Simplified Pushover-Based Vulnerability Analysis of Traditional Italian RC precast structures

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ABSTRACT :

Vulnerability curves of traditional Italian reinforced concrete (RC) precast structures are evaluated in this paper through the Simplified Pushover-Based Earthquake Loss Assessment method (SP-BELA) proposed in Borzi *et al.* (2008).
Although the prefabrication of RC members in Italy is widely spread, its application has been targeted almost exclusively to industrial buildings, probably because of the speed in construction and the relatively low costs. In the case of residential buildings or several multi-storey structures, other materials and techniques are usually preferred. The main characteristics of the aforementioned RC precast structures are not always consistent with seismic criteria (e.g. monolithic columns fixed at the base and pin-ended beams, absence of seismic rigid frames, shear-resisting connections). In particular, the connections are one of the weak points in terms of local resistance capacity and global seismic response (*fib*, 2003; Calvi *et al.*, 2006).
Within the procedure adopted and described in this paper, four structural typologies are defined as representative of the majority of the current Italian production and used to randomly generate a population of buildings. The structural behaviour of this population is evaluated through simplified pushover analysis. The generation of vulnerability curves is based on displacement capacity limits of the structures and on the displacement demand. The input motion severity is described through the peak ground acceleration (PGA), as it is a parameter commonly used for the seismic zonation in the modern seismic design codes

KEYWORDS: Vulnerability Assessment, Simplified Pushover, RC Precast Structures, Loss Estimation

1. INTRODUCTION

Seismic vulnerability is a measure of how prone a building is to suffer damage for a given severity of the ground shaking. The aim of much of the research work dedicated to this subject is to give a mathematical formulation to the vulnerability. The two most largely utilised methods concerning its definition are: damage probability matrices and vulnerability curves. The latter describe the conditional probability of exceeding a certain damage limit state given the intensity of the ground motion. The same probability in discrete terms is a component of a damage probability matrix. The conditional probability P_{ik} that the damage state d_i will be achieved or exceeded for a ground motion severity s_k is expressed as:

$$P_{ik} = P\ (D \geq d_i \mid S = s_k) \qquad (1.1)$$

where D and S are the variables representing the damage and the severity of the input motion, respectively.
In this paper, the Simplified Pushover-Based Earthquake Loss Assessment method (SP-BELA) for the definition of vulnerability curves proposed by Borzi *et al.* (2008) is employed in order to study the vulnerability of RC precast buildings selected to be representative of the "as built" in Italy. The input motion severity is described through the peak ground acceleration (PGA) as this is consistent with the parameter commonly used for the seismic zonation in current design codes. Displacement response spectra anchored to PGA are also utilized, thus allowing the relationship between the frequency content of the ground motion and the period of vibration of the different structural typologies to be considered.

This paper begins with a brief description of the structural typologies selected for the research study. The details of the SP-BELA methodology in terms of definition of building capacity and the seismic demand have been already presented in Borzi *et al.*, (2008). Therefore, in this paper only the specific details of the precast structures are described. A validation of the proposed methodology has been possible in terms of non-linear response of RC precast structures and not directly for vulnerability curves, since there is no literature dedicated to this subject. Finally, the output is shown in terms of vulnerability curves.

2. STRUCTURAL TYPOLOGIES

In order to define the most representative structural typologies, a time window corresponding to the last 50 to 60 years will be considered. The reasons why prefabrication of reinforced concrete and prestressed reinforced concrete members seemed to feature greater potential for economy than cast-in-situ concrete in Italy lay in the increased speed of construction, the reduction in site labour and the general environmental conditions, higher performances of the materials obtained from special plants, improved durability, flexibility combined with short delivery times. At present, as opposed to other seismically prone countries, these reasons still seem to represent the choice of reinforced concrete pre-cast structures in Italy against the cast-in situ solutions. Right from the start of its diffused employment, the great demand for reinforced concrete pre-cast structures was coming from the field of industrial buildings. The lack of confidence in the prestressing and post-tensioning techniques in seismic zones and the poor practice in the construction of pre-cast moment resisting frames still limit the use of pre-cast solutions, therefore buildings of high importance such as schools and hospitals have always been constructed using alternative not pre-cast structural typologies or not totally consistent with seismic criteria. This situation has favoured a great development of one to three storey pre-cast structures characterized by a relatively high degree of flexibility and with a structural system composed of monolithic columns fixed at the base and free at the top, with pinned beams supported on corbels. Usually the one-storey structures consist of a series of portal frames connected through the precast roof slab. The stability against the horizontal actions is entirely demanded to the columns, which act like vertical cantilevers, while the global ductility is substantially due to the maximum rotational capacity of the plastic hinges at the base of the columns. The multi-storey structures are usually constructed with L-shaped and inverted T-shaped floor beams and double T-elements or hollowcore slabs cast-in-situ or pre-cast concrete cores used for staircases and lift shafts may partially provide horizontal stability, but they should follow seismic criteria and arranged in order to reduce the torsional effects. The use of perimeter shear walls or rigid frames is not yet a common and largely utilized solution in Italy.

