Earthquakes and Structures, Vol. 4, No. 2 (2013) 157-179
DOI: http://dx.doi.org/10.12989/eas.2013.4.2.157

Seismic demand estimation of RC frame buildings based on simplified and nonlinear dynamic analyses

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(Received February 3, 2010, Revised February 13, 2012, Accepted March 24, 2012)

Abstract. Vulnerability studies on the existing building stock require that a large number of buildings is analyzed to obtain statistically significant evaluations of the seismic performance. Therefore, analytical evaluation methods need to be based on simplified methodologies of analysis which can afford the treatment of a large building population with a reasonable computational effort. Simplified Pushover-Based Earthquake Loss Assessment approach (SP-BELA), where a simplified methodology to identify the structural capacity of the building through the definition of a pushover curve is adopted, was developed on these bases. Main objective of the research work presented in this paper is to validate the simplified methodology implemented in SP-BELA against the results of more sophisticated nonlinear dynamic analyses (NLDAs). The comparison is performed for RC buildings designed only to vertical loads, representative of the "as built" in Italy and in Mediterranean countries with a building stock very similar to the Italian one. In NLDAs the non linear and degrading behaviour, typical of the structures under consideration when subjected to high seismic loads, is evaluated using models able to capture, with adequate accuracy, the non linear behaviour of RC structural elements taking into account stiffness degradation, strength deterioration, and pinching effect. Results show when simplified analyses are in good agreement with NLDAs. As a consequence, unsatisfactory results from simplified analysis are pointed out to address their current applicability limits.

Keywords: vulnerability; existing buildings; reinforced concrete; nonlinear dynamic analyses; simplified methods

1. Introduction

A Simplified Pushover-Based Earthquake Loss Assessment (SP-BELA) method has been developed for different structural types as widely documented in Borzi *et al*. (2008a, 2008b) and Bolognini *et al*. (2008) for RC cast in place buildings, masonry buildings and RC pre-cast buildings, respectively. SP-BELA combines:

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- the definition of a pushover curve using a simplified mechanics-based procedure, which is for RC building, similar to the one proposed by Cosenza *et al*. (2005) and Iervolino *et al*. (2006) to define the base shear capacity of the building stock;
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- a displacement-based framework, such that the vulnerability of building classes at different limit states can be obtained comparing the displacement capacity corresponding to the aforementioned limit conditions with the displacement demand.

The SP-BELA procedure uses Monte Carlo simulation to generate a random population of

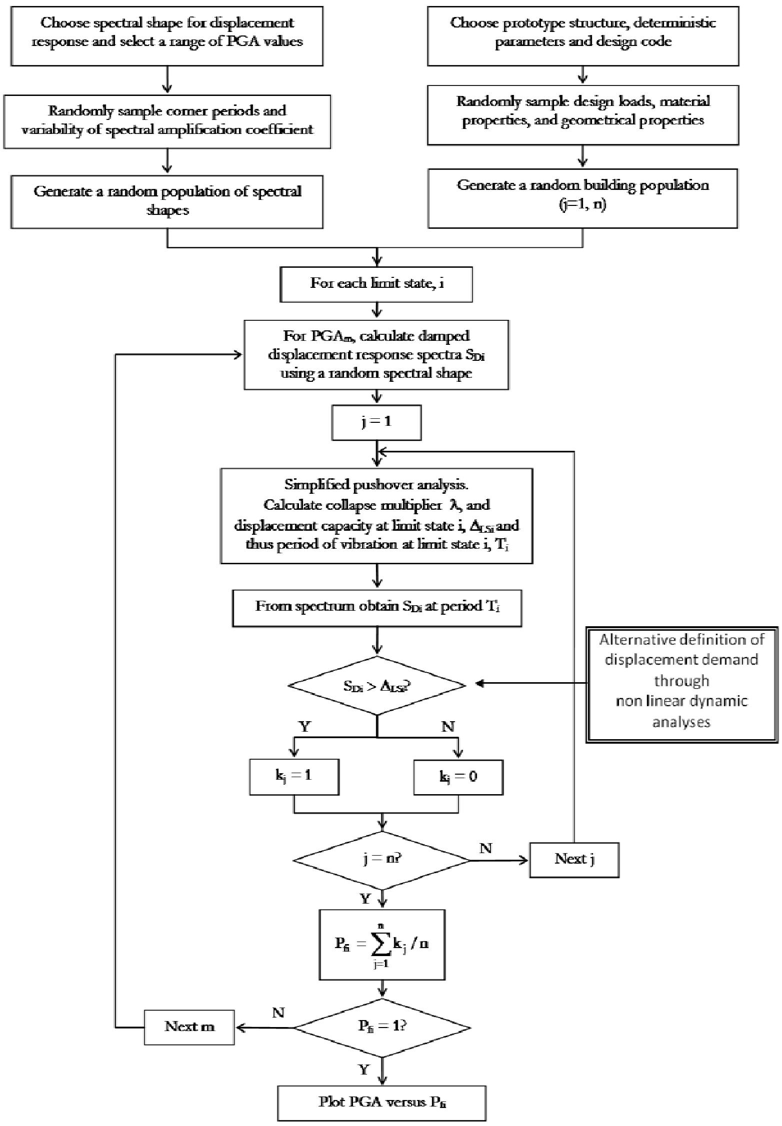

Fig. 1 Flow chart of SP-BELA vulnerability methodology

buildings. In order to have a statistically significant population, a sample size of several hundred buildings must be generated. Therefore, each building cannot be studied through sophisticated nonlinear analysis and simplified methodologies of analysis need to be taken into account.

The main component of the methodology (see Fig. 1) involves the definition of the capacity of a population of buildings based on a prototype structure. The building capacity is worked out using simplified pushover analysis. Extensive validation of simplified methodologies to derive pushover curves has been carried out as documented in Borzi *et al.* (2008a, 2008b) and Bolognini *et al.* (2008) for RC cast in place buildings, masonry buildings and RC pre-cast buildings, respectively.

The demand in SP-BELA is modelled using a displacement response spectrum. The magnitude of displacement spectral ordinates is obtained by anchoring a spectral function adimensionalised on PGA, the parameter that has been assumed as representative of the ground shaking severity. The PGA is then increased incrementally in order to define all the points of the vulnerability curves. Alternatively, as a further development in SP-BELA, the demand could be defined through the peak of displacement of the building when subjected to a certain dynamic input. The peak of displacement cannot be obtained through dynamic analysis of the original structure, because the computational effort required is unaffordable. Hence, an alternative procedure needs to be set up. In this paper, an equivalent SDOF system is defined on the basis of the seismic performance worked out through the pushover analysis. Therefore, the nonlinear analyses can be performed on the SDOF system instead of the original structure.

In this paper the capability of the equivalent SDOF system to capture some demand parameters influencing the seismic performance of the original structure is evaluated. Such target is pursued comparing the results of the simplified methodology of analysis (i.e., pushover and dynamic on the equivalent SDOF system) with results of proper nonlinear dynamic analyses on structures. RC cast in place bare frame none seismically designed buildings are here taken into account. Main goal of this paper is to verify the capability of the equivalent SDOF system, whose characteristics are defined through the pushover curve, in estimating the seismic demands of the original structure with reference to more accurate NLDAs.

Section 2 describes the structures under examination and provides the main aspects of the procedure based on the NLDAs. Section 3 briefly summarises the simplified assessment methodology. The results of the comparison between sophisticated and simplified methodologies of analysis are summarised in Section 4.

