AN EXTENSIVE VALIDATION EXERCISE ON SIMPLIFIED METHODS FOR THE SEISMIC PERFORMANCE EVALUATION OF RC FRAME BUILDING

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ABSTRACT :

Vulnerability is a probabilistic measure of how prone a structure is to damage under a certain shaking severity. A large number of buildings need to be analysed to have the statistic of the seismic performance. Therefore, analytical methods for the vulnerability evaluation make use of simplified methodologies of analysis, which can afford the treatment of a large building population with a reasonable computational effort. On these bases a Simplified Pushover-Based Earthquake Loss Assessment method (SP-BELA) has been developed. SP-BELA adopts a simplified methodology to identify the structural capacity of the building through the definition of a pushover curve. Displacement capacity limits are identified on the pushover curve, and these limits are compared with the displacement demand for each building in the random population, thus leading to the generation of vulnerability curves. The displacement demand can be defined through a displacement spectral ordinate, which represents the peak of response of a Single Degree of Freedom (SDOF) system equivalent to the original structure in terms of amount of dissipated energy and period of vibration. Alternatively, a nonlinear time history analysis can be performed using an equivalent SDOF system with a behaviour that can be identified through the pushover curve. The aim of the research work presented in this paper is to validate the simplified methodology implemented in SP-BELA against the results of sophisticated FE nonlinear dynamic analyses. The comparison between simplified and FE analyses is carried out for RC buildings representative of the "as built" in Italy.

KEYWORDS: reinforced concrete, vulnerability, simplified methods, nonlinear FE analysis

1. INTRODUCTION

A Simplified Pushover-Based Earthquake Loss Assessment (SP-BELA) method has been developed for different structural types as documented in Borzi et al. (2007 a and b) and Bolognini et al. (2008) for RC cast-in-place buildings, masonry buildings and RC pre-cast buildings, respectively. SP-BELA combines: the definition of a pushover curve using a simplified mechanics-based procedure, which is for RC buildings similar to the one proposed by Cosenza et al. [2005], to define the base shear capacity of the building stock and 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 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 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. The demand in SP-BELA is modelled using displacement response spectra anchored to a value of PGA. The latter can be incrementally increased in order to define 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 bases 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. The aim of this paper is to evaluate whether the equivalent SDOF system, with characteristics defined through the pushover curve, is capable to capture the seismic performance of the original structure. 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 nonlinear dynamic analyses on full structural models. RC cast-in-place bare frames non-seismically designed buildings are here taken into account.

2. NONLINEAR DYNAMIC FINITE ELEMENT ANALYSES

A valuable reference for nonlinear dynamic analyses is the research work of Masi (2003), which aimed to study the seismic performance of RC buildings using a purposely set-up procedure, where structures are 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). Beyond this work, investigations on the Italian construction standards before (Vona & Masi, 2004) and after the 1971 (Masi & Vona, 2004) have been undertaken in order to design buildings than can be considered representative of the "as built" in Italy and in Mediterranean countries with a building stock very similar to the Italian one. The proposed model took into account two bays plane frame with a number of storey equal to 2, 4 and 8 with 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 (mainly the longer building direction), whereas the frame effect in the orthogonal direction, the one investigated in the research work of Masi & Vona (2004), is guaranteed by the contribution of the floor slabs spanning between the frame (No Beam, NB). Due to the presence of masonry infills, it is furthermore 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 (Flexible Beam, FB) and emerging beams (Rigid Beam, RB) are very common in the construction standards. The cases of buildings having 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. Figure 1 shows all the case studies taken into account in the current research work.

