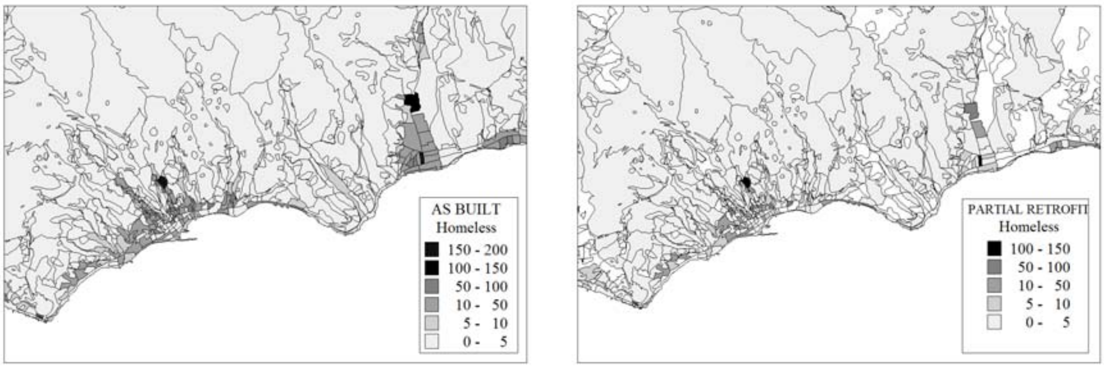

Figure 7: Homeless people: a) as built conditions, b) after a partial retrofit intervention

As additional advantage of the results provided by a damage scenario analysis within a GIS-environment, a comprehensive and rational risk mitigation strategy can be defined, consisting of alternative levels of intervention (ranging from total retrofit to no- action) within specific unit of analysis, depending on the computed seismic risk. As an example it has been considered the possibility to limit the retrofit intervention only to the census tracts with an average damage level greater or equal than D_k=2. The number of buildings involved in the intervention the resulting costs and the obtained benefits are presented in Table 4

Table 4. Consequences to building and people before and after the retrofit intervention within selected census tract

		<'71 all			<'71 high -rise			<'71 pilotis all			<'71 pilotis high-rise		
N° building		317			169			45			24		
Surface (m^2)		273118			190315			27086			18239		
Inhabitants		4626			394			3191			247		
Retrofit interventions		AB	PR	TR	AB	PR	TR	AB	PR	TR	AB	PR	TR
Retrofit Cost (M€)		-	*57*	*68*	-	*39*	*47*	-	*6*	*7*	-	*4*	*5*
Buildings	Uninhabitable	108	31	19	57	17	10	26	13	9	14	7	5
	Collapsed	13	2	1	7	1	0	6	1	1	3	1	0
People	Homeless	1449	392	235	962	255	153	214	114	76	132	70	46
	Casualties	53	6	2	33	3	1	16	4	2	9	2	1
Earthquake Cost (M€)		*287*	*101*	*69*	*156*	*55*	*38*	*62*	*33*	*24*	*34*	*18*	*13*

The example has been proposed only with the purpose of showing the potentiality of damage scenario analysis implemented in a GIS environment. It is well recognized that a comprehensive cost benefit analysis should require to account for many others factors such as: 1) initial benefits and costs (including costs of retrofitting and relocation of occupants during retrofitting, and benefits of increased property values), 2) time dependent benefits and costs (including maintenance, depreciation, insurance and assessed rental differentials), 3) earthquake dependent benefits and costs (including building damage, loss of contents, injuries and fatalities and an allowance for overall business interruption and social disruption). An example of detailed benefit-cost study of actually designed retrofit intervention can be found in Hopkins et al., (2006), as part of a feasibility study (funded by a World Bank) to retrofit residential buildings in Istanbul.

Moreover, it is worth noting that, while the results presented in this paper as case-study damage scenario have been referred to a specific earthquake event, the whole procedure can be directly implemented in the form of a complete probabilistic framework, by assuming a probabilistic hazard demand (e.g. PSHA, Cornell, 1968).

6. CONCLUSIONS

In this contribution, the potentialities of using damage scenario and seismic risk analysis as a support to seismic retrofit strategies at a territorial scale level has been discussed and exemplified, with reference to a macrosesimic and a mechanical vulnerability models, recently developed as part of European research projects. Positive features of the proposed vulnerability methods and risk analysis tool include: the possibility to be implemented with different level of data availability, an easy implementation from the computational point of view and the possibility of cross-correlation between the two methods. Based on the experimental and numerical evidences on the seismic response of pre-1970s reinforced concrete buildings with or without masonry infills, tentative suggestions for refinements of the current mechanical model (or equivalent macro-sesismic) to more accurately represent the seismic vulnerability of pre-1970s reinforced concrete buildings with or without infills have also been given. Comparative evaluation of the effects of alternative retrofit solutions, relying on selective upgrading or weakening techniques, have been carried out and presented in terms of fragility curves. In conclusion, an example of a damage scenario analysis prior and after the adoption of a multi-level retrofit strategy, has been given, referring to a case study area in Italy. Based on the results, it can be suggested that a quick implementation in critical sub-areas or regions of "partial" retrofit solution can be seen as a rational, feasible and efficient strategy able to drastically reduce to a manageable level the consequences of the seismic event.

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