Accounting for Progressive Damage in Large Scale Seismic Risk Assessment of RC Buildings

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SUMMARY
Performing nonlinear dynamic analyses to assess the seismic vulnerability of structures is time consuming and requires a certain level of knowledge of the buildings and the seismic events. In this work, a simplified method to assess the seismic performance of reinforced concrete (RC) frame structures that accounts for progressive damage is derived. The SP-BELA method is modified in order to estimate the capacity curve of a structure that is damaged by a sequence of earthquakes. The formulation requires only a basic knowledge of the structure and the displacement spectrum, related to the zone where the building is settled. The method is firstly developed for a single degree of freedom system and then extended to two RC frame structures. The results show good agreement with finite element analysis, with a 20-30% range of error, which is good considering that the method is intended to be applicable in large-scale seismic vulnerability assessment.

Keywords: seismic assessment, progressive damage, RC frame buildings, SP-BELA method

1. INTRODUCTION

The problem of assessing the capacity of existing structures to sustain the effects of an earthquake has always represented a big challenge for structural engineers, in particular in the past few decades, when specific and detailed seismic codes have been developed and applied in the vast majority of the countries of the world. Several methods have been proposed in the past to fulfil the purpose on large scale structural assessment. Most of them are based on the post-processing of the results of nonlinear analyses on a detailed model of the structure, which has to be implemented in a finite element (FE) code, and this can be a strong limitation for several reasons. First of all, nonlinear analyses can be very time consuming. Moreover, FE codes are often expensive and require very skilled and trained users to be utilized as a reliable tool. Finally, sometimes the large amount of information about the structure, that are required by the existing procedures to obtain good results, can be unavailable. In fact, in some countries with a long construction tradition, like Italy, some of the original designs can be lost or damaged or lacking in the key information for developing a detailed FE model.

Another important aspect of the seismic assessment of existing buildings is the effect of progressive damage. In some cases, a specific seismic event is characterized by a main shock of strong intensity that is followed, in the next hours or days, by one or more aftershocks. Even if the intensity of the aftershock alone would not be able to cause any serious damage to some of the structures, considering that the damage, which has been produced by the main shock, has not been repaired and that the structure has not been retrofitted, the subsequent seismic event can cause, in some structures, an heavy level of damage and, in some cases, the structure itself can collapse. Moreover, sometimes the history of the seismic events, that a particular construction has undergone, is unknown, as well as the eventual retrofitting or repair interventions that have been carried out on it. For all these reasons, the development of a procedure to evaluate the capacity of an existing building and to predict, with good approximation, its structural response under the effect of progressive damage is of crucial importance. This procedure should be simple and quick to apply, it should require no FE analyses and it should need the smallest possible level of detail in the knowledge of the building characteristics. As regards the information about the damaging events, a good and simple procedure should not require any original record but only a basic knowledge of the seismicity of the site where the construction is located, in the form of a spectrum. The purpose of this work is to develop a building assessment methodology which presents all the aforementioned characteristics.

Borzi *et al.* (2008) have developed the Simplified Pushover-Based Earthquake Loss Assessment (SP-BELA) method for RC structures. SP-BELA is a method that estimates, with small error, the capacity curve of a structure, without requiring numerical analysis or detailed knowledge of the structure itself. It is based on the principles and equations of displacement based design and considers the yield, severe damage and collapse limit states. However, classical SP-BELA methodology only allows the user to predict the capacity curve of the undamaged building.

The aim of this work is to expand the SP-BELA method in order to obtain a simple procedure, which is able to estimate the residual capacity curve of a RC frame building subjected to progressive damage. Firstly, Single Degree Of Freedom (SDOF) cantilever structures, with different cross section characteristics, have been analysed in order to empirically determine a general correlation between the stiffness reduction of the capacity curve due to damage, the seismic input and the cross section properties. This correlation has been included in the simplified procedure to estimate the residual capacity curve of the SDOF systems after a damaging seismic event. Then, the simplified procedure has been applied to two RC multi storey prototype frame buildings and the results have been compared to the ones obtained with FE analyses. Finally, the simplified procedure has been applied to the same two buildings to estimate the residual capacity after a series of two damaging seismic events and the results have been checked performing FE analyses of the two buildings. All the models and the analyses described in this paper have been developed using the Finite Element software GEN 2011 (MIDAS Information Technology Co., 2010), considering force-based fiber elements.

2. DEVELOPEMENT OF A SP-BELA BASED PROCEDURE ACCOUNTING FOR DAMAGE ON A SINGLE DEGREE OF FREEDOM STRUCTURE

The analysed SDOF structure consists of a 3 meters tall RC column and it is loaded, at its top, with a 69 tons lumped mass. The section is a 0.3×0.3 m square of Rck30 concrete and it is reinforced with 4ϕ16 FeB44k steel rebars, corresponding to a mechanical percentage of reinforcement $\omega^{(1)}$=0.103. In order to find a general procedure, allowing the application of the SP-BELA methodology to the damaged section and, eventually, to evaluate its residual capacity, the column has been subjected to several pushover analyses: the first one on the initial undamaged structure and others after performing a nonlinear time history analysis. In total, 12 time histories and, consequently, 12 pushover analyses have been performed. The definition of both the undamaged and the damaged capacity curves of the column has been carried out according to the following steps:

1. Pushover analysis to determine the capacity curve of the undamaged section of the system;
2. Time-history analysis to generate some damage to the section;
3. Pushover analysis on the damaged section to determine the residual capacity curve of the SDOF;
4. Application of the SP-BELA procedure to the undamaged structure.