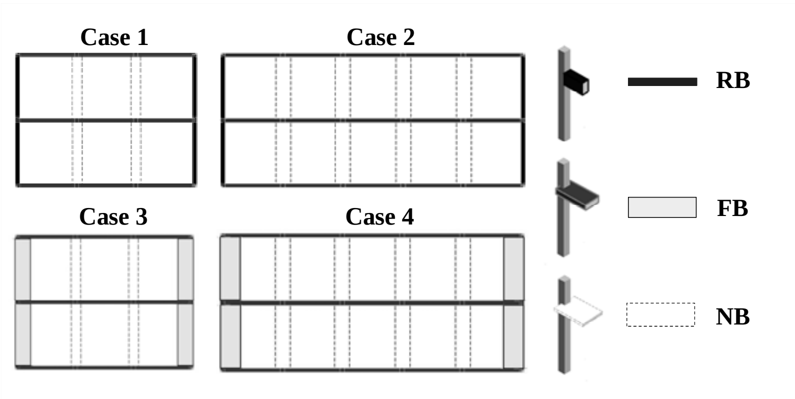


Figure 1: Case study from the research work of Masi & Vona (2004).

The material properties considered for buildings built before and after the 1971 are documented in Vona & Masi (2004) and in Masi & Vona (2004). In the work a macro-modeling based on lumped plasticity has been adopted using the computer program IDARC-2D (Valles et al. 1996). Non linear and degrading behavior, 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 behavior of RC structural elements taking into account stiffness degradation, strength deterioration, and pinching effect. 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). 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 developed completely regarding post-71 RC buildings while, about ante-71 RC buildings, only 4 stories type has been presently analyzed.
In order to investigate also the seismic behaviour of typical buildings in the direction of the frames, a 3D model of a building has been performed. The plan view of the building is shown in Figure 2. The building has 4 storey with an interstorey height of 3 m. The bay length in x and y direction is 6 m and 5 m, respectively. The building was designed considering only gravity and wind loads. On the 3D building the seismic performance has been studied along both the principal directions. The undertaken analyses are: pushover analyses assuming a triangular distribution of lateral forces and dynamic analyses assuming as input motion an accelerogram recorded at the Bolu-Bayindirlik station during the 1999 Duzce earthquake (magnitute 7.2). The accelerogram has a PGA of 7.34 m/sec^2 after 5.75 sec. The FE code used for the aforementioned analyses is SeismoStruct (SeismoSoft, 2007), a fibre element based program for seismic analyses of framed structures. In this computer code the RC section is subdivided in fibres which might have the nonlinear behaviour of concrete and steel. There is therefore no need for an a priori definition of the moment-curvature and moment-rotation relationships, but rather the section size, the dimension of confined core and the longitudinal reinforcement should be directly used as input for the program. Since the building was not seismically designed, thus in the columns only the minimum stirrup amount is present, the effects of confinement have been neglected and the same stress-strain relationship has been adopted for the concrete core and the cover concrete in every section.

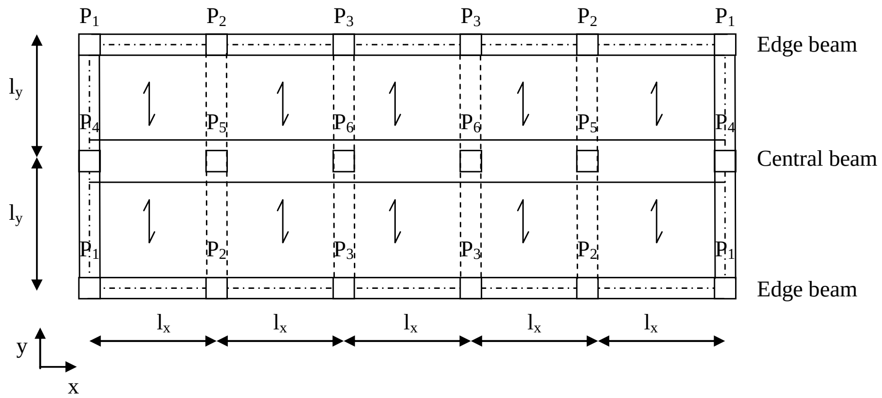


Figure 2: Plan view of 3D building.