2. Non linear dynamic analyses

The seismic performances of RC frame buildings are evaluated through Non Linear Dynamic Analyses (NLDAs) in the research work of Masi (2003) using a purposely set-up procedure. Structures carefully designed, taking into account only vertical loads, on the basis of the codes in force, of the available handbooks and of the current practice of the period (simulated design) are analysed. Beyond this work, investigations on the Italian construction standards before and after the 1971 have been undertaken in order to design buildings that can be considered representative of the “as built” in Italy and in Mediterranean countries with a building stock very similar to the Italian one. It is worth noting that in Italy 1971 was a key year for construction engineering, as a new code for RC buildings was in effect after this year where more effective and detailed criteria, compared to the previously in force code, were provided for structural design and execution. Particularly, material characteristics underwent remarkable changes in the two periods. As for pre-71 structures, low strength concrete and smooth steel bars were typically used; while after 1971 higher strength concrete and deformed steel have been increasingly used. A large DataBase of experimental data relevant to existing RC buildings constructed in different periods, examined in (Masi and Vona 2009), shows significant differences in terms of concrete strength, as it is taken into account in the present study. On the contrary, there was no evidence of differences concerning other mechanical properties, therefore an average value of cu equal to 0.005 has been considered, irrespectively of the building age. However, some differences have been assumed with respect to the degrading behavior, as explained in the following.

The proposed model took into account plane frames with two bays and a number of storey equal to 2, 4 and 8 having a constant interstorey height of 3 m. Such frames correspond to a regular plan lay out of 5 m by 5 m and are representative of the most flexible direction of typical buildings. Very often non seismically designed buildings present proper frames only in one direction (typically the longitudinal, longer building direction). In the other "weak" direction, the one investigated in the research works of Masi and Vona (2004), the frame effect is due to the possible presence of exterior frames while is internally guarpreed only by the contribution of the floor slabs spanning between the columns (No Beam, NB).

As for the "weak" direction is concerned, the typical characteristics of the Italian as well European building stock show that, due to the presence of masonry infill walls, it is very common to find edge beams spanning between the columns of the two exterior frames. The edge beams can have different stiffness factor since both conditions of beams within the floor slab thickness to find edge beams spanning between the columns of the two exterior frames. The edge beams can have different stiffness factor since both conditions of beams within the floor slab thickness (Flexible Beam, FB, 70 x 22 centimetres) and emergent beams (Rigid Beam, RB, 30 x 50 centimetres) are very common in the construction standards. The cases of buildings having small (Cases 1 and 2, 15 x 10 meters) and large (Cases 3 and 4, 25 x 10 meters) plan area, that is made up of 4 and 6 frames, have been analysed. The analyses undertaken on bare frame buildings, i.e., buildings where the infill contribution to the strength and stiffness of the structure can be neglected, have been selected. This choice is consistent with the main objective, of the work, that is performing a first comparison between SP-BELA and NLDAs. Bearing in mind such a goal, authors have chosen the simplest case, where possible differences in terms of results between SP-BELA and NLDAs can more easily highlighted.

Finally, 2, 4 and 8 storey frames have been analysed, representative of low-, mid- and high-rise buildings. Fig. 2 shows all the structural types considered in the paper. The configurations studied are summarised in Fig. 2.

The material properties considered for buildings built before and after the 1971 are documented in Vona and Masi (2004) and in Masi and Vona (2004), respectively, and summarized in Table 1. In the work a macro-modelling based on lumped plasticity has been adopted using the computer program IDARC-2D (Valles *et al.* 1996). Non linear and degrading behaviour, typical of the structures under consideration when subjected to high seismic loads, has been evaluated using the three parameter hysteretic Park model (Park *et al.* 1987). This model, based on a tri-linear monotonic envelope, is able to capture with adequate accuracy the non linear behaviour of RC structural elements taking into account stiffness degradation, strength deterioration and pinching effect. It has been widely tested with reference to the behavior of damaged buildings observed after seismic events, Park *et al.* (1987). The values of the degrading parameters were adopted on the basis of the work of Ghobarah *et al.* (1999) and on the experimental results obtained by

Table 1 Properties of concrete and steel assumed for post-1971 and pre-1971 RC buildings

Buildings built before 1971	Concrete C 10/12	Cubic characteristic resistance	R_{ck}=12 MPa
		Cylindrical characteristic resistance	f_{ck}=10 MPa
		Cylindrical average resistance	f_{cm}=16 MPa
		Ultimate deformation	ω_{cu}=0.5 %
	Steel Type Aq42	Characteristic yielding resistance	f_{yk}=280 MPa
		Average yielding resistance	f_{ym}=250 MPa
		Ultimate deformation	ω_{su}=2 %
Buildings built after 1971	Concrete C 20/25	Cubic characteristic resistance	R_{ck}=25 MPa
		Cylindrical characteristic resistance	f_{ck}=20 MPa
		Cylindrical average resistance	f_{cm}=(f_{ck}+8)=28 MPa
		Ultimate deformation	ω_{cu}=0.5%
	Steel Type A38 (now Feb 38K)	Characteristic yielding resistance	f_{yk}=380 MPa
		Average yielding resistance	f_{ym}=400 MPa
		Ultimate deformation	f_{su}=2 %

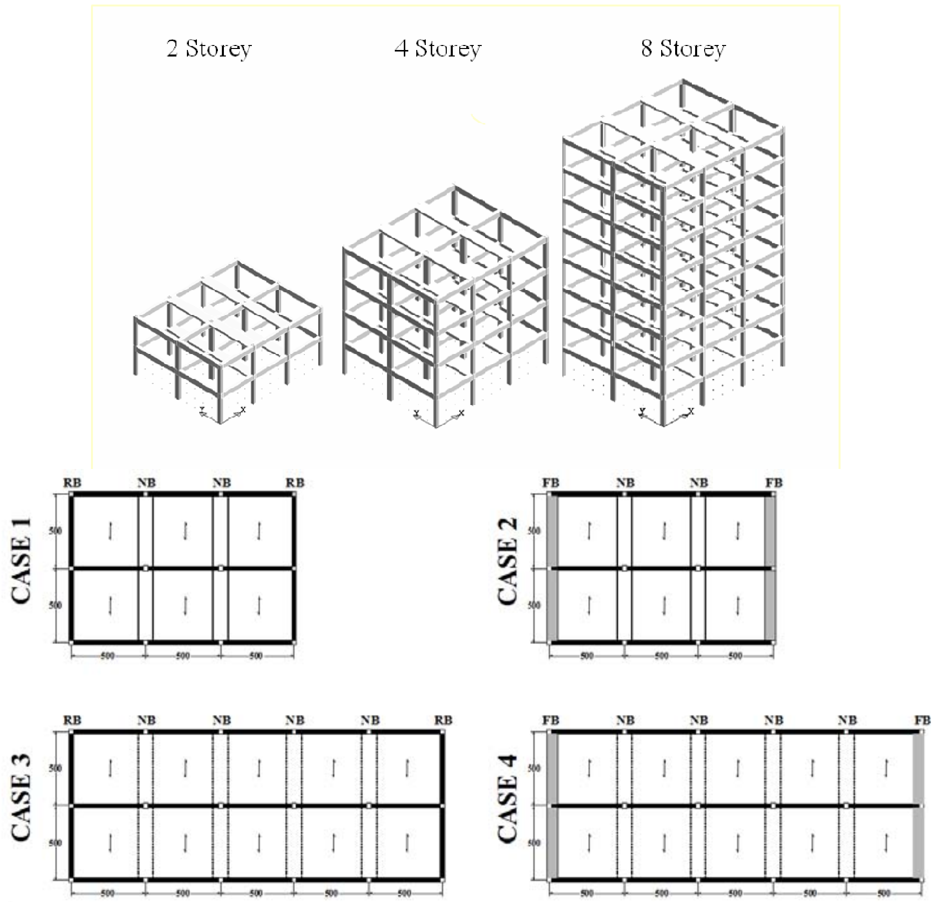

Fig. 2 Outline of case studies considered in the study

Table 2 Adopted values of degrading parameters for Ante71 RC buildings

	Stiffness degradation (α)	Strength deterioration (β)	Pinching effect (γ)
Beams (internal joints)	1.5	0.15	0.6
Beams (external joints)	1.5	0.15	0.7
Internal Columns	1	0.15	0.6
External Columns	1	0.15	0.4

Table 3 Adopted values of degrading parameters for Post71 RC buildings

	Stiffness degradation (α)	Strength deterioration (β)	Pinching effect (γ)
Beams	2	0.1	0.7
Columns	1.5	0.1	0.7

Kunnath *et al.* (1995a, 1995b), Liu and Park (2000), Pampanin *et al.* (2002), Masi *et al.* (2009) on sub-assemblages having typical details of gravity load designed buildings, as well as on a consideration of the characteristics of the structures under examination. Considering the differences in terms of reinforcement details, the values of the degrading parameters for the Ante 71 and Post 71 structures are reported in Tables 2 and 3, respectively. The moment-rotation characteristic of the plastic hinge is obtained from the moment-curvature multiplied by the plastic hinge length calculated according to CEB 240 (1998).

