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SEISMIC ASSESSMENT AND RETROFITTING OF R.C. EXISTING BUILDINGS

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SUMMARY

A great part of the existing r.c. structures built in Italy (but even in the Mediterranean basin) have been designed without considering seismic-induced actions and seismic criteria for strength and ductility design.
Assessing of seismic behaviour of existing building can be faced according to main focuses, namely in terms of either maximum strength against the horizontal seismic actions and maximum ductility, consisting in the capability for plastic displacements.
According with the most modern seismic codes and guidelines (i.e. Eurocode 8 and FEMA 178 and 273) seismic behaviour of structures must be analysed in terms of performance both under service and destructive earthquakes.
Seismic response of r.c. structures under seismic shaking should be analysed by means of complex non-linear dynamic procedures requiring a large number of accelerograms for describing the possible seismic input and great computational efforts. However, Capacity Spectrum Method (CSM) provides a solution very easy-to-obtain and reliable enough.
In the present paper, such a method has been widely adopted for evaluating the performance point of the structures considered in the analyses.
Seismic assessment of non-seismically designed structures generally results in pointing out structural deficiencies related to a general lack of strength and ductility of both a certain number of members and the structural system as a whole. Retrofitting is generally a need for such a structures and various strategies can be considered for improving the seismic performance of the structure under earthquake shaking.
In the present paper different possible retrofitting solutions (additional steel bracing systems, wrapping and strengthening of structural members by means of traditional materials or FRP) able to improve the seismic behaviour of non-seismically designed structures have been considered and compared, pointing out the differences between the various strategies.

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INTRODUCTION

Seismic behaviour, vulnerability assessment and design of strengthening intervention for seismic retrofit of the existing r.c. structures are one of the most challenging topics for the structural engineer in the twenty-first century, especially in that region where social and economical development has occurred in the last decades, after the Second World War. That is the case of Europe where reconstruction activity started in the first 50s and continued throughout the 60s and 70s during the huge economy growth registered in some European countries such as Italy, Spain or Greece. In these countries, as in the other emerging countries lying on the Mediterranean basin, a lot of the built heritage has been generally designed without seismic criteria, even if all the mentioned region is characterized by a medium-to-high seismicity. For this reason the most recent European Code of Standards for structures in seismic zones (Eurocode 8, 2003a) gives great attention to both r.c. and masonry existing structures; Part 3 of Eurocode 8 (Eurocode 8, 2003b) is completely devoted to that subject dealing with some specific aspects such as knowledge requirements of existing structures, members and materials, analysis methods, strengthening techniques. Italian New Seismic Code (2003) is substantially inspired to the last version of Eurocode 8, dealing with both new and existing structures.

Various documents devoted to assessment and retrofit of existing structures have been issued in more seismic-prone countries outside Europe as well; for example, in the U.S. recommendations for quickly assessing the structural vulnerability of structures has been firstly proposed (FEMA 178, 1995), while some guideline for rehabilitating the same structures has been provided two years on (FEMA 273, 1997).

Research activity in the last decade has been focused on various topics related with seismic assessment and retrofitting of existing buildings. The most important contributions about reinforced concrete existing structures have been collected in a State-of-the-Art Report (*fib*, 2003a) edited by the Task Group 7.1; it firstly states the performance objectives for existing structures and secondly deals with both characterizing strength and deformation capacity of non-seismically detailed components and seismic retrofitting techniques. On the contrary seismic assessment procedures, both force based and displacement based, are only briefly treated. Indeed, another State-of-the-Art Report (*fib*, 2003b) is completely devoted to Displacement Based Seismic Design of Structures reviewing the most important contributions available in the recent scientific literature and pointing out that displacement based procedures are even suitable for existing structures assessment before and after strengthening.

The first contribution about displacement based analysis of structures is due to Freeman (1975), which proposed the well-known Capacity Spectrum Method consisting in obtaining the performance point of a structures under seismic action by utilizing the capacity curve of the structure and the damped elastic spectrum, considering a substantial equivalence of hysteretic dissipation and damping in terms of structural response. Indeed, this assumption received many criticisms by various researchers which contributes to improve the original Freeman formulation of the CSM method. The very synthetic and complete review of the evolution of the CSM-like methods for seismic analysis of structures is reported in Fajfar (1998), before presenting the N2-method which defines the structural performance by comparing the capacity curve of the structures with an inelastic spectra related to the seismic action and the structural inelastic dissipation.

Both CSM and N2 methods work with capacity curves of the structural system. Such curves can be obtaining by means of static non-linear analysis (the so-called *pushover analysis*) that is for sure much less time-consuming than time-history analysis. So, simplified method such as CSM or N2 have been widely adopted by Code of Standards (recently the latter one has been adopted by Eurocode 8, 2003a,b), because they represents a reasonably adequate procedure for design purposes. Software for determining capacity curves of structures by means of pushover analysis are no more confined to the academic framework, but is getting more and more popular between the practicing structural engineers.

Mechanical non-linearity can be introduced according to different models. The most refined software adopt a *fiber discretization* of the cross-sections throughout the element axis in order to utilize unidirectional material stress-strain-relationships for defining the non-linear mechanical behavior of sections and members (OpenSees, 2003). A simplified model can be implemented by considering a *sectional approach*, based on the direct utilization of moment-curvature relationships for member section (Kunnath et al. 1992). For design purposes, a so-called *global approach* may be applied, consisting in concentrating member non-linear behavior in few sections where deformations are expected to be high (SAP2000, 1995); such sections are called plastic hinges and their behavior is characterized by a moment-rotation curve that has to be determined in its main parameter on the bases of the structural strength and deformation capacity. Furthermore, yielding and ultimate bending moments have to be evaluated and corresponding hinge rotations have to be defined and determined. While determining moment values is not so complicated, defining and evaluating hinge rotations at yielding and at ultimate limit state is generally much harder. Various approaches and models have been proposed in scientific literature for quantifying such rotations and a wide review is reported in the above mentioned fib Report (2003a). One of the most recent proposals is due to Panagiotakos and Fardis (2001) which proposed two methods for quantifying ultimate and yielding rotation. The first one is empirical in character, based on the multiple regression of a database composed by about 1300 tests results on beam-column sections. The second one considers the basic definition of plastic hinge and introduces an expression of the plastic hinge length (to be multiplied by the section plastic curvature) depending upon the shear length (or the section depth) and the ratio between steel yield stress and the square root of the concrete strength: the hinge length is higher as such a ratio is high, because more likely is the slip occurring between steel bar and concrete in that case. Both formulae have been adopted by Eurocode 8 (2003b) and will be utilized in the following.