3. SIMPLIFIED METHODOLOGY OF ANALYSIS

To assess the seismic vulnerability at urban scale, simplified methodologies of analyses 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., 2007 a). 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 multidegree of freedom structure. In the following details on the simplified pushover and dynamic analysis are given.

3.1 Simplified Pushover Analysis

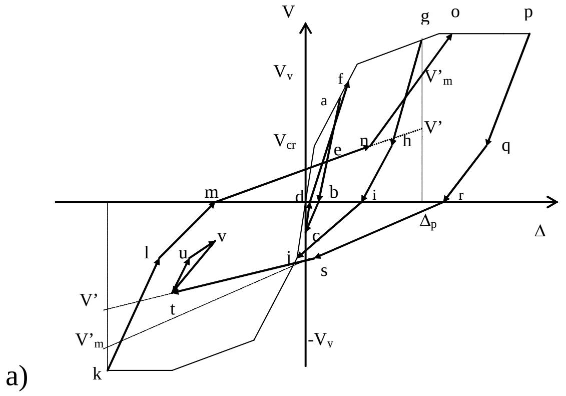
In the proposed simplified method an elastic-perfectly-plastic behaviour is assumed. This effectively means that in order to define the pushover curve, only the collapse multiplier λ (corresponding to the ratio between base shear force and seismic weight) and the displacement capacity need to be defined. The pushover analysis has been performed for horizontal forces linearly distributed along the height, noting however that 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, the shear corresponding to the flexural capacity of the column, and 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. Once the shear capacity has been calculated for every storey, the collapse multiplier is the smallest collapse multiplier at the floors. 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 fail in bending, than a column-sway collapse mechanism will be activated;
- If after the development of plastic hinges in all beams, plastic hinges form in all columns at a certain level, a beam-sway collapse mechanism will be activated.

There could be a situation in which at the storey corresponding to the smallest λ_i some of the columns would be stronger than the beams, or vice versa. Therefore, it cannot be clearly identified whether a beam or a column-sway mechanism will be activated. Hence, a mixed mechanism is assumed. On the pushover curve the displacement capacity corresponding to yielding and collapse should be defined. The displacement capacity is the displacement at the centre of mass of the building, 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 SP-BELA for yield and collapse limit condition the rotation capacity is limited by the chord rotation such as proposed by Panagiotakos & Fardis (2001; CEN, 2003). In order to compare the results with DCs (Damage Curves) models of the plane frames analysed by Masi & Vona (2004), the relationships which lead to the yield rotation capacity have been modified with respect to the ones originally implemented in SP-BELA.

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 Figure 4. 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 (Saatcioglu et al., 1988; Saatcioglu & Ozcebe, 1989). The input parameters for the HHS model described above is the pushover curve. An approximation of the primary curve with three linear branches has been assumed (Figure 4b). 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 stiffness before the cracking load and the secant stiffness (K_{cr}/K_y) and the slope of the post yield branch. 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 & Priestley (1992), Calvi & 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 and 4 times smaller than 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 than approximately 30% of the yielding load V_y. Finally, the HHS hysteretic rules are influenced by the level of axial load. 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. Further details on the HHS model here used to describe the hysteretic behaviour of the equivalent structure are given in Borzi et al. (2000 a, b and c).



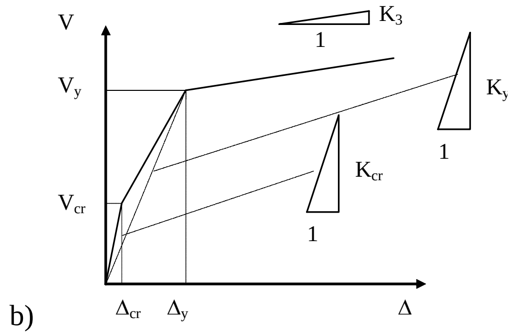


Figure 4: a) HHS model for structural members and b) shape of primary curve used in this work