Recent literature (e.g. Kwon and Elnashai (2006), Nanos and Elenas (2006)) points out the crucial role of seismic input for a correct evaluation of structure response. Taking into account the prominent role of the seismic input on the structural non linear response, the accelerogram set was carefully selected according to the procedure described in Masi *et al.* (2011). Subset of accelerograms are made up of real accelerograms selected from the European Strong-Motion Database, Ambraseys *et al.* (2004). The first random selection was modified excluding accelerograms with known problems. Finally, thirty-one natural accelerograms with a PGA level ranging from 0 e 0.5 g have been selected to reproduce the input ground motion at the frame foundations.

The proposed methodology has been completely applied on post-71 RC buildings while, regarding pre-71 RC buildings, only the 4 storey type has been presently analyzed.

3. Simplified methodology of analysis

To assess the seismic vulnerability at urban scale, simplified methodologies of analysis need to be selected. A two step analysis is undertaken. As first step a simplified pushover analysis is performed. Such methodology is implemented in SP-BELA (Borzi *et al.* 2008a). The results of the aforementioned analysis are then used to define the parameter of an equivalent SDOF system, which is corresponding to the original structure in terms of period of vibration, displacement capacity and quantity of dissipated energy. Hysteretic rules are defined for loading and unloading branches such as the dynamic analysis is performed on the equivalent SDOF system instead of the original multi degree of freedom structure. In the following details on the simplified pushover and dynamic analysis are given.

3.1 Simplified pushover analysis

In the proposed simplified method an elastic-perfectly-plastic behaviour is assumed. This effectively means that in order to define the pushover curve, only the collapse multiplier λ (corresponding to the ratio between base shear force and seismic weight) and the displacement capacity need to be defined (see Fig. 3).

The pushover analysis has been performed for horizontal forces linearly distributed along the height since for the building population taken into account the first vibration mode is almost linear. However, different distributions may be easily assumed when relevant (e.g. for taller buildings where the effects of higher modes become important). The procedure, which takes inspiration from the work of Priestley and Calvi (1991), then calculates, for each column of the frame, the maximum value of shear that the column can withstand as the smallest of:

- The shear capacity of the column;

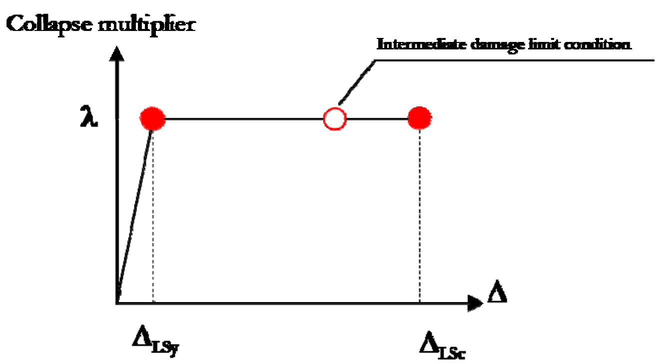

Fig. 3 Capacity curve for elastic perfectly plastic structural behaviour

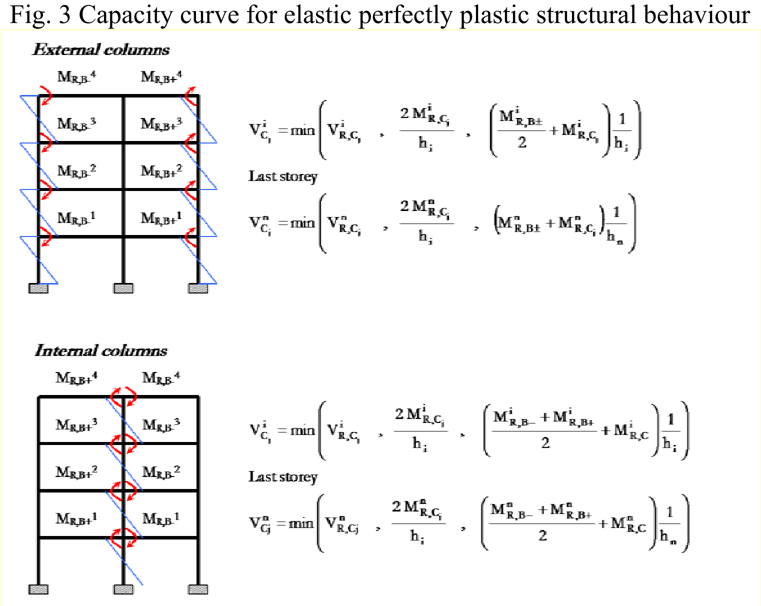

Fig. 4 Maximum shear force that the columns in a frame can withstand accounting for (1) shear and flexural failure mechanism in columns and (2) flexural failure mechanism in beams

- The shear corresponding to the flexural capacity of the column;
- The shear corresponding to the flexural capacity of the beams supported by the column.

For the beams only the flexural collapse mechanism is taken into account, given that the beams tend to be less prone to shear failure than the columns since gravity load design typically features high shear forces in the beams. These elements have thus traditionally been provided with an adequate amount of shear reinforcement. Furthermore, since the aim of the simplified analysis is to define the global seismic performance, the beam capacity is needed only to define the internal actions that the beams transfer to the columns. Hence, even if sometimes a beam may collapse for a mechanism that is different from the flexural one, this will lead to neglect local collapse failure mechanism and to overestimate the internal action that the beam transfers to the column considering that the beam can develop all its flexural capacity without having the interference of other failure mechanisms. These assumptions are considered to be acceptable in a simplified analysis methodology, which is aimed to describe the behaviour of a building stock.

The checks conducted during the procedure to define the cause of failure in each column are illustrated in Fig. 4, wherein the subscript R is for resistance and the subscripts C and B represent column and beam, respectively.

If the beam opens a plastic hinge before the columns, it is assumed that plastic hinges form at the base of the columns, as can be gathered from the equations in Fig. 4. This is due to the fact that a mechanism can develop only when plastic hinges are activated in all columns at the same level. The equilibrium at the beam-column joints in the case of weak beams is shown in Fig. 5.

Once the shear capacity has been calculated for every storey, the collapse multiplier is defined by the following relationship

$$\lambda^i = \frac{V_C^i}{W_T} \frac{\sum_{j=1}^{n} W_j z_j}{\sum_{k=i}^{n} W_k z_k} \tag{1}$$

where W_T is the global building weight, W_i is the weight associated to floor i located at height z_i. The final collapse multiplier used to define the capacity curve will be the smallest λ_i.

Finally, in order to evaluate the collapse mechanism of the building the procedure uses the following criteria:

- If there is a shear failure mechanism detected in at least one column, the capacity curve will be interrupted at the lateral force that produces this failure. This choice is consistent with the fact that the shear failure mechanism is brittle and does not have associated dissipative capacity. Therefore, the structure cannot enter the nonlinear range;
- If all the columns within a certain storey activate a plastic hinge, then a column-sway collapse mechanism will be activated (see Fig. 6a);
- If after the development of plastic hinges in all beams above a certain floor, plastic hinges form in all columns at the aforementioned floor, a beam-sway collapse mechanism will be activated (see Fig. 6b for a beam sway mechanism that open at the ground level).

There could be a situation in which at the storey corresponding to the smallest λ_i some of the columns are stronger then the beams, or vice versa. Therefore, it cannot be clearly identified whether a beam or a column-sway mechanism will be activated.