The considered accelerograms correspond to real seismic events that have been recorded in Italy, during the past forty years, by different stations. The records have been taken from the Italian Accelerometric Archive (ITACA, http://itaca.mi.ingv.it) database and have been automatically corrected when exported. Among all the possible available records, the choice of these twelve has been taken in order to cover an acceptably wide range of PGA values (from 2.06 to 6.53 m/s^2). For further details on the SDOF structure and on the damaging earthquakes, see Miglietta *et al.* (2012).

For each of the i^{th} seismic event, the following quantities have been calculated for the section 1 with $\omega^{(1)}$=0.103:

$K_{PO,Un}^{(1)}$ stiffness of the elastic segment of the undamaged section pushover curve computed, according to Circolare 2 febbraio 2009 N.617 CS.LL.PP, as in Eqn. 2.1;

$K_{PO,Dam}^{(i,1)}$ stiffness of the elastic segment of the damaged section pushover curve, which is computed as the previous one;

$r_{K1}^{(i,1)}$ ratio between the elastic stiffnesses of the undamaged and damaged section pushover curves, computed as in Eqn. 2.2;

$K^{(1)}{}_{SP-BELA,Un}$ stiffness of the elastic segment of the SP-BELA bilinear curve, corresponding to the undamaged section, equal to 1460.4 *kN/m*;

$T_1^{(1)}$ secant period of vibration, calculated as in Eqn. 2.3;

$S_{d1}^{(i,1)}$ spectral displacement corresponding to $T_1^{(1)}$ on the displacement spectrum of the i^{th} seismic event.

$$K_{PO,Un}^{(1)} = \frac{0.6 \cdot V^{(1)}{}_{base,max,Un}}{\Delta_{0.6 \cdot V^{(1)}{}_{base,max,Un}}} = 2137.70\ kN/m \tag{2.1}$$

$$r_{K1}^{(i,1)} = \frac{K_{PO,Un}^{(1)}}{K_{PO,Dam}^{(i,1)}} \tag{2.2}$$

$$T_1^{(1)} = 2\pi \sqrt{\frac{M}{K^{(1)}{}_{SP-BELA,Un}}} \tag{2.3}$$

Assuming, as a basic hypothesis, that the ratio between the elastic segments of SP-BELA and of the capacity curve is the same whether the undamaged or the damaged section is considered, i.e. supposing that:

$$\frac{K_{SP-BELA,Dam}^{(i,1)}}{K_{PO,Dam}^{(i,1)}} = \frac{K_{SP-BELA,Un}^{(1)}}{K_{PO,Un}^{(1)}} \tag{2.4}$$

it is possible to compute the value of $K_{SP-BELA,Dam}^{(i,1)}$ as:

$$K_{SP-BELA,Dam}^{(i,1)} = \frac{K_{SP-BELA,Un}^{(1)}}{K_{PO,Un}^{(1)}} \cdot K_{PO,Dam}^{(i,1)} = K_{SP-BELA,Un}^{(1)} \cdot \frac{K_{PO,Dam}^{(i,1)}}{K_{PO,Un}^{(1)}} = \frac{K_{SP-BELA,Un}^{(1)}}{r_{K1}^{(i,1)}} \tag{2.5}$$

Knowing the ratio $1/r_{K1}^{(i,1)}$and the value of $K_{SP-BELA,Un}^{(1)}$, obtained through the SP-BELA procedure, it is possible to determine $K_{SP-BELA,Dam}^{(i,1)}$ using Eqn. 2.5. In order to obtain a general expression for $r_{K1}^{(i,1)}$, its values for the 12 seismic events have been interpolated to find a data-fitting second-order polynomial function that correlates the quantities $r_{K1}^{(i,1)}$ and $S_{d1}^{(i,1)}$with the minimum squares method. The function has the expression of Eqn. 2.6. Defining then $V_{R,C}^{(1)}$ as the section shear capacity, $M_{R,C}^{(1)}$ as the flexural capacity of the section computed considering, as axial force, the weight of the lumped mass, and *l* as the length of the column, the final shear capacity of the column $V_C^{(1)}$ is computed through Eqn. 2.7.

$$r_{K1}^{(i,1)} = \frac{K_{PO,Un}^{(1)}}{K_{PO,Dam}^{(i,1)}} = 122.36 \cdot \left(S_{d1}^{(i,1)}\right)^2 - 4.8456 \cdot S_{d1}^{(i,1)} + 1.2406 \tag{2.6}$$

$$V_C^{(1)} = \min\left\{V_{R,C}^{(1)}; \frac{2M_{R,C}^{(1)}}{l}\right\} \tag{2.7}$$