4. RESULTS COMPARISON

Figures 5 and 6 and 7 to 8 show the comparison of results between the proposed simplified methodology of analyses and the Damage Curves (DCs) methodology proposed by Masi & Vona (2004) for 4 storey frames designed according to standards before and after the 1971, respectively. The “case 3” and “case 4” configurations in Figure 1 are selected for comparison purposes. The results are presented and compared in terms of peak values of base shear force and displacement at the centre of mass of the structure. On the x axes the PGA values of each of the considered accelerograms is reported. From the figures here presented it is possible to conclude that the comparison between simplified and sophisticated FE analysis does not depend on the structural configuration. Therefore, as far as the simplified model of the structure is corresponding to the original structure the chances to capture the seismic performance of the building are independent from number of stories (here not presented), number of bays and construction standards. The curves which summarise the results of the analyses in terms of peak of λ and displacement of the centre of mass match quite well, and the agreement between simplified analyses and FE nonlinear dynamic analysis is considered sufficient to validate the usage of simplified methodologies to analyze buildings at a urban scale level. Furthermore, the peak of response in terms of displacement, that corresponds to the accelerograms with frequency contents which match the fundamental building frequency, are corresponding for simplified and FE analyses. This further confirms that for regular buildings without effective masonry infills like the ones here analysed, rather common in the building stock of Mediterranean countries, the usage of an equivalent SDOF system leads to acceptable results. Finally, it is worth noting the somewhat irregular trend of the curves mainly due to the poor capacity of the PGA to represent the damage potential of a ground motion, particularly in case of natural recordings (Masi, 2003, Masi et al., 2008).

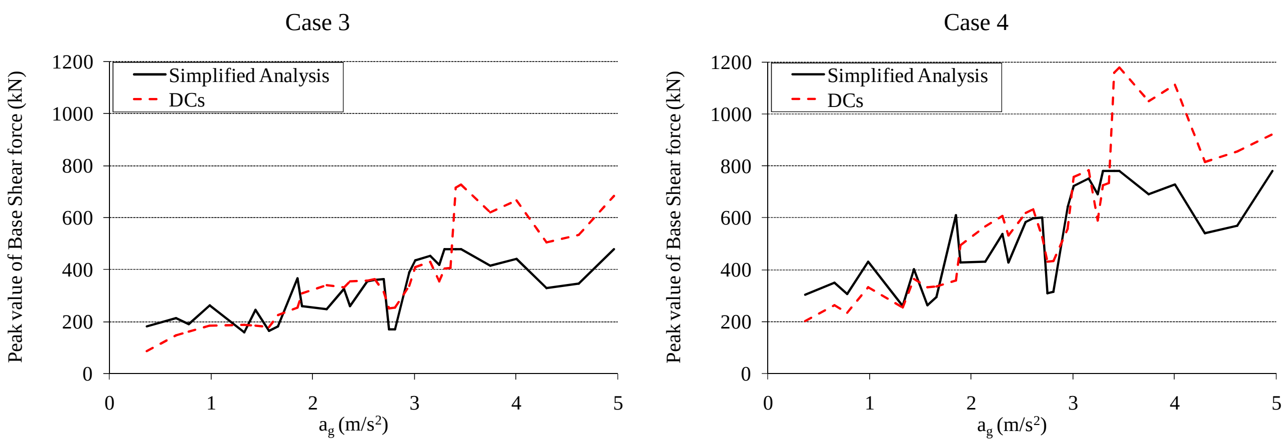


Figure 5: Comparison in terms of peak value of base shear force for 4 stories buildings built according to standards before 1971