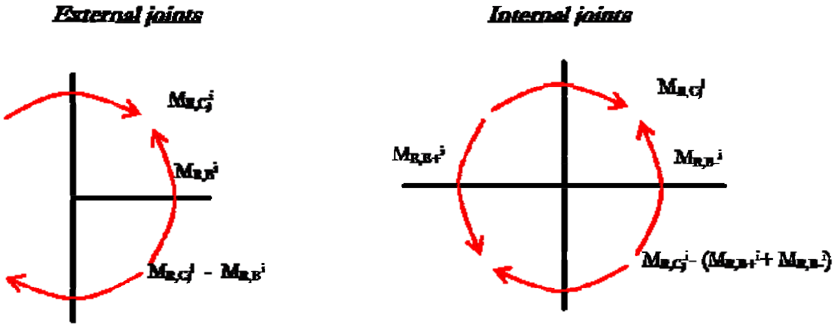

Fig. 5 Equilibrium at the joint in the case of weak beams

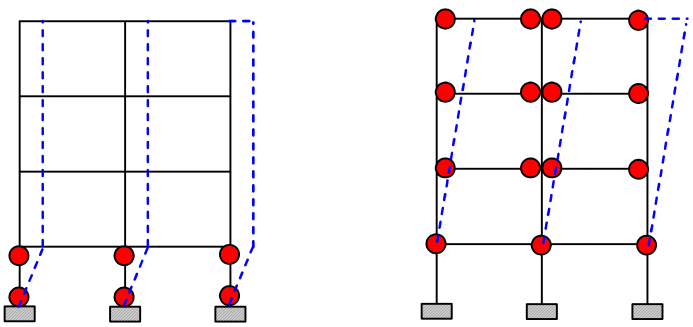

Fig. 6 Possible collapse mechanisms for a frame: (a) column-sway collapse mechanism and (b) beam-sway collapse mechanism

On the pushover curve the displacement capacity corresponding to yielding and collapse should be defined. The displacement capacity is the displacement at the building height corresponding to the position of centre of mass, being defined on the basis of limit conditions and deformed shape associated to the failure mechanism. In the proposed methodology the limit conditions are given in terms of chord rotations that, for columns, correspond to the interstorey drift.

In order to compare the results with IDARC-2D models of the frames analysed by Masi and Vona (2004), the relationships which leads to the yield rotation capacity have been modified with respect to the ones originally implemented in SP-BELA Borzi *et al.* (2008a). In SP-BELA for yield and collapse limit condition the rotation capacity is limited by the chord rotation such as proposed by Panagiotakos and Fardis (2001). For the yield curvature the following relationship is considered

$$\phi_y = \frac{M_{res}}{E_c I_e} \tag{1}$$

Where Mres is the resisting moment of the section, E_c is the Young modulus of concrete and I_e is the effective stiffness of the RC cracked section. The chord rotation is then calculated as

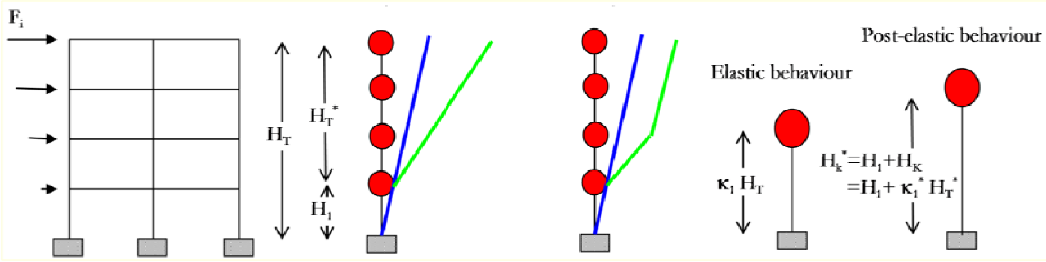

Fig. 7 Deformed shape for (left) beam-sway and (right) column-sway collapse mechanisms activated above the first floor. The black line represents the elastic deformed shape and the grey line the post-yield mechanism

$$\Theta_y = \phi_y \frac{L_V}{3} \tag{2}$$

Where L_V is the shear span (equal to the ratio between bending moment and shear). For columns, a double bending distribution is commonly assumed, and hence L_V is half the interstorey height.

Finally, to define the displacement capacity on the pushover curve corresponding to the interstorey drift, the height of an equivalent SDOF system has to be evaluated. According to Priestley *et al.* (2007), a coefficient κ_1 to be applied to the total building height is introduced

$$\begin{aligned} &\kappa_1 = 0{,}64 && \text{for n} \le 4 \\ &\kappa_1 = 0{,}64 - 0{,}0125(n-4) && \text{for } 4 < \text{n} < 20 \\ &\kappa_1 = 0{,}44 && \text{for n} \ge 20 \end{aligned} \tag{3}$$

where n is the number of storeys of the building. Although the equations above refer to the global collapse mechanism activated at foundation level, an intensive validation exercise (Borzi 2006) has been undertaken and the outcome is that such equations are adequate also for other type of failure mechanisms.

A linear deformed shape is assumed within the elastic range. Therefore, the displacement capacity associated to the yielding point, is given by

$$\Delta_{LSy} = \kappa_1 \, H_T \, \Theta_y \tag{5}$$

where H_T is the global building height.

In the post-elastic range the deformed shape is assumed as shown in Fig. 7 for beam-sway and column-sway mechanisms. When the beam-sway collapse-mechanism is activated, the procedure accounts for the centre of mass moving up towards the centre of mass of the building part that is involved in the collapse mechanism. Hence, Eqs. (6) and (7) define the displacement capacity for beam and column-sway failure mechanisms, respectively

$$\Delta_{LSc} = \Delta_{LSy} \frac{H_k^*}{\kappa_1 H_T} + \left(\Theta_u - \Theta_y\right) H_k \tag{4}$$

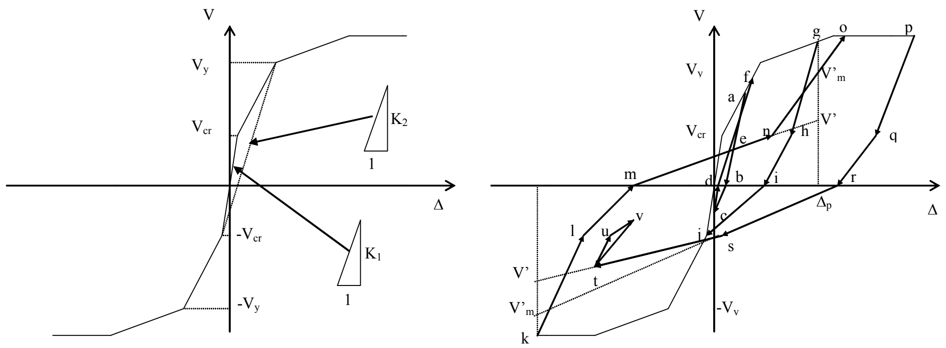

Fig. 8 HHS model for structural members

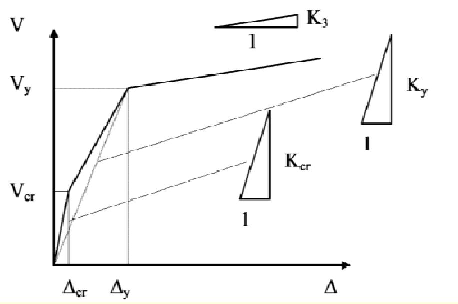

Fig. 9 Shape of primary curve used in this work

$$\Delta_{LSc} = \Delta_{LSy} + \left(\Theta_u - \Theta_y\right)h_p \tag{7}$$

where h_p is the interstorey height and H_k is the equivalent height of the part of the building above the activation of the global collapse failure mechanism and H_k^* is H_k plus the height of activation of the mechanism.

Further detail on the pushover methodology for RC buildings implemented in SP-BELA can be found in Borzi *et al.* (2008a).

3.2 Simplified dynamic analysis

The dynamic analysis is performed by employing a hysteretic hardening-softening model (HHS). The structural model is characterised by the definition of a primary curve and unloading and reloading rules. The primary curve for a hysteretic force-displacement relationship is defined as the envelope curve under cyclic loads. For non-degrading models the primary curve is considered as the response curve under monotonic load, i.e., the pushover curve. On the primary curve two points have to be defined as cracking and yield loads (V_{cr} and V_y) and the corresponding displacements (Δ_{cr} and Δ_y) as shown in Fig. 8. If this model is used to describe the hysteretic behaviour of RC buildings, the cracking load would correspond to the spreading of cracks in the concrete and the yielding load would be the load at which the mechanism is activated. Unloading and reloading branches of the HHS model have been established through a statistical analysis of experimental data. A comprehensive experimental investigation was conducted for this purpose by Saatcioglu *et al.* (1988) and Saatcioglu and Ozcebe (1989).