Having determined $K_{SP-BELA,Dam}^{(i,1)}$ by using Eqn. 2.5, one can calculate:

$$\begin{aligned}\Delta_{LS1,Dam}^{(i,1)} &= \frac{V_C^{(1)}}{K_{SP-BELA,Dam}^{(i,1)}} = V_C^{(1)} \cdot \frac{r_{K1}^{(i,1)}}{K_{SP-BELA,Un}^{(j)}} = \Delta_{LS1,Un}^{(1)} \cdot r_{K1}^{(i,1)} = \\ &= \left(122.36 \cdot \left(S_{d1}^{(i,1)}\right)^2 - 4.8456 \cdot S_{d1}^{(i,1)} + 1.2406\right)\end{aligned} \tag{2.8}$$

where $\Delta_{LS1,Un}^{(1)}$ is the elastic limit displacement of the SP-BELA bilinear curve of the undamaged structure (Borzi *et al.*, 2008) and $\Delta_{LS1,Dam}^{(i,1)}$ is its equivalent on the SP-BELA bilinear curve of the damaged structure.

The displacement values of the damaged structures $\Delta_{LSi,Dam}^{(i,1)}$, corresponding to the other limit states LS2 and LS3, are set equal to the ones of the original structure. In case of $\Delta_{LS1,Dam}^{(i,1)} > \Delta_{LS2,Un}^{(1)}$, the system is considered to be less ductile, as a consequence of the damaging earthquake, and in case of $\Delta_{LS1,Dam}^{(i,1)} > \Delta_{LS3,Un}^{(1)}$, the system behaviour is assumed to be elastic-brittle. Relationships aiming to modify the displacement capacity of the damaged limit states will be the objective of further development. With these considerations, it is possible now to draw the SP-BELA$_{Damaged}$ bilinear curve for the cracked section.

Since the final goal is to find an empirical relation that gives the value of $\Delta_{LS1,Dam}^{(i,j)}$, where j indicates the generic section, as a function of $\omega^{(j)}$, $S_{d1}^{(i,j)}$ and $K_{SP-BELA,Un}^{(j)}$, the whole procedure has been repeated. More specifically, the analyses have been conducted on other six SDOF systems, having a transversal section with the same geometric characteristics of the original one, but with a different amount of longitudinal reinforcement A_s, in order to change the mechanical percentage of reinforcement $\omega^{(j)}$. In this way, it has been possible to have enough observed results for the development of the formulation.

The seismic input for the nonlinear time-history analyses has been kept the same. Table 2.1 displays all the different mechanical percentages of reinforcement $\omega^{(j)}$ that have been analysed and the corresponding equations that relate $r_{K1}^{(i,j)}$ to $S_{d1}^{(i,j)}$. Since a considerable number of graphs relating $r_{K1}^{(i,j)}$ and $S_{d1}^{(i,j)}$ are now available, it is possible to plot some graphs that correlate the coefficients $a^{(i,j)}$ $b^{(i,j)}$ $c^{(i,j)}$ of the Eqn. 2.9 with the mechanical percentage of reinforcement $\omega^{(j)}$. The general correlation law is expressed by Eqn. 2.10.

$$r_{K1,quad}^{(i,j)} = a^{(i,j)} \cdot \left(S_{d1}^{(i,j)}\right)^2 + b^{(i,j)} \cdot S_{d1}^{(i,j)} + c^{(i,j)} \tag{2.9}$$

$$\begin{aligned}r_{K1,quad}^{(i,j)} &= \left(-19064 \cdot \left(\omega^{(j)}\right)^2 + 4360.3 \cdot \omega^{(j)} - 167.84\right) \cdot \left(S_{d1}^{(i,j)}\right)^2 \\ &+ \left(2086.9 \cdot \left(\omega^{(j)}\right)^2 - 546.43 \cdot \omega^{(j)} + 34.847\right) \cdot S_{d1}^{(i,j)} \\ &+ \left(-46.337) \cdot \left(\omega^{(j)}\right)^2 + 12.423 \cdot \omega^{(j)} + 0.2766\right)\end{aligned} \tag{2.10}$$

Table 2.1. Summary Of The Relationships Between $\boldsymbol{r_{K1}^{(i,j)}}$ And Different Values Of $\omega^{(j)}$