In fact, in the present paper, displacement based procedures for assessing seismic behavior of structures before and after strengthening are widely utilised for assessing the seismic performance of r/c buildings in the light of both EuroCode 8 (EuroCode 8, 2003) and the recent Italian Seismic Provisions (New Italian Seismic Code, 2003). In particular, one of the building type presented in Faella et al. (2002) is considered in the analysis; it is a typical structures built in Italy between the 60s and the 70s for residential purposes. Seismic assessment are conducted starting from a capacity curve obtained by a pushover analysis in which plastic hinges characteristics are determined according to the relationships provided by the codes mentioned above. The N2-method is widely utilized as a simplified procedure for determining the maximum displacement demand for structures under a seismic event described by the corresponding response spectrum; a performance-based multi-level assessment procedure is described for quantifying seismic vulnerability of the examined structure.

Seismic assessment of non-seismically designed structures generally results in pointing out structural deficiencies related to a general lack of strength and ductility of both a certain number of members and the structural system as a whole. Retrofitting is generally a need for such structures and various strategies can be considered for improving the seismic performance. In the present paper a performance-based procedure for rationally strengthening of seismically inadequate structures is presented as well.

ASSESSMENT PROCEDURE

In the present section the structure characteristics, the numerical model and the determination of plastic hinge properties are treated in order to obtain the capacity curve of the structure and determin the achieving of the different Performance Levels introduced by Eurocode 8 (2003b) for existing structures.

Performance levels for existing structures

Three performance levels, referred as Limit States, are considered in Eurocode 8 – Part 3 (2003b) and in the New Italian Seismic Code (2003) for existing structures:

- Limit State of Damage Limitation (LS of **DL**, as referenced in Eurocode – Part 3, 2003b);
- Limit State of Significant Damage (LS of **SD**);
- Limit State of Near Collapse (LS of **NC**).

The given structure achieves one of the Limit States (namely, Performance Level in the PBD terminology) mentioned above when the first of its members or sections achieves the corresponding demand in terms of generalized displacement; if the structure is analyzed by means of a *concentrated* *plasticity model*, the member demand can be considered in terms of plastic hinge rotations. Eurocode 8 (2003b) adopts the Panagiotakos and Fardis (2001) proposals for evaluating both yielding θ_y and ultimate θ_u rotations of plastic hinges. The former quantity can be evaluate as follows

$$\theta_y = \phi_y \frac{L_V}{3} + 0.0025 + \alpha_{sl} \frac{0.25\varepsilon_{sy} d_b f_y}{(d - d')\sqrt{f_c}} \tag{1}$$

being

ϕ_y	the section curvature at steel yielding;
L_V	the so-called "shear length" equal to the ratio between the bending moment and the shear force acting in the section;
$\alpha_{sl} \frac{0.25\varepsilon_{sy} d_b f_y}{(d - d')\sqrt{f_c}}$	is the contribution of bar slip to the yielding rotation which is greater as the steel yielding strain ε_{sy} and stress f_y and the diameter d_b are great, while decreases with the section depth (related to the distance $d - d'$ between the main reinforcing bars) and steel-to-concrete bonding strength which is proportional to the square root of the concrete compressive strength f_c; α_{sl} is a boolean parameter: if $\alpha_{sl} = 1$ slip contribution is considered, while it is neglected for $\alpha_{sl} = 0$.

On the contrary, both an empirical model based on experimental data regression and a mechanical model have been provided for ultimate rotation θ_u. In the following, only the latter one is reported, commented and will be utilized in the analyses; it defines ultimate rotation depending on ultimate and yielding curvature and the extension of plastic hinge providing the following expression:

$$\theta_u = \theta_y + (\phi_u - \phi_y) \cdot L_{pl} \cdot \left(1 - \frac{0.5 L_{pl}}{L_V}\right) \tag{2}$$

where rotations θ and curvatures ϕ are coupled with the subscripts "u" or "y" when referred to ultimate or yielding condition, respectively, while L_{pl} is the plastic hinge length that can be evaluated as follows

$$L_{pl} = 0.08 \cdot L_V + \frac{1}{60} \alpha_{sl} d_b f_y \tag{3}$$

by adding two contributions related to member dimensions (represented by the shear length L_V or, alternatively, by the section depth) and a possible bar slip.
Further comments about both yielding and ultimate rotation formulas (1) , (2) and (3) can be found in one of the referred State-of-the-Art Reports (fib, 2003b).
Rotations evaluated by means of the above relationship can be referred to the Limit States introduced for describing the Performance Level of the given structure:

- LS of **DL** is achieved when the first member (namely, the first hinge) reaches the yielding rotation defined by relationship (1) ($\theta_{DL} = \theta_y$);
- LS of **SD** is achieved when in the first hinge the rotation reaches the value defined as follows

$$\theta_{SD} = \theta_y + \frac{3}{4} \cdot (\theta_u - \theta_y) \tag{4}$$

- LS of **NC** is achieved when the first hinge reaches the ultimate rotation defined by relationship (2) ($\theta_{NC} = \theta_u$).

Figure 1 shows a likely moment-rotation curve for plastic hinges considering the values of rotations which defines the various Limit States considered by Eurocode 8 for existing structures.