The input parameters for the HHS model described above is the pushover curve. However, the HHS model adopts a three linear branches primary curve as shown in Fig. 9. Consequently, the input parameters defining the shape of the primary curve are:
- The relationship between the cracking and the yielding load (V_{cr}/V_y);
- The relationship between the stiffness before the cracking load and the secant stiffness (K_{cr}/K_y);
- The slope of the post yield branch.

The first two relationships allow to define the bilinear branches that describe the elastic behaviour on the primary curve, starting from the elastic perfectly plastic assumption undertaken to calculate the pushover curve. An elastic-perfectly-plastic behaviour is taken into account. Therefore, the slope of the post yield branch is null. From the experimental results of Paulay and Priestley (1992), Calvi and Pinto (1996) and Pinto (1996), it is reasonable to consider a secant stiffness value at the yield point in the range between 40% and 50% of the stiffness before V_{cr}. V_{cr} is considered to be between 3, 4 times smaller then V_y since the ratio between the cracking and the yield load influences the pinching, phenomenon that does not often occur for structures with loads higher then approximately 30% of the yielding load V_y.

The initial load follows the primary curve until unloading starts. Loading and unloading follows the primary curve if the force has not exceeded the cracking load in both directions. When the cracking load is exceeded during cyclic deformations the slope of unloading and reloading branches was defined on the basis of experimental observations, as explained below, Saatcioglu *et al.* (1988), Saatcioglu and Ozcebe (1989). The rules defining the branches of the HHS model are expressed in terms of selected parameters, whose effect on the response was observed to be significant. These parameters include:
- Displacement ductility ratio;
- Number of cycles at a given deformation level;
- Magnitude of axial load.

Two slope of the primary curve are used to define the unloading branches under cyclic loads. These are:
- The slope of the line connecting the origin to the crack point $K1$ (Fig. 8);
- The slope of the line connecting the yield point and the cracking point in the opposite quadrant $K2$ (Fig. 8).

The unloading slope depends on deformation and force levels attained at the beginning of unloading. Experimental results indicated that if unloading starts between the cracking and the yield load, and the yield load has not been exceeded in the quadrant of the unloading, then the unloading stiffness is enclosed by K_1 and K_2. In this model a linear variation between these limits was proposed as a function of displacement ductility. If the unloading load exceeds the yield load, the unloading curve changes the slope to a value close to the cracking load. The rules proposed for the model are listed below:

(1) If V_{cr} has been exceeded at least once in one direction, and the yield load V_y has not been previously exceeded in the quadrant where the unloading is taking place, unloading follows a straight line up to the zero load axis. The slope of this line is given by

$$K = K_1 - \frac{K_1 - K_2}{\Delta_y - \Delta_{cr}} \left(\Delta - \Delta_{cr}\right)$$

for a load higher then V_{cr} (5)

$K = K1$ for a load lower then V_{cr} (6)

Where

$$K_1 = \frac{V_{cr}}{\Delta_{cr}} \; ; \; K_2 = \frac{V_{cr} + V_y}{\Delta_{cr} + \Delta_y}$$

and Δ is the displacement in which the unloading starts;
(2) If V_y has been exceeded at least once in the quadrant where the unloading occurs, the slope of response changes when the cracking load is reached. The two slopes that define this behaviour are

$$K = K_2 \left(1 - 0.05 \frac{\Delta}{\Delta_y}\right)$$

for loads higher then V_{cr} (7)

$$K = 0.6\, K_2 \left(1 - 0.07 \frac{\Delta}{\Delta_y}\right)$$

for loads lower then V_{cr} (8)

Structural members show stiffness degradation under cyclic loading. When the number of cycles or the magnitude of inelastic deformation increases, the system becomes softer. Furthermore, the hysteretic behaviour is affected by pinching. The latter is connected to sliding on the cracked surface, formed during the previous load cycles and to the deformation required to close previously-opened cracks. The axial load is an important parameter in predicting pinching effects (due to the onset of crack closure). The slope of reloading branches increases beyond the crack load. The rules that describe loading and reloading behaviour for HHS model are:
(1) If the member has not been loaded beyond the cracking load in one direction, the initial load in that direction points at the cracking load even if the member was loaded to the cracking load in the opposite direction;
(2) If V_{cr} was exceeded in the direction of loading then:
- reloading up to V_{cr} will follow a straight line passing through point (Δ_p, $\overline{V}_p$);
- reloading beyond V_{cr} will follow a straight line passing though point (Δ_m, $\overline{V}_m$);
- beyond the intersection of the reloading branch with the primary curve, loading follows the primary curve;

where

$$\overline{V}_p = V_p \exp\left(\alpha \frac{\Delta_p}{\Delta_y}\right) \tag{9}$$

$$\alpha = 0.82 \left(\frac{N}{N_0}\right) - 0.14 \le 0 \tag{10}$$

$$\overline{V}_m = V_m \exp\left(\beta\, n + \gamma \frac{\Delta_m}{\Delta_y}\right) \tag{11}$$

$$\beta = -\,0.014 \sqrt{\frac{\Delta_m}{\Delta_y}} \tag{12}$$

$$\gamma = -\,0.010 \sqrt{n} \tag{13}$$

where Δp is the previous peak displacement, Vp is the previous peak load, Δy is the yield displacement, Δm is the maximum displacement and V_m is the shear force on primary curve corresponding to the maximum displacement, all in the direction of the load. N is the axial compressive force and $N0$ the nominal concentric axial compressive capacity based on ACI 318-83 (American Concrete Institute 1983). Considering the expression for α above, with this model the results obtained for axial load higher then 20%N_0 are the same. This is because in this formulation the second order effects have not been considered and the axial load has an influence only on pinching. For the range of axial loads on the analysed structures a very marginal influence of the axial load itself has been detected. Therefore, a constant axial load equal to 10% of the nominal axial load is assumed.

The parameter n is a counter of the number of cycles in one direction at the current maximum displacement Δm. Upon the first unloading for the current maximum deflection n is 1. The value of n is incremented by 1 every time unloading occurs for a displacement in the range $\Delta m \pm \Delta cr$. If unloading occurs for a displacement greater then the current maximum displacement, Δm is updated and n is initialized to 1;

(1) If the unloading is completed prior to reaching the zero load axes, reloading in the same quadrant will trace a straight line pointing at the immediately preceding loading point (Fig. 8).

Further details on the HHS model here used to describe the hysteretic behaviour of the equivalent structure are given in Borzi *et al.* (2000a, 2000b, 2000c).

4. Result comparison

In the Figs. 10-17 the results provided by the simplified methodology of analysis (SA) and the non linear dynamic analyses (NLDAs) methodology, for frames designed according to standards before and after the 1971, are shown and compared. The results are presented in terms of peak values of base shear force and displacement at the centre of mass of the structure.