$\omega^{(j)}$	**Relationship for** $\boldsymbol{r_{K1}^{(i,j)}}$
0.04	$r_{K1}^{(i,j)} = 54.451 \cdot \left(S_{d1}^{(i,2)}\right)^2 + 6.8963 \cdot S_{d1}^{(i,2)} + 0.8209$
0.058	$r_{K1}^{(i,j)} = -17.242 \cdot \left(S_{d1}^{(i,3)}\right)^2 + 14.872 \cdot S_{d1}^{(i,3)} + 0.7733$
0.079	$r_{K1}^{(i,j)} = -41.228 \cdot \left(S_{d1}^{(i,4)}\right)^2 + 15.206 \cdot S_{d1}^{(i,4)} + 0.7638$
0.103	$\boldsymbol{r_{K1}^{(i,j)} = 122.36 \cdot \left(S_{d1}^{(i,1)}\right)^2 - 4.8456 \cdot S_{d1}^{(i,1)} + 1.2406}$
0.163	$r_{K1}^{(i,j)} = 0.0385 \cdot \left(S_{d1}^{(i,5)}\right)^2 - 0.0999 \cdot S_{d1}^{(i,5)} + 1.0486$
0.198	$r_{K1}^{(i,j)} = -34.742 \cdot \left(S_{d1}^{(i,6)}\right)^2 + 9.5796 \cdot S_{d1}^{(i,6)} + 0.9173$

Eqn. 2.10 is the final empirical relation that gives the value of $r_{K1}^{(i,j)}$ as a function of $\omega^{(j)}$ and $S_{d1}^{(i,j)}$ uniquely. This formula is very simple to apply, because the only information required is the mechanical percentage of reinforcement, which is easy to evaluate, and the displacement spectrum of the seismic event, which can be simply obtained from the accelerogram record using programs available for free online like SeismoSignal (SeismoSoft, 2011). Once the value of $r_{K1}^{(i,j)}$ is obtained, the value of the stiffness $K_{SP-BELA,Dam}^{(i,j)}$ can be evaluated by using Eqn. 2.11, which is the general form of Eqn. 2.5, and, consequently, a good bilinear approximation of the capacity curve of the structure with section *j* damaged by the earthquake *i* can be obtained without performing any finite element analyses.

$$K_{SP-BELA,Dam}^{(i,j)} = \frac{K_{SP-BELA,Un}^{(j)}}{K_{PO,Un}^{(j)}} \cdot K_{PO,Dam}^{(i,j)} = K_{SP-BELA,Un}^{(j)} \cdot \frac{K_{PO,Dam}^{(i,j)}}{K_{PO,Un}^{(j)}} = \frac{K_{SP-BELA,Un}^{(j)}}{r_{K1}^{(i,j)}} \tag{2.11}$$

The final purpose of this work will be to allow a prediction of the behaviour of the damaged structure by knowing just the displacement spectrum which corresponds to the seismic event that will possibly take place in the region where the structure is located. Given this assumption, in order to verify the potential accuracy of Eqn. 2.10 when the displacement demand $S^{(i,j)}_{d1}$ is obtained from displacement spectra, the whole set of 6 sections has been analysed using 3 accelerograms belonging to a suite of records which are compatible with a target spectrum of a site of interest in Reggio Calabria (Italy), obtained using a free software (Iervolino *et al.*, 2010). For more information regarding the accelerograms, see Miglietta *et al.* (2012).

Each one of the six columns with different sections $\omega^{(j)}$, that have been considered until now, has been tested performing a nonlinear time-history analysis and a subsequent pushover analysis on the damaged structure. Each column has been subjected to the three available earthquakes previously mentioned. After that, the SP-BELA bilinear curve for the damaged section has been drawn following the procedure described above. Finally, Eqn. 2.10 has been applied to draw the simplified SP-BELA bilinear curve of the damaged structure (SP-BELA Approximated curve), while the SP-BELA Effective curve has been obtained using Eqn. 2.2. The error $\Delta K_{SP\text{-}BELA,\ Dam}$ (%) has been computed using Eqn. 2.12. Table 2.2 shows the results for one of the considered seismic events. For more results related to this and to the other accelerograms, see Miglietta *et al.*(2012).

$$\Delta K_{SP-BELA,Dam} = \frac{\left|K_{SP-BELA,Dam,Effective} - K_{SP-BELA,Dam,Approximated}\right|}{K_{SP-BELA,Dam,Effective}} \cdot 100 \tag{2.12}$$

From the results of these analyses it can be observed that Eqn. 2.10 gives good approximations of the initial elastic branch of the capacity curve corresponding to the damaged structure, even in the case of accelerograms which are displacement spectrum compatible. This result confirms that the simplified proposed formulation might be used with good results to predict the state of damage of a particular structure using the displacement spectrum of the national code.

Table 2.2. Elastic Stiffnesses Of The SP-BELA Bilinear Curves For The Seismic Event 000055xa

Section	ω	$K_{SP\text{-}BELA,\ Dam,\ Effective}$ $(1/m)$	$K_{SP\text{-}BELA,\ Dam,\ Approximated}$ $(1/m)$	$\Delta K_{SP\text{-}BELA,\ Dam}$ *(%)*
1	0.103	1.720	1.415	17.76
2	0.04	1.406	0.899	36.03
3	0.058	1.352	1.030	23.82
4	0.079	1.485	1.188	19.98
5	0.163	2.780	2.056	26.02
6	0.198	2.341	2.389	2.02
			Average Error	**21**

3. APPLICATION OF THE SIMPLIFIED PROCEDURE TO TWO RC FRAME BUILDINGS

The further step of this research is to apply the methodology, described in the previous paragraph, to realistic buildings. With this purpose, the simplified procedure has been applied to two prototype RC frame buildings. The validity of the procedure is then checked by performing nonlinear time history analyses followed by pushover analyses, in order to obtain the real residual capacity curve to be compared to the bilinear approximated one. All the modelling and all analyses have been performed using the software GEN 2011 (MIDAS Information Technology Co., 2010).