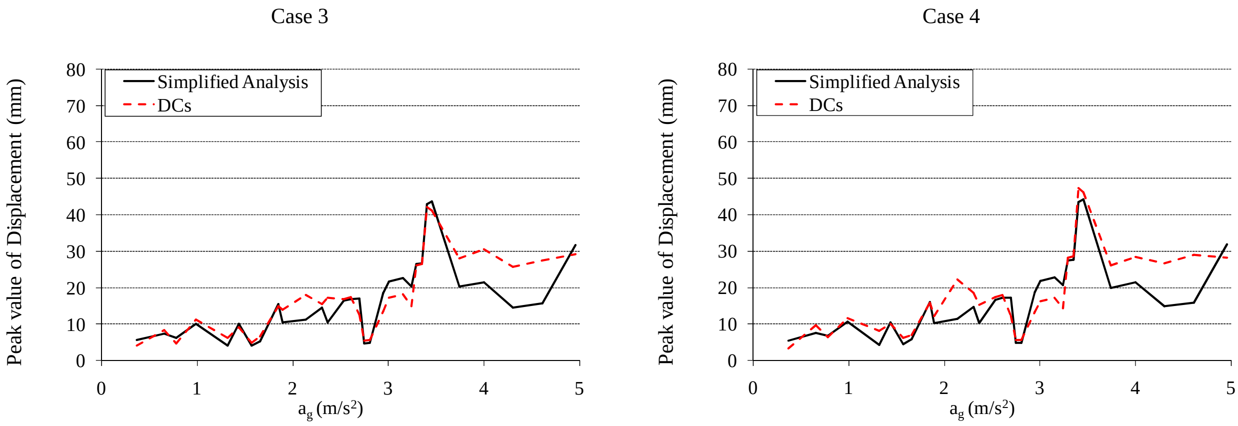


Figure 6: Comparison in terms of peak value of displacement for 4 storey buildings built according to standards before 1971

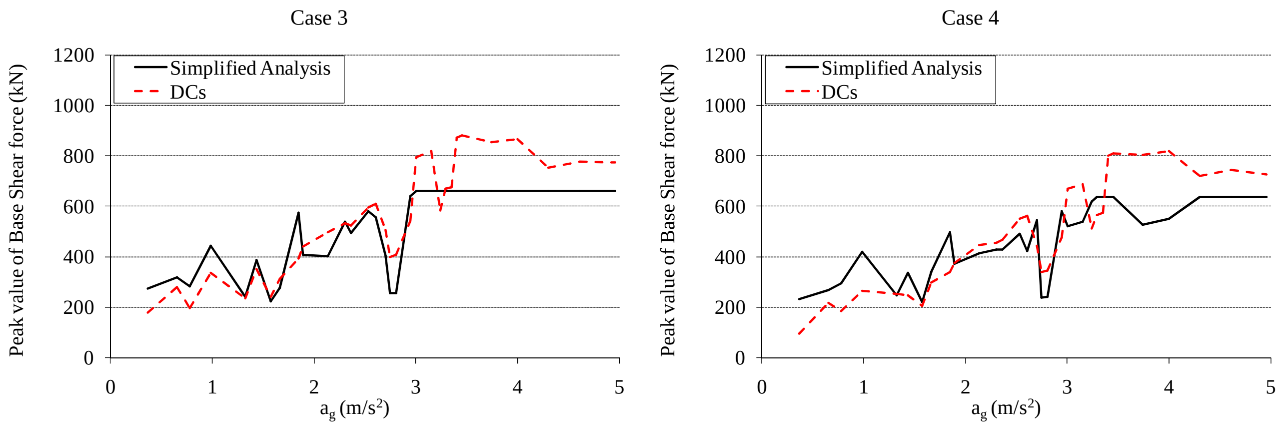


Figure 7: Comparison in terms of peak value of base shear force for 4 storey buildings built according to standards after 1971