Usually the precast elements are connected through discontinuous connections composed by steel elements (bars or bolts and C-shaped profiles) assembled without cast-in-place concrete. Although the typology of the connections is extremely wide, their seismic behaviour is generally based on the shear strength capacity (examples of typical connections are depicted in Figure 2.1.

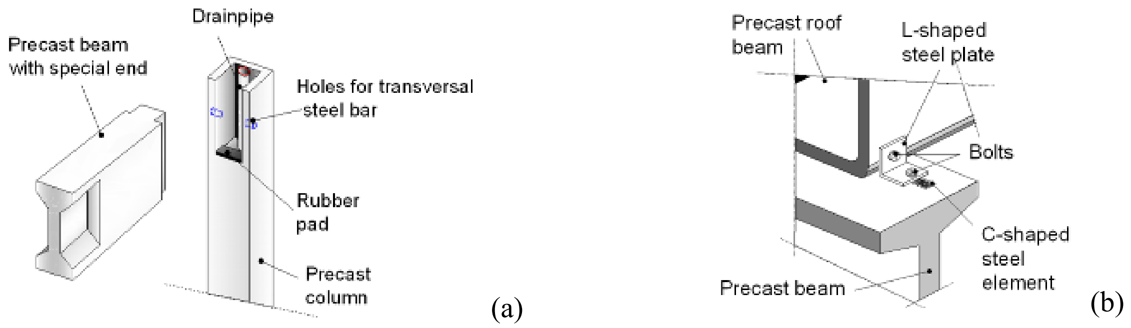
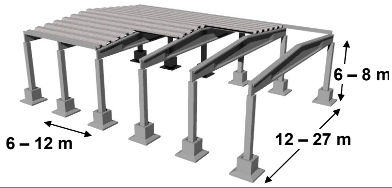
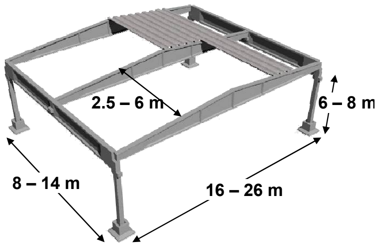
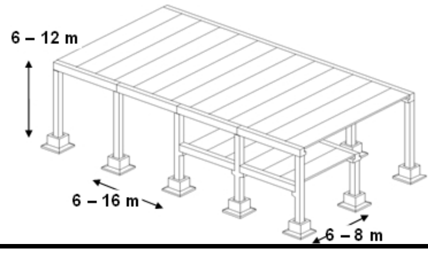


Figure 2.1: Example of typical connections: a) transversal steel bar for the connection between column and I-shaped saddle beam (diameter of the bar ranging from 24 to 30 mm, steel grade 8.8); b) bolted connection between the roof elements and the beam (steel grade 8.8)

The structural typologies considered to be representative of the Italian "as built" on the basis of what has been stated above are summarised in Table 2.1, where the dimensions near the figures refer to typical solutions in seismic zones. All structural types will be considered as seismically and not seismically designed, with the exception of the two storey structure (Type 4) only considered as seismically designed according to the old code

(D.M: 16 gennaio 1996). This is justified by the fact that multi-storey pre-cast structures are relatively new and, therefore, they have been designed accounting for seismic forces, since in the recent Italian regulation the whole territory is considered, although with different level of hazard, as seismic.

Table 2.1: Case studies representative of the Italian "as built" RC pre-cast buildings with dimensions typically utilised in seismic regions.