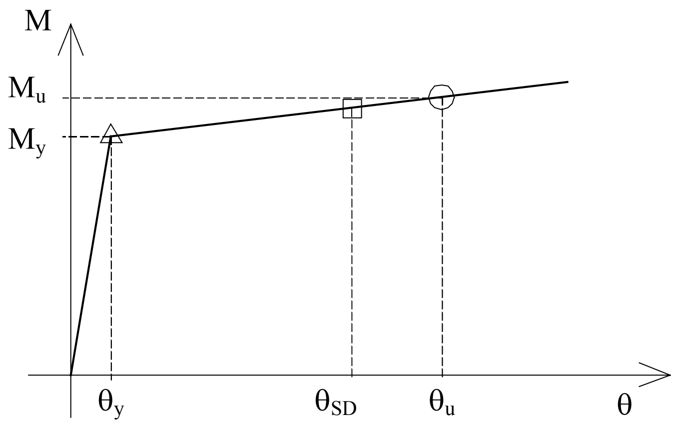


Figure 1: Plastic hinge moment-rotation curve and Limit States according to Eurocode 8 – Part 3 (2003b)

Member capacity controls structural response and results in defining the structural capacity at the various Limit States defined above. Different Earthquake Design Levels are coupled to each Limit State; seismic demand is generally assigned by means of the shape of the Elastic Response Spectrum and PGA value. The first one mainly depends on the frequency content of the seismic shaking expected in the given site and is generally related to the subsoil stiffness: five subsoil classes are considered by Eurocode 8 – Part 1 (2003a). The second one depends on the seismicity of the zone. Earthquake Design Level is assigned in terms of PGA for a given spectrum shape depending on the site where structure lies:

- LS of **DL**: a design ground acceleration PGA/2.5 has to be considered, corresponding to a return period of about 70 years;
- LS of **SD**: a design ground acceleration PGA, corresponding to the reference return period of 475 years;
- LS of **NC**: a design ground acceleration 1.5 PGA, corresponding to a return period of about 1000.

Seismic demand has to be amplified (or reduced) for structures and facilities of major (minor) importance for handling emergence when seismic event occurs, realizing an implicit Performance Based framework. Coupling a Performance Level and the corresponding Earthquake Design Level, the required Performance Objective is obtained that can be more restrictive depending on the destination of the structure.

Assessment procedure

In the present section a procedure for determining the seismic behavior of existing structures in the framework of a multi-level performance based framework, like that described above, is discussed in detail. It is based on N2-method described by Fajfar (1998) which works with a capacity curve obtained by means of a pushover analysis and a capacity spectrum. Throughout the capacity curve the three points corresponding to the achievement of the Limit States defined above can be considered in terms displacement capacity (Figure 2).

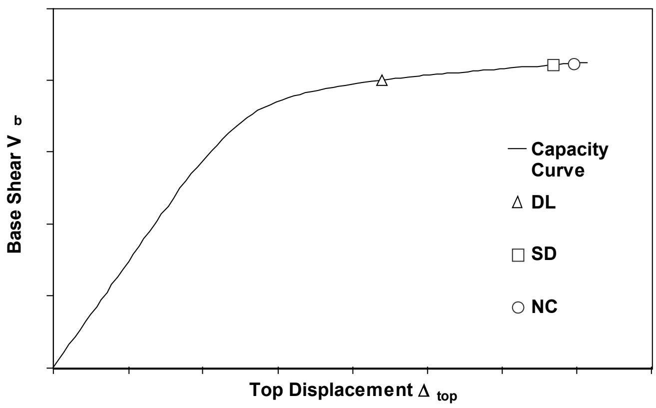


Figure 2: Possible capacity curve and Limit States for the given structure according to Eurocode 8 – Part 3 (2003b)

N2-method works with a transformed capacity curve for the equivalent SDOF system which generic parameter P^* can be obtained by the corresponding parameter P evaluated for the MDOF structure as follows:

$$P^* = \frac{P}{\Gamma} \tag{5}$$

being Γ a parameter accounting for the assumed lateral displacement shape and masses:

$$\Gamma = \frac{\sum_i m_i \Phi_i}{\sum_i m_i {\Phi_i}^2} \tag{6}$$

where Φ_i is the modal displacement and m_i is the mass of the i-th floor of the structure. Eurocode 8 prescribes that two different displacement shapes have to be considered in the analyses:

- an "uniform" pattern, based on lateral forces that are proportional to mass regardless of elevation;
- a "modal" pattern, proportional to lateral forces consistent with the lateral force distribution determined in elastic analysis.

Transformed capacity curve has to be bi-linearized to an elastic-plastic format considering (namely, area) energy equivalence between the two curves. Such a curve can be represented in an ADRS format after dividing the base shear values by the equivalent SDOF mass m^*; in the same reference system the seismic demand can be represented.

Displacement demand can be now determined by means of N2-method; if the generic Limit State LS is considered, a value $\Delta_{c,LS}$ of the displacement capacity of the structure can be read directly on the capacity curve (Figure 2). Moreover, the corresponding displacement demand $\Delta_{d,LS}$ can be evaluated by applying N2-method. Generally, $\Delta_{d,LS}$ can be evaluated by determining the structure Performance Point by intersecting the Inelastic Response Spectrum and the Capacity Curve; in Figure 3 the graphical construction is represented for a case in which the "equal displacement rule" applies, because the elastic SDOF, characterized by the same period of the elastic-plastic SDOF, intersects the Elastic Response Spectrum in the constant Pseudo-Velocity branch. In the same figure, an usual situation occurring for existing structures, designed without considering seismic actions and anti-seismic criteria, is represented; in fact, displacement demand $\Delta_{d,LS}$ is larger than the corresponding capacity value $\Delta_{c,LS}$. Utilizing these two quantities, a vulnerability parameter $V_{DSP,SL}$ in terms of displacement can be defined for the considered Limit State as follows:

$$V_{DSP,SL} = \frac{\Delta_{d,SL}}{\Delta_{c,SL}} \ . \tag{7}$$

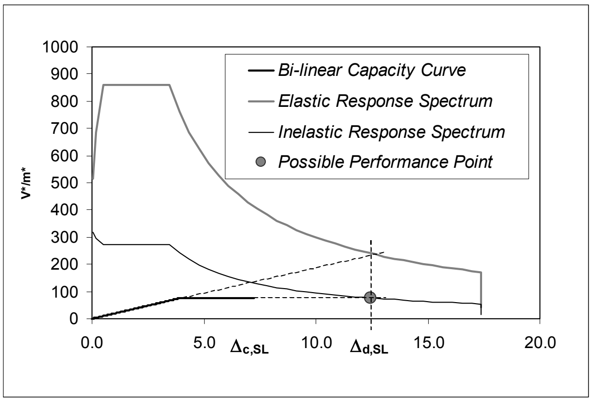


Figure 3: Graphical representation of N2-method for determining displacement demand.

In the framework of a multi-level approach for evaluating seismic performance of existing structures the parameter V_{DSP} can be defined for the structures with reference to all the three considered Limit States:

$$V_{DSP} = \max_{SL}\{V_{DSP,SL}\} \ . \tag{8}$$

The parameter V_{DSP} measures the seismic vulnerability of a structure: if it is lesser than the unity, the given structures complies the security levels prescribed by Eurocode 8 – Part 3 (2003b). On the contrary, if its value is greater than one, the capacity is not great enough with respect to the expected demand. In this case, the given structure has to be strengthened and the same parameter can be utilized for assessing the effectiveness of the strengthening intervention and for operating design choices between different retrofitting techniques.

Finally, for a given structure characterized by a displacement capacity $\Delta_{c,LS}$ with respect to the generic Limit State, it is possible to determine the limit value $PGA_{c,SL}$ of the peak ground acceleration for which the displacement demand $\Delta_{d,LS}$ is equal to the capacity $\Delta_{c,LS}$ (see Faella et al., 2002). If $PGA_{d,SL}$ is the value of the peak ground acceleration corresponding to the same Limit State, a similar vulnerability parameter could be introduced in terms of forces:

$$V_{PGA} = \max_{SL}\{V_{PGA,SL}\} \qquad \text{being} \qquad V_{PGA,SL} = \frac{PGA_{d,SL}}{PGA_{c,SL}} \ . \tag{9}$$

In the present paper, only the first one will be considered because in the authors' opinion displacement based assessment techniques are more direct than force based ones; indeed, a design criterion will be proposed for retrofitting intervention in the framework of a displacement based design approach.

The analyzed structure

Assessment procedure presented in the previous section is now applied to a typical seven-storey building representing a very common structure built in Italy in 60s and 70s for residential purpose. The floor scheme is represented in Figure 4 where three deep beams runs throughout the longitudinal direction and bear the floor slab, forming longitudinal frames; in the transverse direction only two internal and two external linking beams are present, while no other links are provided between the main longitudinal frames. Stair structure is realized by means of an inclined beam. A more detailed description of the present structure can be found in Faella et al. (2002).

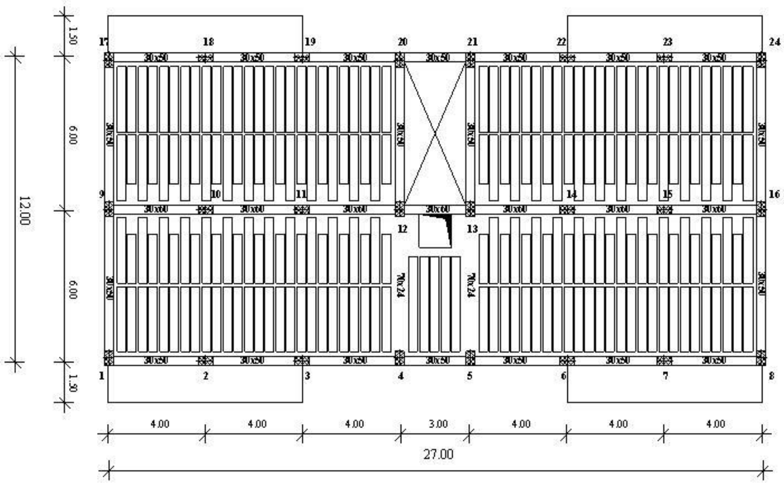


Figure 4: Floor scheme of the considered structure

Pushover analysis can be carried out under different hypotheses influencing the definition of the structural model and the assumption in the global analysis; in particular, it is possible to consider or neglect the two following aspects:

- the presence of rigid offsets in the element corresponding to the beam-to-column joints zone;
- the analyses can be carried out considering or neglecting P-Δ effects.

Four different analysis cases can be obtained by combining the two aspects listed above. Pushover analyses have been conducted in both longitudinal and transversal direction considering or neglecting element rigid ends and P-Δ effect.

In longitudinal direction (Figure 5), considering P-Δ effects generally results in a significant reduction in terms of both strength and ductility; on the contrary, slightly higher stiffnesses and significantly greater strengths are generally reached when considering rigid ends for beam elements. Displacement capacity generally decreases when accounting for P-Δ effects, while increases in that cases in which rigid end presence in the beam-to-column joint zone is considered.

In transversal direction, it can be interesting to consider the difference in behavior depending on the contribution of the stair structures (whose position in plan can be seen in Figure 4) to stiffness and lateral strength. Stairs can be generally realized by means of an inclined slab supported by beams spanning in longitudinal direction or can be realized considering each step as a cantilever member clamped in an inclined beam (generally referred as "knee beam"). In the first case no contribution of the stair structure to the global behavior can be assumed, while in the second case the presence of inclined beams ranging from a floor to the following (or the previous) provides a significant contribution to lateral stiffness and strength of the structure. Such contribution can be quantified by comparing capacity curves represented in Figure 6 and Figure 7 for the four combinations obtained considering and neglecting the effects listed above.