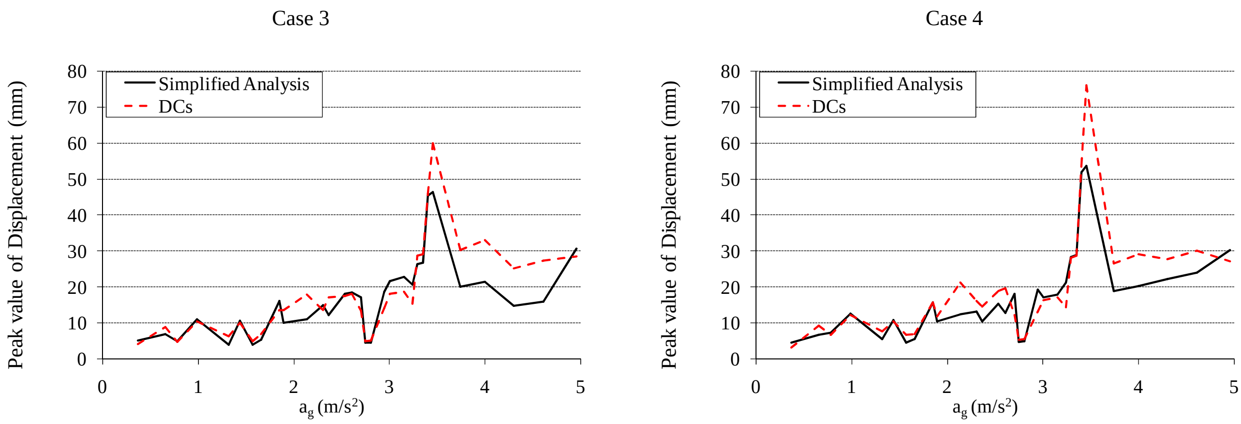


Figure 8: Comparison in terms of peak value of displacement for 4 storey buildings built according to standards after 1971

The comparison between the results of the simplified methodology and the FE analyses on the 3D building models are shown in Figures 9 and 10. Figure 9 highlights the comparison in terms of pushover curves for the two main building directions. For the building direction x, both the FE and the simplified method here proposed predict the activation of a soft storey mechanism at the 3rd interstorey: thus, the comparison between the

obtained results is very satisfactory. For the building direction y the simplified analysis predicts the activation of a global collapse mechanism with the formation of plastic hinges at the base of the columns in the 3rd storey. The FE analysis shows the activation of a global mechanism with concentration of plastic deformation at the 3rd interstorey. In y direction the prediction of the collapse mechanism does not have a perfect agreement, however the comparison between the obtained global results is still satisfactory.
For sake of comparison of dynamic analyses results the building direction x is selected (Figure 10), and the flexible building direction has been already investigated before. The simplified dynamic analyses capture reasonably well the seismic response during the earthquake. Such comparison shall be considered together with the cpu time required by simplified analyses and FE analyses, which range from few minutes to several hours. Therefore, also for 3D buildings the accuracy in terms of detected seismic performance in simplified methodologies is sufficient. This allows the usage of such methods in applications where it is necessary to consider a large building dataset, like in the case of urban scale vulnerability study.