Main objective of the present work is to validate the simplified methodology implemented in SP-BELA in terms of global seismic demand against the results of the more accurate nonlinear dynamic analyses. However, in Figs. 10 and 11 the results in terms of peak value of base shear obtained for the 4 storey frame built according to standards after 1971 have been firstly compared considering two different seismic input parameters, that is Peak Ground Acceleration (a_g) and Housner Intensity (I_H), respectively. Housner Intensity (Housner 1952), I_H, has been computed as the value of the area under the pseudovelocity spectrum in the range of period 0.1 and 2.5 seconds, as shown in Eq. (17)

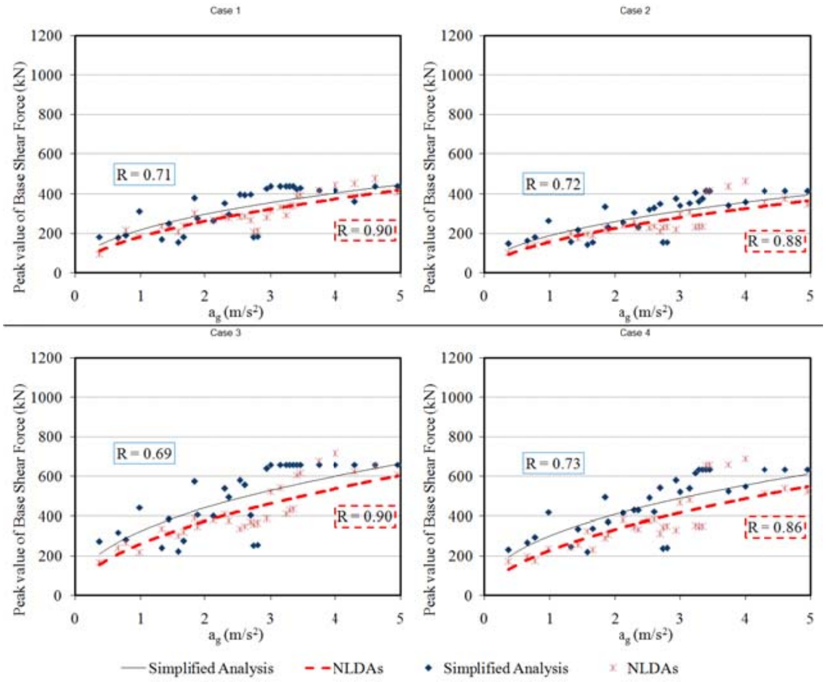

Fig. 10 Comparison in terms of peak value of base shear force for 4 storey buildings built according to standards after 1971 considering Peak Ground Acceleration (*R*, coefficient correlation of non linear regression)

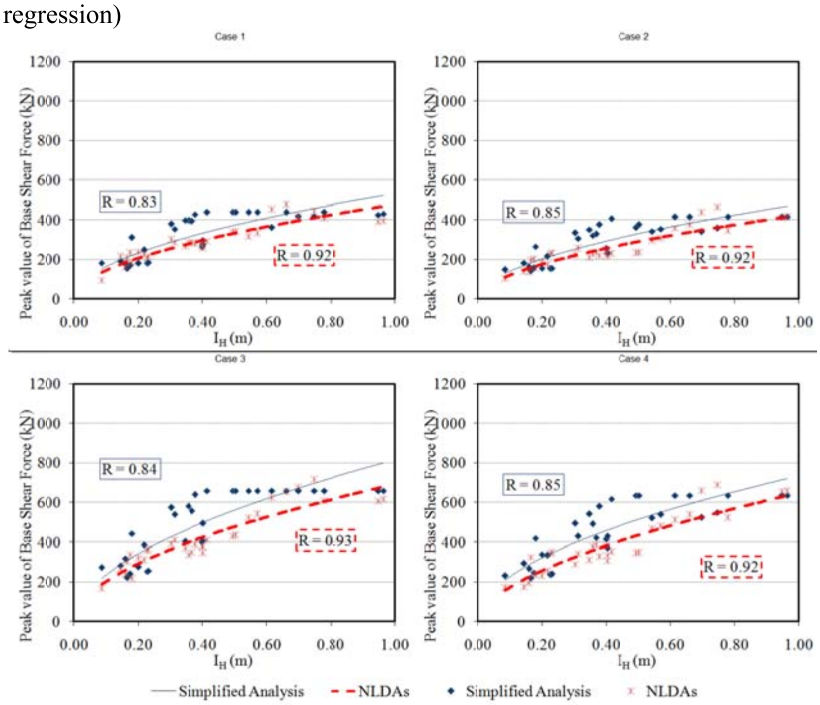

Fig. 11 Comparison in terms of peak value of Base Shear for 4 storey buildings built according to standards after 1971 considering Housner Intensity (*R*, coefficient correlation of non linear regression)

$$I_H = \int_{0.1}^{2.5} PVS\ (T, \xi)dT \tag{17}$$

As it was already shown in Masi *et al.* (2011) an integral seismic parameter, such as I_H, is more effective than peak (e.g. acceleration, a_g) or spectral (e.g. elastic spectral ordinate at the fundamental period of vibration of the building) parameters in representing the damage potential of a ground motion. Moreover, some authors of this paper have developed (Chiauzzi *et al.* 2011) a relationship between EMS-98 (Grünthal 1998) and Housner Intensity, on the basis of strong motion recordings and macroseismic data catalogues. In this way, Housner Intensity becomes a fundamental element to construct loss scenarios when numerical simulation techniques of the seismic response are used (for example Puglia *et al.* 2012).

The comparison shows that when the results are displayed with reference to the Housner Intensity the correlation is higher then using a_g, particularly for the Simplified Analysis methodology. The same happens comparing the results in terms of peak values of displacement (herein not reported for sake of brevity). For this reason, in the following the results are always displayed with reference to the Housner Intensity.

Generally, SA method shows a higher dispersion, and the agreement between the SA and NLDA methods' results, can be considered satisfactory with respect to the base shear force. NLDAs provide generally higher values (up to 20%) in the 2 and 4 storey buildings, while differences of about ±15% can be found for the 8 storey buildings.

On the contrary, the SA method underestimates the displacement values in the 4 and 8 storey buildings, showing increasing differences when the seismic intensity increases. Further, the results of the SA method show a slight overestimation for the 2 storey buildings.

The influence of the stiffness of the edge beams (Flexible Beam, FB or Rigid Beam, RB) as well as the size of the floor (small, Cases 1 and 2 or large, Cases 3 and 4) appears to be quite irrelevant, irrespective of the adopted approach either SA or NLDA. The values of base shear force are almost coincident for case 1 and case 2 (small floor size and RB and FB, respectively). Some differences can be observed in terms of peak value of displacement especially when the results of SA are taken into account. Differences, although still quite small, in terms of base shear force can be observed for the cases 1 and 2 and the cases 3 and 4 corresponding to small floor size and large floor size, respectively. On the other hand, there is no influence in terms of peak value of displacements.

As for the SA methodology, it has been observed that the buildings slip into the non linear range for accelerograms having I_H values of about 0.5 m for 2 storey buildings, and 0.8 m for 4 storey buildings. 8 storey buildings generally remain into the elastic range. This behaviour is outlined by the fact that after a certain limit of I_H the base shear force tends to remain constant because the yield limit of the structure has been reached. This effect is more evident for the SA then for the NLDA because in the SA an equivalent SDOF system with elastic perfectly plastic behaviour is assumed.

The peak values of displacement tend to be quite insensitive to the building height. This behaviour occurs for all the investigated buildings when a SA is performed, the where peak values computed with the strongest ground motions increase up to about 40 mm for all the building heights under study (see Figs. 11, 13 and 15). On the other hand, when an NLDA is undertaken, the displacements are very similar for the 4 and 8 storey buildings (max values up to about 50 mm, see Figs. 11 and 15), and quite lower for the 2 storey buildings (max values up to about 30 mm, see Fig. 13). The conservation of displacements is due to the fact that the peak value of

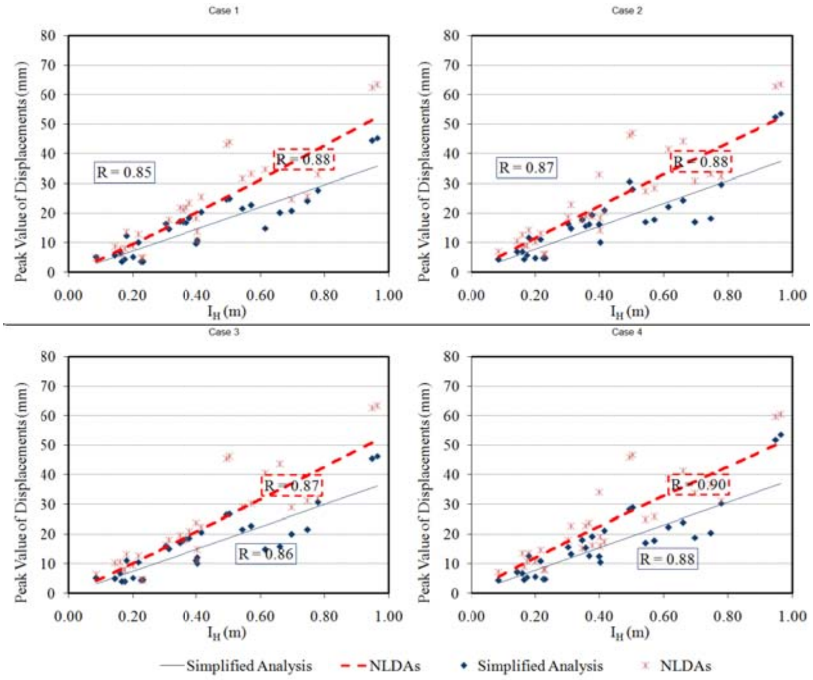

Fig. 12 Comparison in terms of peak value of displacement for 4 storey buildings built according to standards after 1971 considering Housner Intensity (*R*, coefficient correlation of non linear regression)