ID	Structural typology	
1	one – storey parallel portals - Saddle double T-shaped beams	
2	one – storey portals – Double T shaped main beams	
3	one – storey parallel portals (see Type 1) – Truss beams	
4	two – storey frames – Saddle double T shaped beams	

3. SIMPLIFIED PUSHOVER-BASED METHOD

The main component of the SP-BELA method involves the definition of the capacity of a population of buildings based on a prototype structure, which is carried out using simplified pushover analysis to obtain the collapse multiplier λ defined as the ratio between lateral to axial loads. The peculiarities of the method implemented for precast structures are explained in this paper, whereas the detailed description of the procedure related to reinforced concrete structures is available in Borzi *et al.* (2008).
In order to compute capacity curves, SP-BELA makes use of a simplified pushover methodology that can be employed in the assessment of a large number of buildings with reasonable computational effort. Within such framework, an elastic-perfectly-plastic behaviour is assumed, so that, in order to define the pushover curve, only the collapse multiplier λ and the displacement capacity Δ at the top floor need to be calculated. In particular, for precast structures, the light damage (LS1) and the collapse (LS3) limit states are defined (see Figure 3.1).

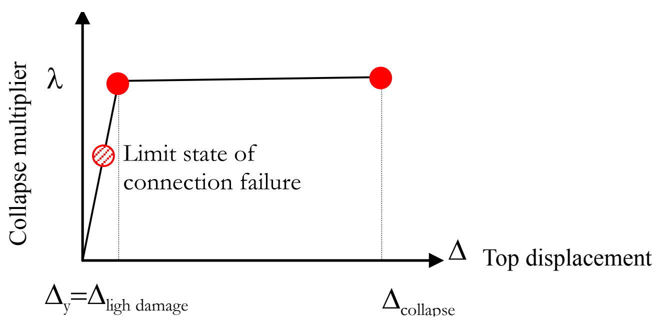


Figure 3.1: Capacity curve for elastic perfectly plastic structural behaviour representative of pre-cast structures

The ultimate displacement is limited by the rotation Θ_u corresponding to the ultimate capacity of the column or by the loss of support condition if the shear capacity of the connections is achieved before the plastic branch of the pushover curve is reached. In this case, the ultimate displacement capacity is equal to the half-length of the support at each end of the beam, since the columns are considered in opposition of phase.
The strength capacity of the connections is calculated for pre-code buildings as the sum of the shear capacity of the RC elements on the top of the columns and the friction resistance of the connection. The friction coefficient

depends on the type of interface which supports the beam (concrete, rubber pads or even steel plates): although these cases may concern a range of values included between 0.3 and 0.8, a value equal to 0.4 is assumed in this study as a conservative assumption, consistent with codes requirements which do not allow relying on friction. Furthermore, an axial load variation of 40% is considered to account for the effects of the vertical component of the seismic acceleration (Calvi *et al.* 2006, 2007a, 2007b). For code designed structures, the shear force due to friction is increased to account for the resistance of the connection, assumed to be the smallest between the shear resistance of the steel and the resistance of the concrete (EOTA, 2001).

In order to determine the dynamic characteristics of single storey buildings, the 60% of the mass of the vertical panels is concentrated at the top of the columns (Calvi *et al.*, 2007b). The equivalent SDOF system of the multi-storey buildings is calculated in accordance with Eq. 13.2.3 of Chopra (2000). Independently of the number of floors, the plastic component of the top displacement is evaluated assuming a mechanism due to plastic hinges located only at the base of the structure. In addition, the length of the plastic hinge is calculated neglecting the contribution due to the strain penetration into the foundation, since socket foundations are used.