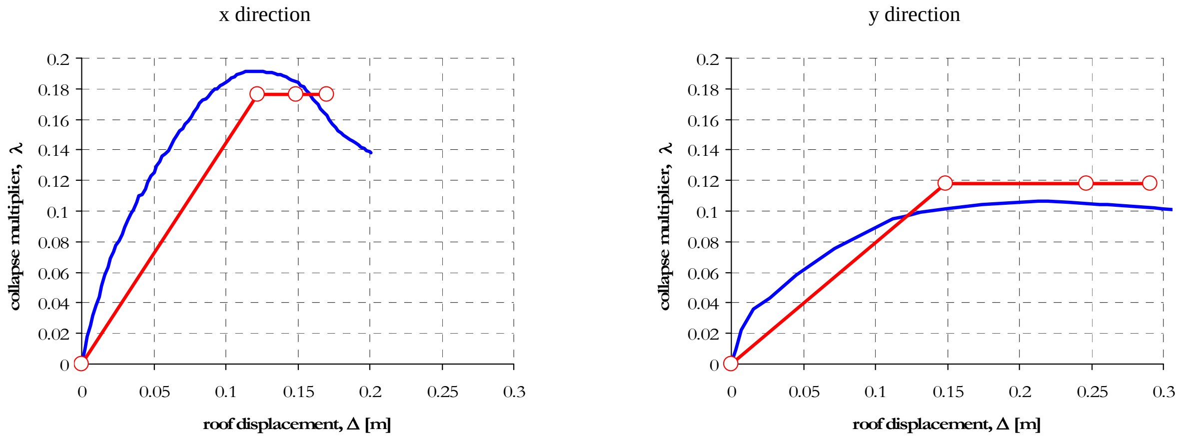


Figure 9: Comparison of pushover curves obtained with the simplified methodology implemented in SP-BELA and FE analyses with SeismoStruct.

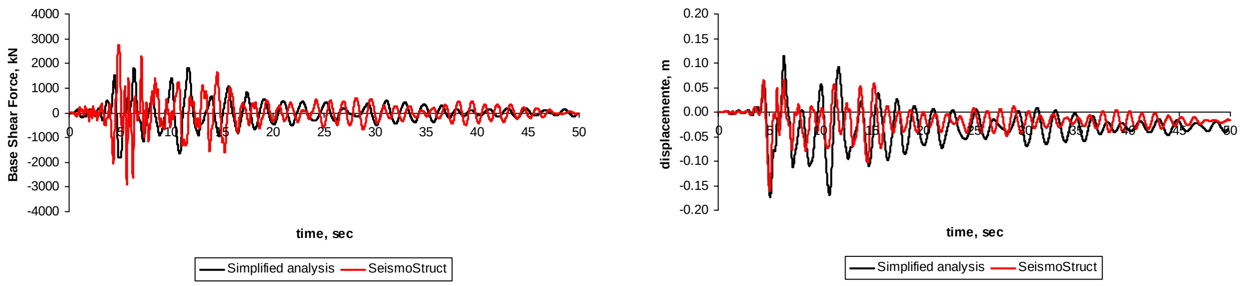


Figure 10: Comparison of time histories of base shear force and displacements in building centre of mass obtained with the simplified methodology and FE analyses with SeismoStruct for the x building direction.

5. CLOSURE

The possibility of implementing in SP-BELA an equivalent SDOF system for the definition of the seismic displacement demand has been explored. Such SDOF system has the advantage of having nonlinear characteristics entirely defined by the pushover curve, and a simplified pushover analysis is already implemented in the current version of SP-BELA. Before proceeding with this development in the definition of vulnerability curves, several comparisons between results of FE dynamic analyses and simplified analyses (i.e pushover and dynamic analysis on the equivalent SDOF system) have been undertaken. The results of nonlinear dynamic analyses are the outcome of the research work of Masi (2003) on 2D frames and Masi & Vona (2004) on 3D frames, as well as nonlinear dynamic analyses of a 3D building undertaken within the current research work. The results of FE and simplified analyses match reasonably well. To this purpose, it should be pointed out that a perfect match was not expected. It is reasonable that on a singular building, results can be also quite different. However, the agreement between results is acceptable for the vulnerability assessment of a large building dataset. Furthermore, no bias in the results comparison has been observed. Therefore, when more buildings will be taken into account, the average prediction is expected to be reasonably realistic.