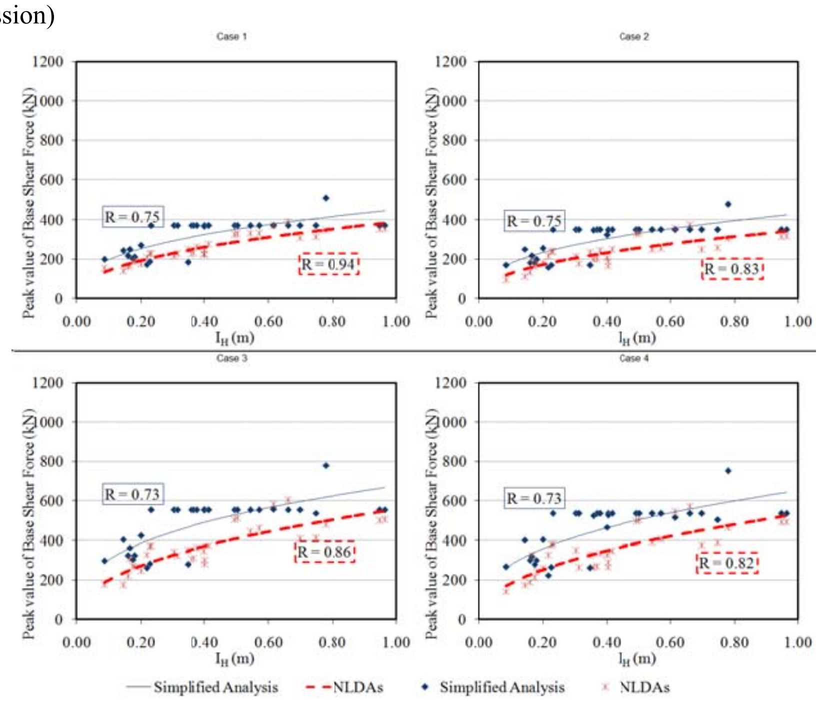

Fig. 13 Comparison in terms of peak value of base shear force for 2 storey buildings built according to standards after 1971 (*R*, coefficient correlation of non linear regression)

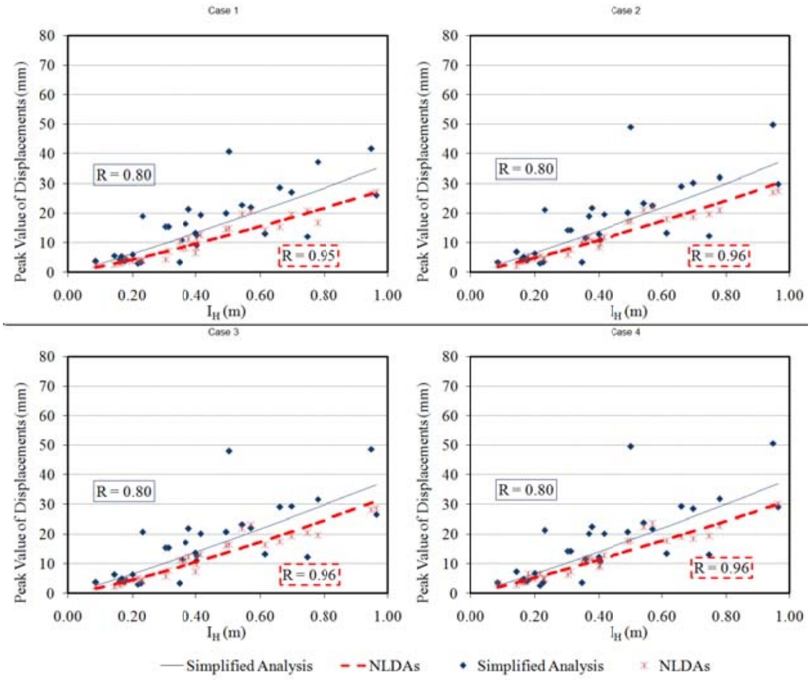

Fig. 14 Comparison in terms of peak value of displacement for 2 storey buildings built according to standards after 1971 (*R*, coefficient correlation of non linear regression)

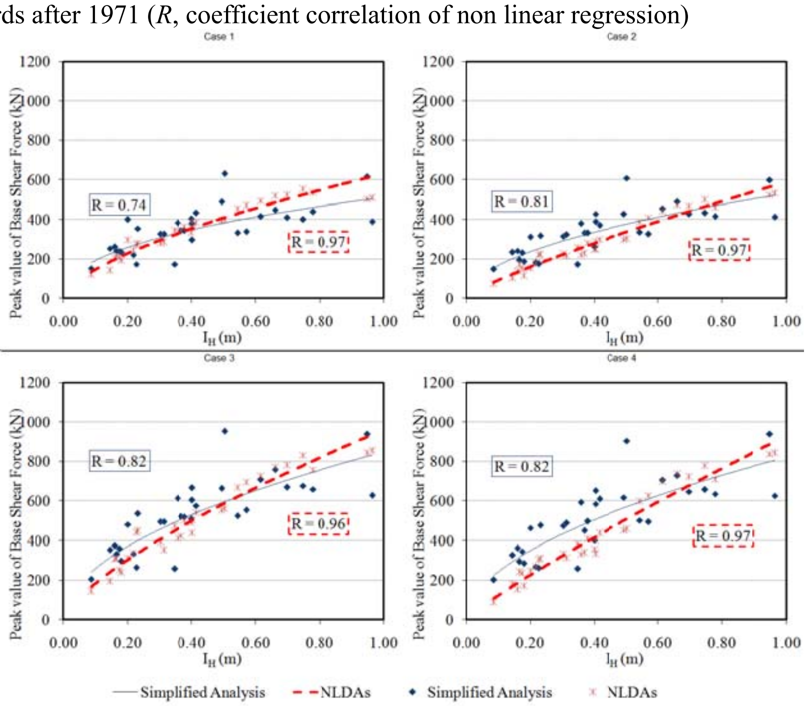

Fig. 15 Comparison in terms of peak value of base shear force for 8 storey buildings built according to standards after 1971 (*R*, coefficient correlation of non linear regression)

Table 4 Equivalent elastic period of vibration for 2, 4 and 8-storey

		$_y$	F_y	T_y	T_{eq}
		[mm]	[kN]	[sec]	[sec]
2 - Storey	Case 1 post 71es	9	370	0.51	1.09
	Case 2 post 71es	10	349	0.54	1.11
	Case 3 post 71es	9	555	0.52	1.22
	Case 4 post 71es	10	534	0.55	1.25
4 - Storey	Case 1 post 71es	19	438	0.98	1.48
	Case 2 post 71es	22	415	1.06	1.59
	Case 3 post 71es	19	659	1.01	1.54
	Case 4 post 71es	22	636	1.08	1.66
	Case 1 pre 71es	24	507	0.99	1.87
	Case 2 pre 71es	24	479	1.01	1.91
	Case 3 pre 71es	24	780	1.00	1.93
	Case 4 pre 71es	24	752	1.02	1.90
8 - Storey	Case 1 post 71es	47	653	1.80	1.32
	Case 2 post 71es	45	607	1.81	1.34
	Case 3 post 71es	47	987	1.85	1.35
	Case 4 post 71es	45	941	1.85	1.36