4. RANDOM VARIABLES TO DESCRIBE UNCERTAINTY IN BUILDING CHARACTERISTICS

The random variables used to define the pushover curves are: (i) the geometrical dimensions and the loads, as summarised in Table 4.1; (ii) the material properties. A random choice between a characteristic yielding value of 380 MPa and 440 MPa is performed for the steel, whereas randomly chosen cubic characteristic strength values of 35, 40, 45 and 50 MPa or 45, 50 and 55 MPa are associated to the concrete for pre-code and post-code buildings, respectively. In addition, two further random variables are considered to calculate the mean resistance of steel and concrete from the characteristic strength. They have average values of 1.15 and 1.30 and coefficients of variation of 7.5% and 15% for steel and concrete, respectively. The dimensions of the cross sections of the columns are not properly random variables. In order to reproduce the appropriate characteristics of the Italian building dataset in terms of cross-section dimensions and percentages of steel reinforcement, a preliminary design has been undertaken in accordance with the allowable-stress design method (including second-order effects limitations). In addition, the code-designed structures will be subjected to displacement limitations imposed for the reduction of the damage to non-structural elements. The evaluation of the displacements will follow the code provisions that date back to 1996 (DM, 1996).

The code-designed structures account for a lateral force corresponding to 4%, 7% and 10% of the overall seismic mass. Such levels of lateral force (from now called F/W) are related to the coefficients S = 6, 9 and 12 of the DM96 (1996), concerning the different seismic intensity levels defined in the seismic zonation before the OPCM (2003) and the recent seismic zonation proposed by D.M 14th January 2008 (Ref. DPC-INGV linea S1). The pre-code structures are designed for a lateral load equal to 2% of the overall mass.

Table 4.1: Random variables used for the design of the case studies

Description	Dead load (kN/m^2)		Live load (kN/m^2)		Column high (m)		Beam span length (m)				Slab span length (m)			
Sub-class	1, 2, 3	4	1, 2, 3	4	1, 2, 3	4	1	2	3	4	1	2	3	4
μ	1.30	4.00	0.50	5.00	7.00	7.50	24.0	10.0	30.0	5.5	10.0	22.0	7.0	8.0
σ	0.10	1.00	0.15	1.33	0.67	0.50	2.33	0.67	1.00	1.33	1.00	3.00	0.83	1.50
Distribution	Normal													

5. VALIDATION OF THE SIMPLIFIED PUSHOVER CURVES

The procedure presented in this paper has been validated through a comparison between pushover curves defined with the simplified procedure implemented in SP-BELA and proper finite element non linear static to collapse analyses. A direct comparison in terms of vulnerability curves has not been undertaken since in the literature there is not availability of references regarding fragility curves of RC precast structures. The finite element analyses have been performed with a fibre-element based computer program, Seismostruct (Seismosoft, 2006), able to predict the large displacement behaviour and accounting for geometric nonlinearities and material inelasticity. The accuracy of the program in predicting the seismic response of RC framed structures has been tested through a comparison with experimental research works (e.g. López-Menjivar, 2004; Casarotti and Pinho, 2006). The RC precast structures studied in this section have been modelled with non-linear monolithic columns subjected to monotonic lateral load and variable axial load and elastic beams pinned at each end. The level of prestressing in the beams considerably reduces the damage, therefore the assumptions of uncracked sections and elastic behaviour seem to be justified and sufficiently reliable. In the following, the example of a three-storey two-bay frame, similar to the type 4 structure, is described (Figure 5.1a). The properties of the materials, the geometry of the structure and the axial loads at each beam – column joint are summarized in Table 5.1. The predictions, in terms of base shear – top displacement curves, obtained from the two methods are very closed: the variations of yielding and ultimate displacements and yielding moment are equal to 1.0%, 8.0% and 1.1%, respectively. However, it is worth noting that the results of the simplified method are very sensitive to the assumptions regarding the concrete and steel strains at the collapse condition, therefore, opportune values of these variables have to be defined with particular care. In this paper, the strains suggested by Calvi (1999) have been utilized in accordance with the confinement level and the axial load on the columns.

Table 5.1: Properties of the materials, of the base section and of columns

Concrete		Steel		Section						
f_c (MPa)	E_c (MPa)	f_y (MPa)	E_s (MPa)	ρ_1	ρ_2	ρ_v	Depth (mm)	Width (mm)	δ_1	α
56.00	37417	550.00	209000	1.16E-02	1.16E-02	8.14E-03	0.50	0.50	0.06	5.59

Column	Height (m)	Axial Load (kN)	Height (m)	Axial Load (kN)	Height (m)	Axial Load (kN)
Interior	2.85	299	5.30	224	7.70	157
Exterior		161		123		84