The first building analysed, from now on referred to as B1, is a 4 storeys 30×10 m structure with regular width of all the spans, while the second one, from now on referred to as B2, is a 6 storeys 24×12 m structure with irregular span width in the *X* direction. Both structures are non-seismically designed. In fact, while the *X* direction is provided with 3 frames, one for each span, the *Y* direction has only 2 external frames. Some frame effect in the internal spans is provided by the contribution of an equivalent width of the floor concrete slab on top of the joists. These buildings are representative of the majority of Italian structures that were built before the development and application of the seismic code. For this reason, they do not fulfil the requirements that today would be applied to new constructions. For further details on the two buildings, see Miglietta *et al.* (2012).

After static and modal analyses, both the structures have been tested in each direction with a displacement control pushover analysis and the original SP-BELA procedure has been applied. The SP-BELA bilinear curve, that approximated the capacity curve of the undamaged structures, is the starting point for the application of the simplified procedure. In fact, the next step consists of the application of the simplified procedure, which has been derived in Paragraph 2. Eqn. 2.10 is applied using as $\omega^{(j)}$ the average ω of the columns of the weak floor, which was the one with the smallest collapse multiplier λ, and as $S_{d1}^{(i,j)}$ the spectral displacement corresponding to the fundamental period of vibration of the building T_1. In this phase, the period T_1 should be the elastic one and thus it can be approximated using Eqn. 3.1. $K_{SP-BELA,Un}^{(i)}$ is the stiffness of the initial branch of the SP-BELA bilinear curve of the undamaged structure, computed as in Eqn. 3.2.

$$T_1 = 0.075 \cdot H_T^{3/4} \tag{3.1}$$

$$K_{SP-BELA,Un}^{(i)} = \frac{V_C}{\Delta_{LS1,Un}} \tag{3.2}$$

The displacement spectrum required to evaluate $S_{d1}^{(i,j)}$ corresponds to the ground motion time history applied to the building. The residual capacity curve can be obtained by a pushover analysis on the damaged structure at the end of the earthquake. In order to allow further developments of the procedure and facilitate its application with the use of code spectra, the input of the nonlinear analyses belongs to a suite of displacement spectrum compatible records (Iervolino *et al.*, 2010).

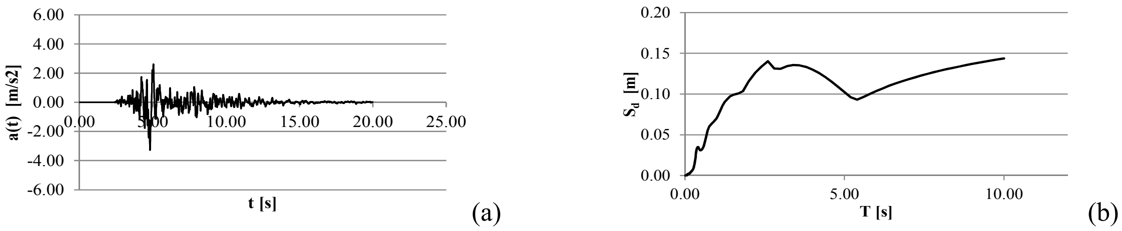

Figure 3.1. Event IN0409ya with PGA = 3.29 m/s^2 (a) time history; (b) displacement spectrum