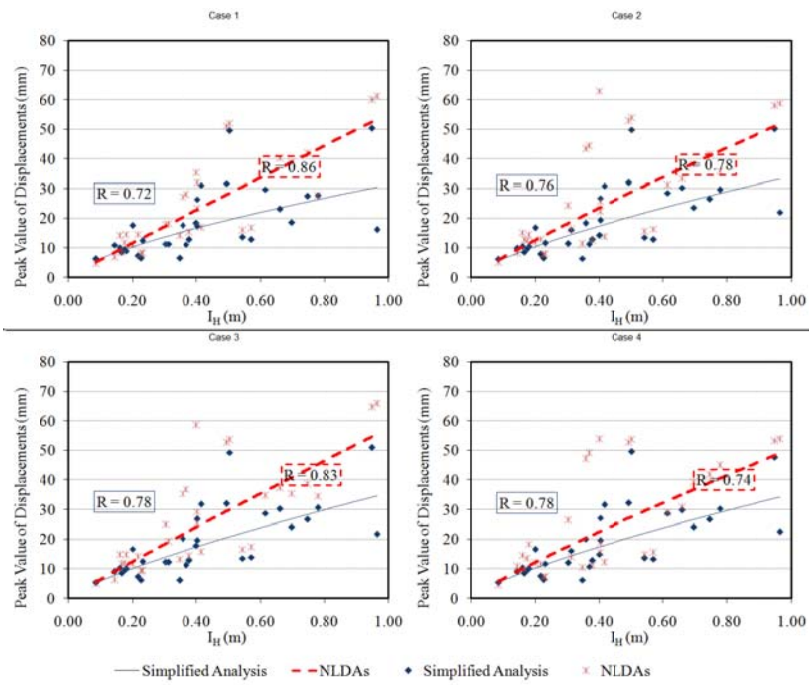

Fig. 16 Comparison in terms of peak value of displacement for 8 storey buildings built according to standards after 1971 (R, coefficient correlation of non linear regression)

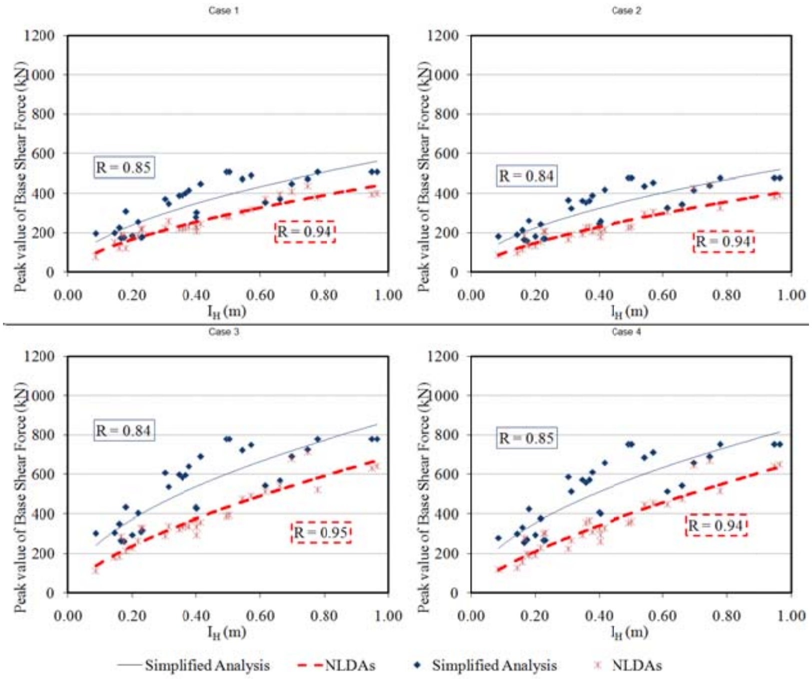

Fig. 17 Results of NLDA and SA on the 4 storey buildings built according to standards before 1971. Comparison in terms of peak value of Base Shear and peak value of displacement for Case 1 and Case 2

displacement is taken into account, therefore beyond a certain period of vibration the displacement spectra, which corresponds to the peak value of displacement for an equivalent SDOF system, tends to have a constant displacement branch. These period values are soon reached for the analysed buildings having rather high period values because the more flexible building direction is investigated and the buildings considered in the study are without infills (bare frames). The period value from which the constant displacement branch starts is quite close to the elastic period of vibration of the 8 storey buildings and it is soon reached for lower rise buildings because they get damaged, and as a consequence, the equivalent elastic period of vibration increases. To support this statements Table 4 summarises the average equivalent elastic period of vibration calculated using the SA model for the accelerogram corresponding to the higher I_H. From the table can be seen that the influence of configuration is almost negligible.

The Figs. 11, 13, 15 show the distribution of the base shear VS Housner Intensity while in the Figs. 12, 14, 16 the peak value of displacement VS Housner Intensity are shown. The solid lines represent the fitted lines of the SA data while the dashed lines represent the fitted lines of the NLDA data. The same model form (non linear regression, $y = a \cdot x^b$) has been used to best fitting of the data of the different approaches. In the Figs. 10-17 the coefficient correlation (R) is reported for each fitted lines.

When NLDA are undertaken the results show that the older buildings (built before 1971) have lower values of base shear force and higher values of peak of displacement when compared to the buildings designed and constructed after 1971, what could be reasonable to be predicted beforehand. On the other hand, the effects of the age are inverted when SA is performed (Fig. 17).

This is due to the fact that for the current work, in the adopted SDOF system for the SA, no degradation effects are taken into account. Therefore, the older buildings that, as a consequence of lower material resistance, have larger structural element size seems to quite better perform. The results outline that SA method cannot be currently applied to older buildings with highly degrading behaviour. Therefore, further study to take into account degrading effects is required.

5. Conclusions

To assess the seismic vulnerability of existing buildings at urban scale, simplified methodologies of analysis need to be adopted. Simplified Pushover-Based Earthquake Loss Assessment approach (SP-BELA) was developed on these bases requiring a two step analysis to identify the structural capacity of a building structure. As first step a simplified pushover analysis is performed to define the parameter of an equivalent SDOF system, which is corresponding to the original structure in terms of period of vibration, displacement capacity and quantity of dissipated energy. Hysteretic rules are defined for loading and unloading branches such as a dynamic analysis can be consequently performed on the equivalent SDOF system instead of the original multi degree of freedom structure.

Main objective of the research work presented in this paper is to validate the simplified methodology implemented in SP-BELA (SA) against the results of more sophisticated nonlinear dynamic analyses (NLDAs). The comparison has been carried out on RC building structures designed only to vertical loads, representative of the “as built” in Italy and in Mediterranean countries with a building stock very similar to the Italian one. 2, 4 and 8 storey frames have been analysed in the paper, representative of low, mid and high-rise buildings. Further, bare frame buildings, i.e., buildings where the infill contribution to the strength and stiffness of the structure can be neglected, have been considered. In NLDAs the non linear and degrading behaviour, typical of the structures under consideration when subjected to high seismic loads, is evaluated using models able to capture with adequate accuracy the non linear behaviour of RC structural elements taking into account stiffness degradation, strength deterioration, and pinching effect.

The comparison between NLSAs and simplified analyses shows that results match reasonably well, even though results relevant to singular buildings can quite different. To this purpose, it should be pointed out that a perfect match was not expected. However, the agreement between results is acceptable for the vulnerability assessment of a large building dataset, where the computational effort of NLDAs can be unaffordable. Nevertheless, further investigations are needed to set the equivalent SDOF system and improve the match between results of SA and NLDA. A better identification of the structural performance through SA could also be obtained introducing correction factors. However, further comparison between SA and NLDA is needed to properly quantify these factors.

Acknowledgments

The work reported in the present paper has been partly carried out in the frame of the DPC-ReLUIS 2005-2008 Project, Research Line n. 10 “Development of Data Bases for Risk Evaluation, Emergency Planning and Management” (Task 2 “Ordinary Buildings”).

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