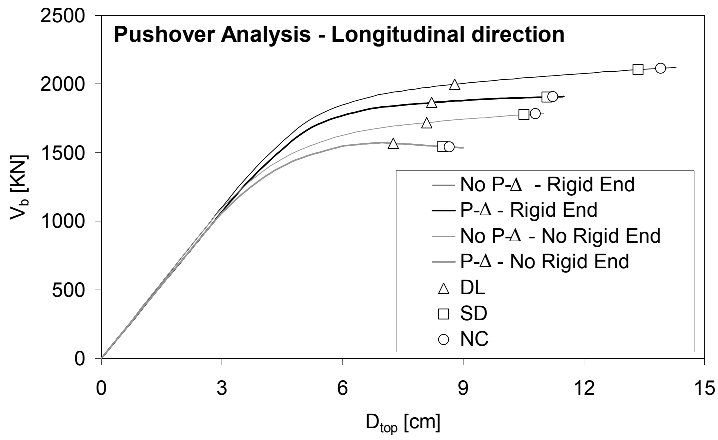


Figure 5: Capacity curves along the longitudinal direction

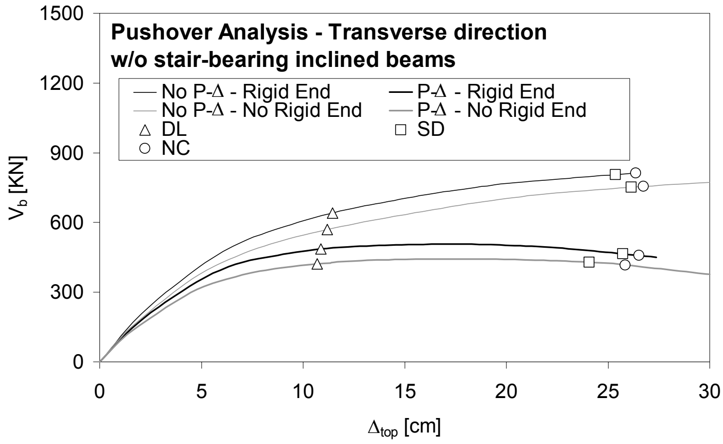


Figure 6: Capacity curves along the transversal direction – without considering inclined stair-bearing beams

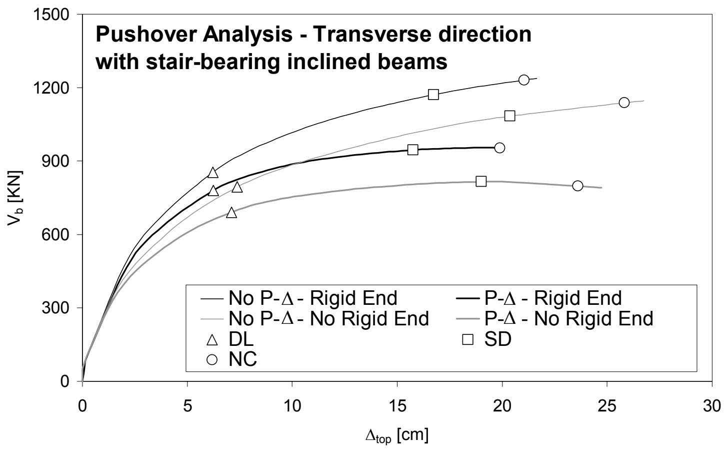


Figure 7: Capacity curves along the transversal direction –considering inclined stair-bearing beams

Curves reported in Figure 7 generally denote a greater value for maximum base shear and lateral stiffness when compared with the corresponding ones in Figure 6.

General trends about the role played by P-Delta effects and rigid ends already pointed out for longitudinal direction can be even confirmed for transverse direction; in particular, accounting for P-Δ effect generally results in a strength decrease, even if for transversal direction a slight increase in capacity is obtained as well. Figure 8 and Figure 9 summarize the influence of rigid ends (RE) and P-Δ effects in terms of displacement capacities.

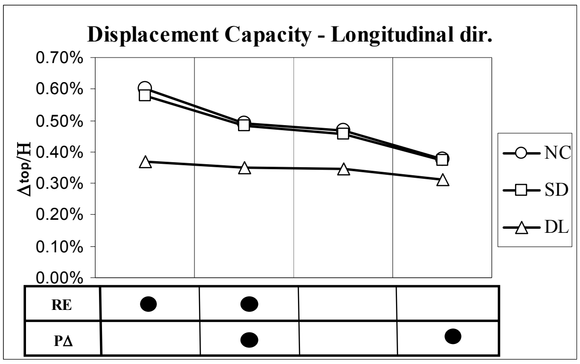


Figure 8: Displacement capacities depending on the hypotheses considered in analysis – Longitudinal direction

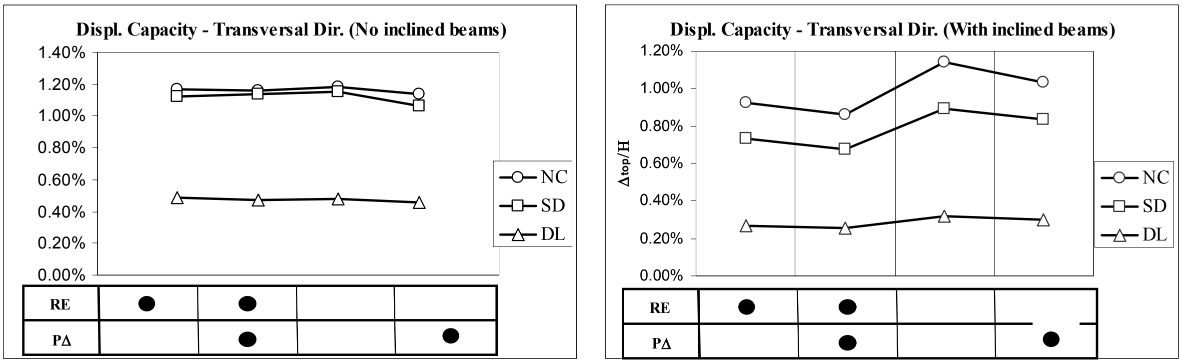


Figure 9: Displacement capacities depending on the hypotheses considered in analysis – Transversal direction