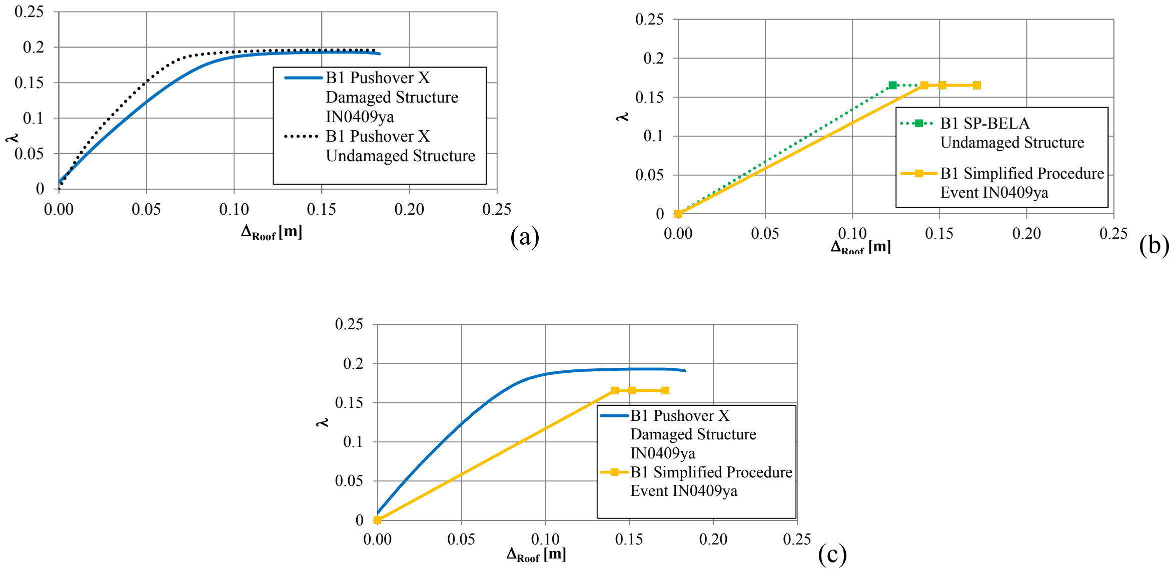

Figure 3.2. Structure B1, event IN0409ya: (a) pushover curves for undamaged and damaged building; (b) bilinear curves for undamaged (SP-BELA procedure) and damaged building (simplified procedure); (c) pushover and bilinear curves for damaged building

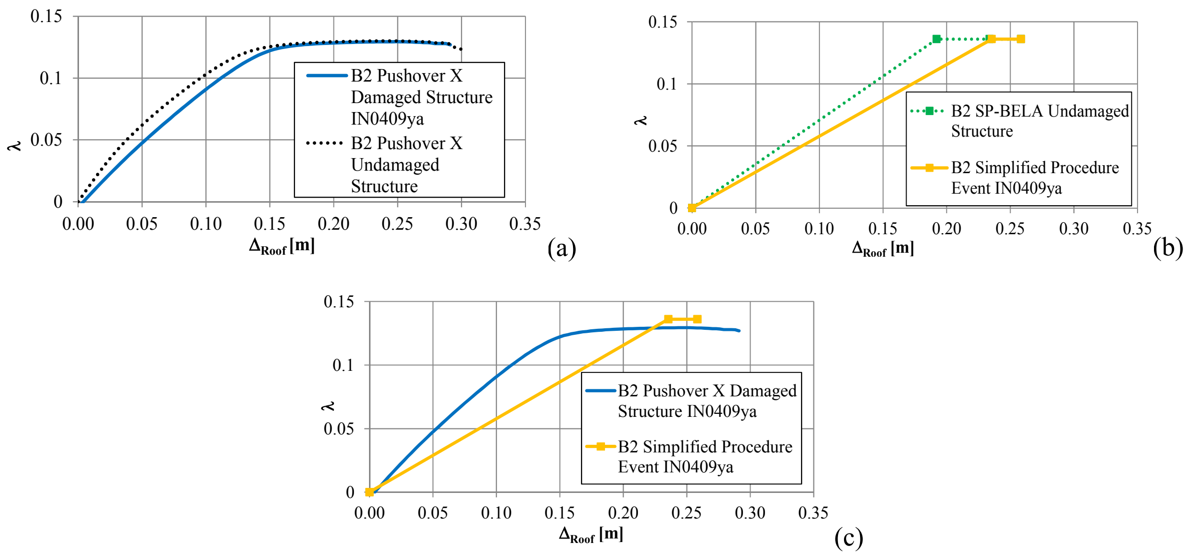

Figure 3.3. Structure B2, event IN0409ya: (a) pushover curves for undamaged and damaged building; (b) bilinear curves for undamaged (SP-BELA procedure) and damaged building (simplified procedure); (c) pushover and bilinear curves for damaged building

The procedure described above has been repeated several times for both B1 and B2 in both directions using different seismic inputs. Only one example in the *X* direction of both structures is presented in this paper (other results in Miglietta et al., 2012). The seismic input is the event IN0409ya, whose accelerogram and displacement spectra are represented in Figure 3.1.

Figure 3.2 shows the results of the applied methodology to the structure B1. In this example, the structure B1 enters its plastic range during the seismic event. Reaching the plastic state causes reduction in the capacity, as shown by Figure 3.2 (a), in which the residual capacity curve has a smaller initial stiffness and a shorter plastic plateau than the original capacity curve. The bilinear curves have different initial stiffnesses too, as displayed in Figure 3.2 (b). Figure 3.2 (c) shows that the simplified procedure gives a good approximation of the residual ductility of the structure B1. No significant residual displacement is observed in this case.

Figure 3.3 shows the results of the applied methodology to the structure B2. Structure B2 enters the plastic range too during Event IN0409ya. This is shown by Figure 3.3 (a), where there is a stiffness reduction in the damaged pushover. The same stiffness reduction is captured by the simplified method, as shown in Figure 3.3 (b) and Figure 3.3 (c).