REFERENCES

Bolognini, D., Borzi, B., Pinho, R. (2008). Simplified Pushover-Based Vulnerability Analysis of Traditional Italian RC precast structures. *14th World Conference in Earthquake Engineering*, China

Borzi, B. (2006). Documento di validazione della metodologia semplificata per la definizione della curva pushover. *Eucentre internal report*

Borzi, B., Calvi, G.M., Elnashai, A.S., Faccioli, E., Bomber, J.J (2000 b). Inelastic Spectra for Displacement-Based Seismic Design. *Journal of Soil Dynamics and Earthquake Engineering*. 21:1, 47-61

Borzi, B., Crowley, H., Pinho, R. (2007 b). Simplified Pushover-Based Earthquake Loss Assessment (SP-BELA) Method for Masonry Buildings. *International Journal of Architectural Heritage* in press

Borzi, B., Elnashai, A.S. (2000 a). Refined Force Reduction Factor for Seismic Design. *Journal of Engineering Structures*, Vol 22:10, 1244-1260

Borzi, B., Elnashai, A.S. (2000 c). Assessment of Inelastic Response of Buildings Using a Displacement and Force Approach. *Journal of Tall Buildings*, 9:4, 251-277

Borzi, B., Pinho, R., Crowley, H. (2007 a). Simplified Pushover-Based Vulnerability Analysis for Large Scale Assessment of RC Buildings. *Engineering Structures* 30:3, 804-820.

Calvi, G.M., Pinto, P.E. (1996). Experimental and Numerical Investigations on the Seismic Response of Bridges and Recommendations for Code Provisions. *ECOEST & PREC8*, Report N° 4

CEB (1998). Seismic design on reinforced concrete structures for controlled inelastic response. *Bulletin 240*, Thomas Telford Ltd, London

Cosenza, E., Manfredi, G., Polese, M., Verderame, G.M. (2005). A multi-level approach to the capacity assessment of existing RC buildings. *Journal of Earthquake Engineering* 9:1, 1-22.

Masi, A. (2003). Seismic vulnerability assessment of gravity load designed R/C frames, *Bulletin of Earthquake Engineering*, Vol. 1, N. 3, pp. 371-395.

Masi, A., Vona, M. (2004). Vulnerabilità sismica di edifici in c.a. realizzati negli anni ’70. *Atti del 11° Convegno Nazionale L'Ingegneria Sismica in Italia*, Genova. (*In Italian*).

Masi, A., Vona, M., Mucciarelli, M. (2008). Selection of natural and synthetic accelerograms for seismic vulnerability studies, *Journal of Structural Engineering*, ASCE Special Issue devoted to “Earthquake Ground Motion Selection and Modification for Nonlinear Dynamic Analysis of Structures” (submitted).

Panagiotakos, T., Fardis, M.N. (2001). Deformation of R.C. members at yielding and ultimate. ACI *Structural Journal*, 98:2 , 135-148

Park, Y.J., Ang, A.H.S., Web, Y.K (1987). Damage limiting a seismic design of buildings. *Earthquake Spectra* vol.3, N1

Paulay, T., Priestley, M.J.N. (1992). Seismic Design of Reinforced Concrete and Masonry Buildings. *Wiley-Interscience Publication by John Wiley & Sons*, Inc.

Pinto, A.V. (1996). Pseudodynamic and Shaking Table Tests on R.C. Bridges. *ECOEST & PREC8*, Report N° 5

Priestley, M.J.N (1997). Displacement-based seismic assessment of reinforced concrete buildings. *Journal of Earthquake Engineering*, 1:1, 157-192

Priestley, M.J.N., Calvi, G.M. (1991). Towards a capacity design assessment procedure for reinforced concrete frames. *Earthquake Spectra* 7:3, 413-437.

SeismoSoft (2007). SeismoStruct – A computer program for static and dynamic analysis for framed structures. *(online) Available from URL: www.seismosoft.com*.

Valles, R.E., Reinhorn, A.M., Kunnath, S.K., Li, C., Madan, A. (1996). Idarc 2d Version 4.0: a program for the inelastic damage analysis of buildings. *Technical Report NCEER 96-0010*. Buffalo, N.Y..

Vona, M., Masi, A. (2004). Resistenza sismica di telai in c.a. progettati con il Regio Decreto 2229/1939, *XI Convegno nazionale dell’Associazione Nazionale di Ingegneria Sismica*, Genova. (*In Italian*).