Finally, it is useful to observe that the above figures make reference to pushover analyses conduced under a "modal" force pattern representing the first mode of vibration of the structure; analyses have been also conducted under a constant force pattern as prescribed by Eurocode 8; the capacity curves obtained under this hypothesis generally exhibit greater displacement ductility with respects to the curves represented in the above figures for the various Limit Sates.

In the last three figures, points corresponding to the achievement of the three Limit States introduced by Eurocode 8 – Part 3 (2003b) are also represented and will be utilized in the following to apply the presented assessment procedure to the considered structure.

Seismic vulnerability evaluation

Assessment procedure presented in the first paragraph of the present section can be now applied to the considered structure whose behavior under horizontal forces has been described in the previous paragraph by representing the capacity curves obtained in both the main directions; in the following, only the case with inclined beams in transversal direction is reported.

Applying the presented assessment procedure results in determining three values of the parameter $V_{DSP,SL}$ defined in equation (7) and choosing the largest one as representative of the structure fitness to face the seismic induced actions. They depend on the Seismic Zone and on the Subsoil Category according to Eurocode 8 – Part 1 (2003a) classification; as a reference, considering the case of a structure lying in Zone 1 (the highest in seismicity with a reference value of PGA=0.35 g) and in Subsoil Class A (the stiffest one according to the Eurocode classification) the values of the parameter $V_{DSP,SL}$ reported in the histograms represented in Figure 10 can be determined following the assessment procedure based on N2-Method presented above. The figure accounts the structural behavior along both the main directions, considering the effect of inclined beams on the transversal response; moreover, P-Δ effects and rigid ends for frame elements have been considered in the following as a reference case.

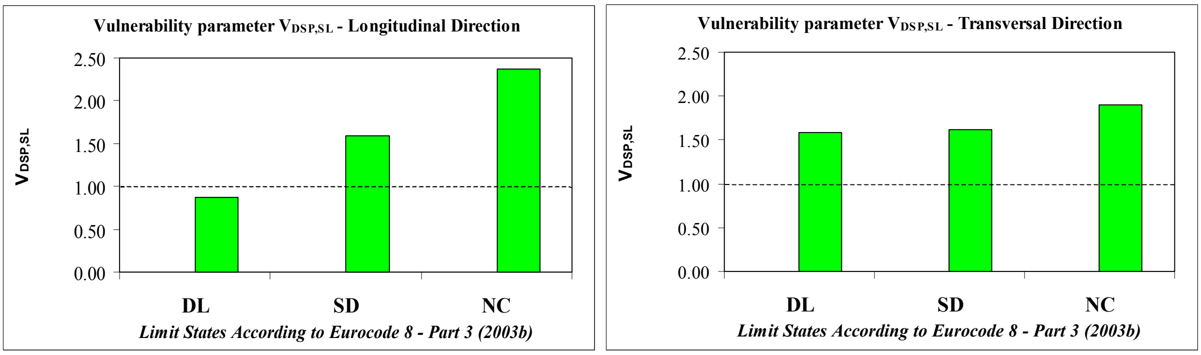


Figure 10: Multi-level Performance Based Vulnerability Assessment for the considered structure.

Figure 10 shows that the analyzed structure does not comply the Limit States verifications according to Eurocode 8 – Part 3 (2003b) requirements for existing structures. The vulnerability parameter $V_{DSP,SL}$ is lower than the unity only for the Limit State of Damage Limitation along the longitudinal direction. On the contrary, its value is grater than one in all other cases giving a measure of how much lower is the *capacity* for the generic limit state SL with respect to the corresponding *demand*. Either in longitudinal and transversal direction the worst situation in term of vulnerability occurs at the Limit State of Near Collapse ($V_{DSP} = V_{DSP,NC}$).

As a final application of the assessment procedure, Figure 11 shows the influence of the subsoil class of the vulnerability parameter $V_{DSP,LS}$. In all cases and along both direction, Limit State of Near collapse is always the control level because $V_{DSP;NC}$ is always greater than the values corresponding to the other subsoil class.

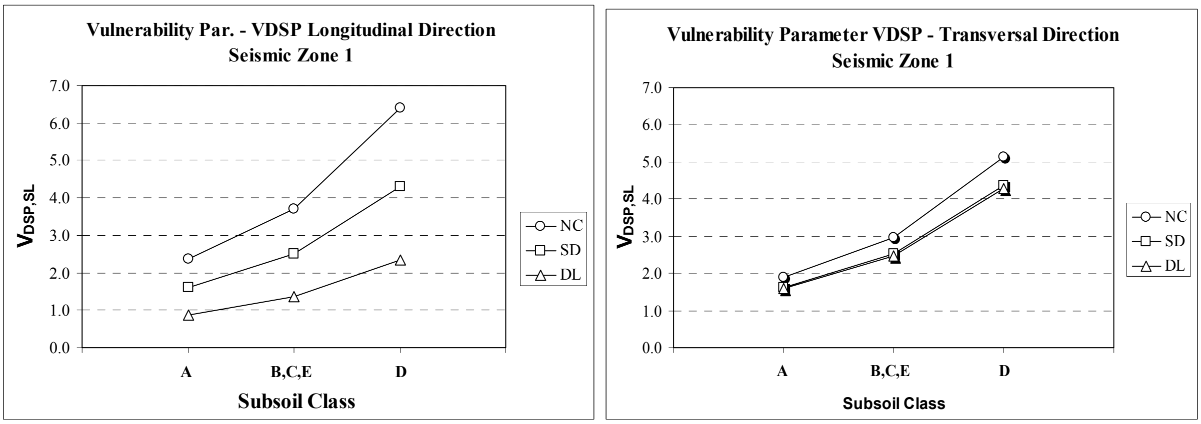


Figure 11: Vulnerability parameter $V_{DSP,SL}$ depending on subsoil class

Finally, Figure 11 shows that subsoil minor stiffness results in increasing unbalance between displacement capacity and demand because the latter one hugely increase as subsoil is less stiff. In all cases the proposed assessment procedure gives a quantitative information about the lack of capacity in the examined structure.