4. USE OF THE SIMPLIFIED PROCEDURE TO ACCOUNT FOR PROGRESSIVE DAMAGE IN BUILDINGS B1 AND B2

Finding a way to apply the simplified procedure for evaluating the cumulative damage caused by a series of 2 seismic events is the object of this final part of the research. The simplified SP-BELA based bilinear curve, which should approximate the residual capacity curve at the end of the second event, has been built in accordance with the procedure described in Paragraph 3, but with the following differences:

1. The stiffness reduction coefficient $r_{K1,quad}^{(i,j)}$, relating to the i^{th} seismic event and to the j^{th} section, is not applied to $K_{SP-BELA,Un}^{(i)}$, that is the stiffness of the original undamaged building, but to $K_{SP-BELA,Dam}^{(i,j)}$, that is, instead, the stiffness of the bilinear curve of the structure damaged by the first seismic event;

2. To find the spectral displacement, the period $T_{1,dam}$ is used. It is computed through Eqn. 4.1, in which μ_{LS1} is the ductility calculated as in Eqn. 4.2 and T_1 is given by Eqn. 3.1.

$$T_{1,dam} = T_1 \cdot \sqrt{\mu_{LS1}} \tag{4.1}$$

$$\mu_{LS1} = \frac{\Delta_{LS1,dam}}{\Delta_{LS1}} \tag{4.2}$$

To check the validity of the simplified method in evaluating the accumulated damage coming from subsequent earthquakes, it is necessary to run a combination of two nonlinear time history analyses in series and, then, to apply a pushover analysis, which keeps the final damage at the end of each analysis as the initial condition for the following one. Several analyses of this kind have been carried out along both directions of each building. In the following, only one result in the *X* direction of both structures is presented. The seismic input corresponding to the first earthquake is the event IN0409ya, already described in Figure 3.1; the second input earthquake, in this example, is event IN0350ya, whose accelerogram and displacement spectra are shown in Figure 4.1.

Figure 4.2 shows the results of the applied methodology to the structure B1. The second seismic event causes a significant loss of B1 shear capacity, as it is shown in Figure 4.2 (a). This effect is not captured by the simplified procedure. However, it is capable to predict the further stiffness reduction due to the progressive damage, as it is possible to observe in Figure 4.2 (b).

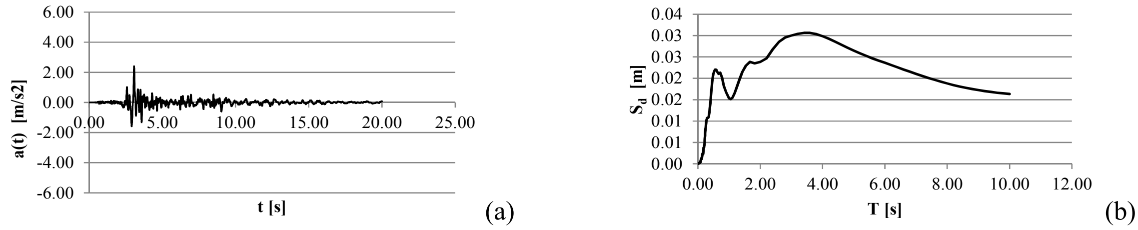

Figure 4.1. Event IN0350xa with PGA = 2.41 m/s^2 (a) time history; (b) displacement spectrum

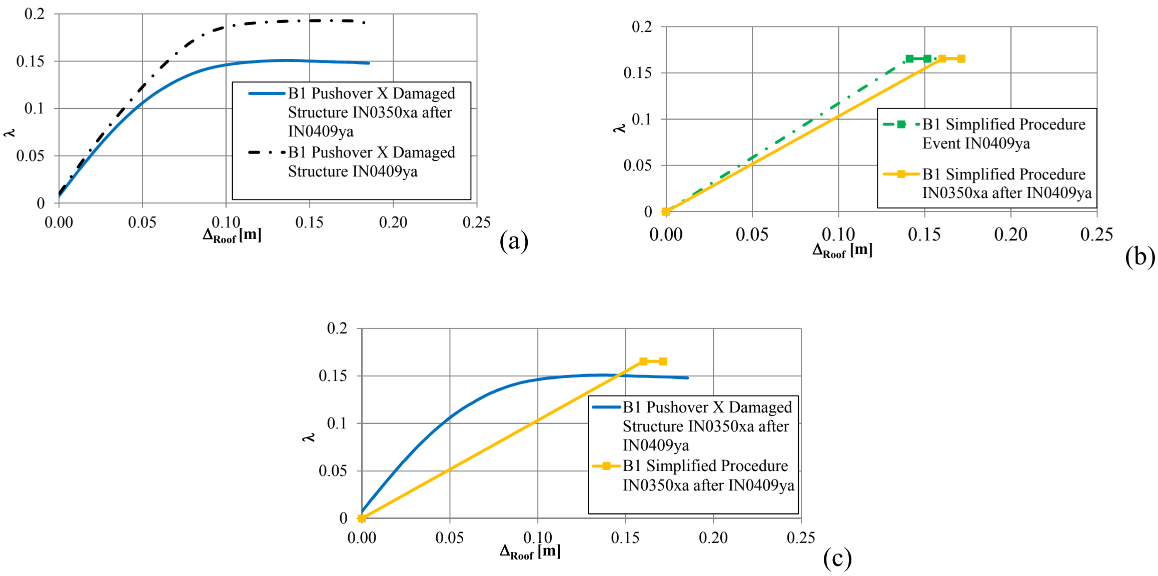

Figure 4.2. Structure B1, event IN0350xa applied after event IN0409ya: (a) pushover curves for singularly and progressively damaged building; (b) simplified procedure bilinear curves for singularly and progressively damaged building; (c) pushover and bilinear curves for progressively damaged building