RETROFIT STRATEGY

The same assessment procedure can be utilized to check the effectiveness of possible strengthening interventions on the structure in exam with the aim of complying the Limit State requirements stated by Eurocode 8 – Part 3 (2003). Displacement based approach to seismic analysis of structures is focused on comparing displacement capacity and demand; the proposed assessment methodology introduce this principle in a performance based framework.

Designing a strengthening criterion consist of choosing some intervention aiming to obtain enough displacement capacity with respects to the demand corresponding to a given performance objective. For this reason retrofitting strategy can be basically founded on *increasing capacity* or *reducing demand* in terms of displacement.

For *improving displacement capacity* of structures it is necessary to improve the capacity of its more engaged members; in this sense, various techniques can be utilized for increasing members capacity in terms of both strength and ductility. However, increasing global capacity up to the demanded levels of strength and ductility by means of increasing local capacity of the structural members can be very expensive or not practicable at all, because the global behavior of structures under seismic-induced action is mainly due to a strength hierarchical conception of the members resulting in a ductile behaviour of the structure as a whole. This principles and criteria are the bases of the well-know Capacity Design that can be suitably utilized for designing new structures, but it is not appropriate to strengthening existing one.

For this reason, in the authors' opinion retrofitting strategy have to be mainly focused on *reducing seismic-induced displacement demand* into the capacity limits of the existing structure. This strategy can be pursued by a further structural system devoted to work in parallel with the existing structure against horizontal actions. Such an abstract structural system can be practically realized by means of r.c. shear walls or steel bracing, that have to be designed to be fit for resisting horizontal forces while its displacements range has to be within the limits of the displacement capacity of the existing structure.

For this reason, designing such retrofitting system has to account of both seismic demand and capacity of the existing structure. In the following, a rational design procedure for this structures is briefly outlined in the framework of a multi-level verification approach and making use of the formalism of the N2-method. Finally, an application to the examined structure is shown as an example.

Formulation of a strengthening strategy

A displacement-based design procedure is now presented for choosing a suitable substructure for reducing seismic demand on the existing structure. Under the formal standpoint the proposed design strategy is based on the N2-Method already referred in the previous paragraphs of the paper.

The design of the bracing substructure manages with the following quantities:

- the displacement Δ^* and the corresponding base shear-to-mass ratio V^*/m^* of the equivalent SDOF defined in the N2-Method;
- the elastic response spectrum corresponding to the seismic demand.

Let Δ^*_{tar} be the displacement capacity of the existing structure; this value represent the maximum displacement even for the bracing structure for being compatible with the existing one capacity. If the "equal displacement" rule applies (as really does in the range of constant pseudo-velocity, but can generally assumed in a design hypothesis), the stiffness of the equivalent SDOF after retrofitting should be evaluated as follows

$$K_d = \frac{m^* S_{el,ADRS}(\Delta^*_{tar})}{\Delta^*_{tar}} , \tag{10}$$

where $S_{el,ADRS}$ is the spectral acceleration corresponding to displacement capacity Δ^*_{tar} according to the assumed elastic response spectrum in the usual acceleration-displacement (ADRS) format (Figure 12).

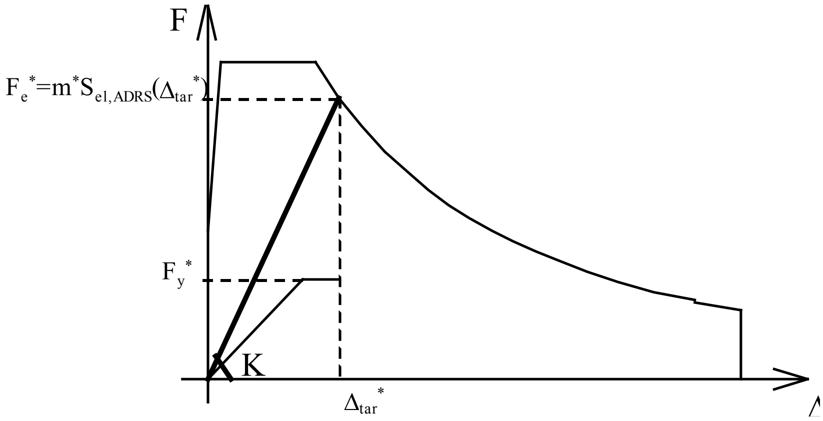


Figure 12: Graphical representation of the strengthening strategy according to the N2-Method formalism

Such stiffness represent a minimum value of K_d for retrofitted equivalent SDOF to comply with the displacement balance between capacity and demand. Indeed, such value is invariant with respect to the transformation between MDOF and the equivalent SDOF represented by equation (5) because the following equality applies:

$$K_d = \frac{F_e}{\Delta_{tar}} = \frac{F_e/\Gamma}{\Delta_{tar}/\Gamma} = \frac{F_e^*}{\Delta_{tar}^*} = \frac{m^* \cdot S_{ADRS}\left(\Delta_{tar}^*\right)}{\Delta_{tar}^*} . \tag{11}$$

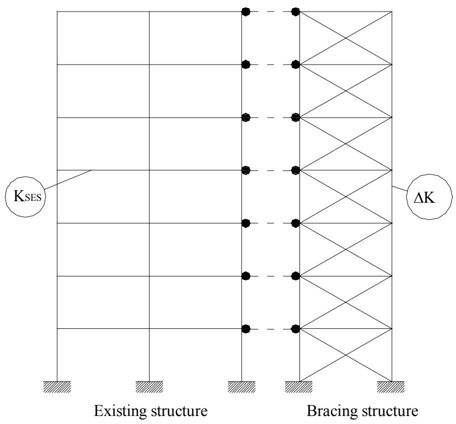


Figure 13: Existing structure and bracing system in parallel

For achieving a global lateral stiffness K_d the bracing system working in parallel with the existing structure needs to have the following lateral stiffness:

$$\Delta K = K_d - K_{ES} \tag{12}$$

K_{ES} being the lateral stiffness of the existing structures.

Furthermore, in a multi-level performance-based design approach, a different value ΔK_{SL} for each limit state considered for the existing structure can be evaluated and the final value of the bracing stiffness can be obtained as follows:

$$\Delta K = max\left\{\Delta K^{DL} ; \Delta K^{DS} ; \Delta K^{CO}\right\} . \tag{13}$$

Example of application of the presented strategy to the analyzed structure

Application of the presented strengthening procedure will be briefly summarized with reference to the existing structure examined above; for the sake of brevity, only the retrofitting intervention in transversal direction will be treated.

Based on the use of the considered building for residential purpose, an architectonic condition is assumed as a constraint: bracing system in transversal direction has to be placed in correspondence of the lateral facades, assumed to be blind as usually occur.

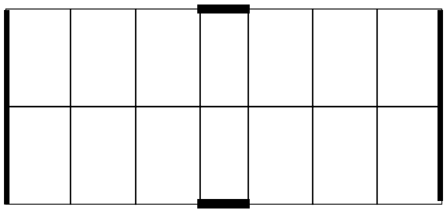
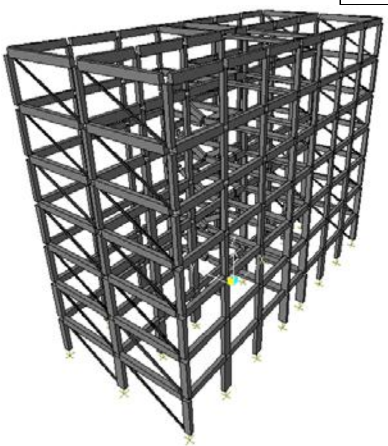


Figure 14: Position of bracing substructures

The design parameters reported in Table 1 can be obtained by applying the design procedure described in the previous section to the existing structure in transverse direction. Even for retrofitting purpose the limit state of Near Collapse controls the design, requiring the greater value of the bracing stiffness for the structure to be retrofit. Steel diagonals providing the required stiffness have been placed in correspondence of the end transversal frames; in such frames and in few other members local strengthening interventions aimed to improve their capacity have been also provided~~,~~ because braces hugely increase their engagement (such intervention would not be required if a completely independent bracing system has been provided).

Table 1: Design-related parameters for bracing structure

	DS	SD	NC
Δ^*_{tar} [cm]	4.002	10.779	13.697
K_d [kN/cm]	491.38	423.22	589.74
K_{ES} [kN/cm]	194.77	141.56	136.03
ΔK [kN/cm]	296.61	281.65	**453.71**



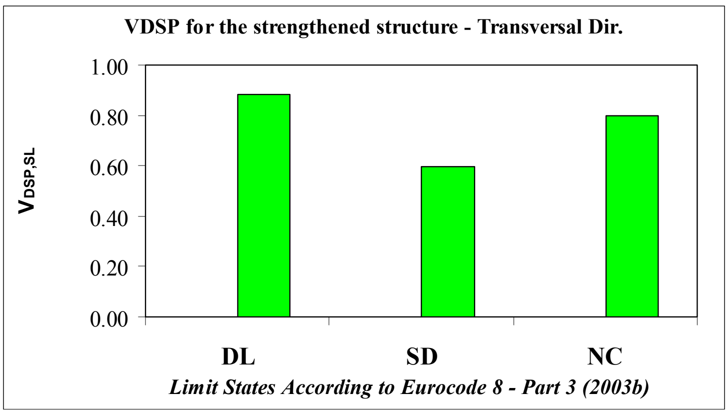


Figure 15: Model of the strengthened structure and results of the assessment procedure

Figure 15 show the model utilized for simulating the effect of the bracing system and the results of the assessment procedure utilized for verify the compliance of the strengthened structure at Eurocode 8 provisions with reference to a subsoil belonging to Class A and Seismic Zone 1 (PGA=0.35 g): a vulnerability index V_{DSP} lesser than 1.0 has been obtained for all the Limit States considered.

CONCLUSIONS

In the present paper, the very challenging topic dealing with assessment and retrofitting of existing buildings designed without seismic criteria has been treated.
An assessment procedure has been proposed starting form N2-method and defining structural performance in terms of displacement capacity and demand. The sample application of the proposed procedure to a typical building emphasized how easy and quick can be its application. As a brief parametrical investigation, the influence of subsoil stiffness on the seismic vulnerability of the building has been analyzed pointing out that vulnerability can be much larger as subsoil is less stiff.
A rational design procedure for choosing the retrofitting system has been proposed with the aim of determining the key mechanical characteristics of a bracing system working in parallel with the existing structure for complying the safety requirement provided by Eurocode 8 – Part 3 entirely devoted to existing (non only reinforced concrete) structures. In the proposed design procedure, according to a displacement-based-approach, the strengthening substructure is designed in terms of lateral stiffness, because displacement demand is strictly controlled by the displacement capacity of the existing structure. For this reason, usual force-based design procedures suitable for new structures, in which displacement capacity is only imposed by the new structure itself, are not directly applicable for bracing system utilized for retrofitting existing structures.

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