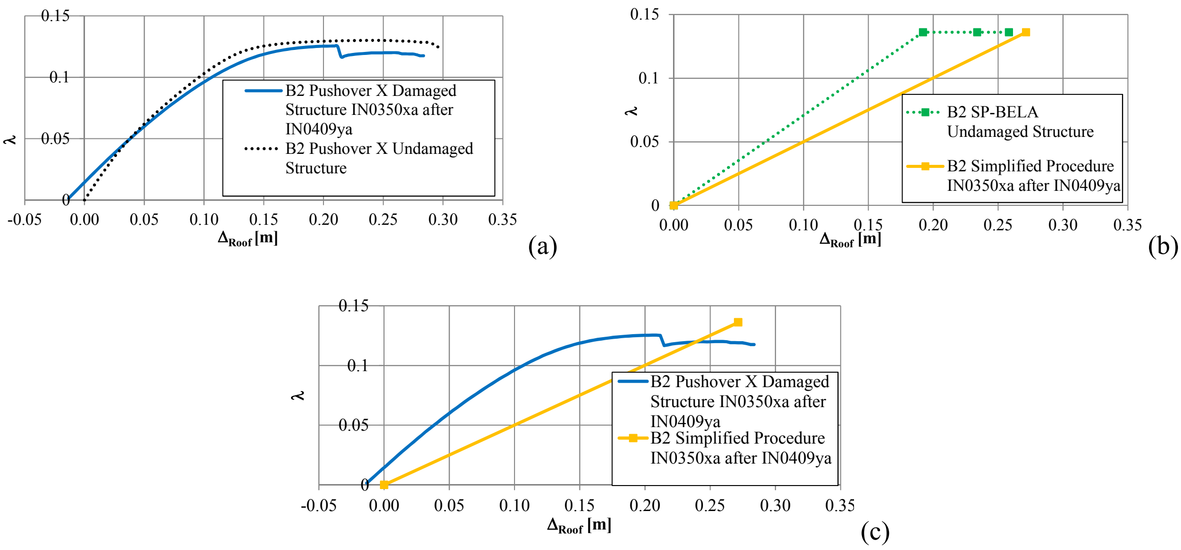

Figure 4.3. Structure B2, event IN0350xa applied after event IN0409ya: (a) pushover curves for singularly and progressively damaged building; (b) simplified procedure bilinear curves for singularly and progressively damaged building; (c) pushover and bilinear curves for progressively damaged building

Figure 4.3 shows the results of the applied methodology to the structure B2. In this case, the second event brings the structure B2 very close to collapse, as displayed by Figure 4.3 (a). The simplified procedure predicts this situation of high damage, as shown in Figure 4.3 (b) and Figure 4.3 (c). Moreover, in this example, B2 is affected by residual displacement caused by the progressive damage, but, in this phase of development, there is no way to account for it in the procedure.

5. CONCLUSIONS

In this study, the problems related to the assessment of the residual capacity of existing buildings after seismic events and to the influence of progressive damage have been discussed and analysed. Starting from the SP-BELA assessment method, a simple procedure that allows the evaluation of the residual capacity of a RC frame building after one (Paragraph 3) or two (Paragraph 4) seismic events has been derived and verified with nonlinear analyses. The proposed simplified procedure presents several advantages. It only requires the engineers to have basic knowledge about the structure; no modelling or time history analyses are required. Moreover, the fact that the procedure has been derived using spectrum-compatible time histories allows the users to have only a basic knowledge of the seismic event whose effects they are investigating. In this way, the modified SP-BELA method can be implemented in a spreadsheet and used to analyse many buildings quickly, providing an adequate degree of certainty about the safety of the entire building stock in a seismic zone. Like other simplified procedures, this method does not give a perfect bilinearization of the real capacity curve, but comparisons of the results achieved by this procedure with those from FEM analyses have shown that the proposed method provides a good approximation of the reduced stiffness for both the SDOF and the building models, even considering the influence of progressive damage.

Several further developments and improvements of the method are still required. First of all, the possible reduction in the total shear capacity at the end of a seismic event is not taken into account by this procedure. Moreover, only the light damage structural limit state displacement (Δ_{LS1}) has been modified herein; a modification of the other two limit state displacements accounting for the change in ductility due to the progressive damage could increase the accuracy of the results. Another improvement in the procedure would be to determine a way to calculate the residual displacement which can occur after a first seismic event, which is currently neglected. Finally, the entire procedure has validity only for reinforced concrete frame buildings. Further developments will be related to the application of the proposed methodology to irregular reinforced concrete structures, such as wall-framed buildings, or other structural typologies (masonry, precast and steel structures).

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