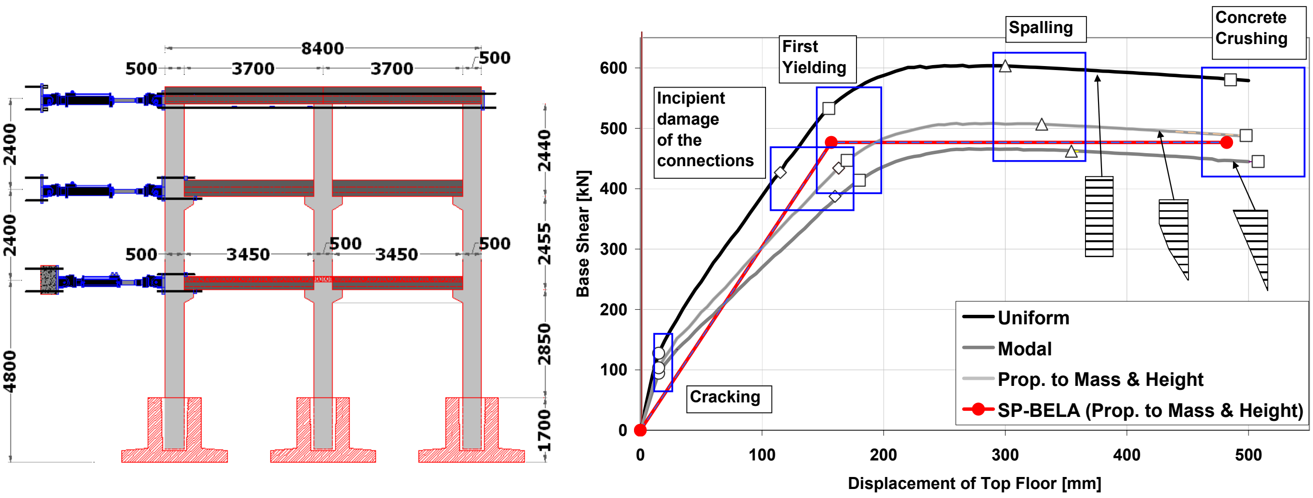


Figure 5.1: a) 2D frame studied; b) prediction in terms of base shear – top displacement curves obtained from FE analyses with different lateral force profiles and simplified method.

6. APPLICATIONS AND RESULTS

For each structural type shown in Table 2.1, a building sample has been generated on the basis of the random characteristics described in the Chapter 4. Each sample, further classified as a function of the seismic design code and seismicity level, consists of 1000 elements for each principal direction. The sample size, carried out to identify and inhibit spurious cases related to unrealistic design solutions, has been found to be sufficient, as a sensitivity study has shown that a further increase would not affect the results of the investigations being undertaken. An amount of 32000 elements of the building population were analysed through the SP-BELA methodology in order to evaluate the distribution of the collapse multiplier λ and of the displacement capacities Δ_{LSi} for the defined limit states, the average and the average ± standard deviation pushover curves and the parameters of the lognormal distribution.

The information obtained from the results of the analysis is briefly discussed in the following. The collapse multiplier depends on the design lateral load level and on the resistance of the type of connection. In Figure 6.1 it can be observed that if the connections are stronger than the capacity of the columns (case 2), the collapse multiplier increases with the design lateral load. The weaker connections of the case 1, instead, result in a collapse multiplier that reduces with the highest values of the design lateral loads. The collapse multiplier is also affected by the minimum dimensions of the column cross-section. Since a 50x50 cm square section has been assumed to be the lower bound solution, in accordance with the characteristics of the majority of the Italian production of precast elements, it means that low-rise structures (e.g. height < 5 m) designed for low lateral forces, are characterized by the more conservative design solution. The high flexibility of the examined structures (the yielding displacement corresponds to the 2% of total drift level) results in displacement ductility values included in the range 3.1 ÷ 4. If the connection failure governs, the ductility is strongly reduced to values lower than 2. The assumption of rigid diaphragms, combined with the fact that in the majority of the cases the cross sections of the columns are squared with symmetrical reinforcement, results in almost the same behaviour of the structures along the two principal directions. Not symmetrical responses are due to the characteristics of the connections; for example, the Case 1 designed for F/W = 10% is subjected to 2% and 8% of the connection failures respectively along the *x* and *y* directions.

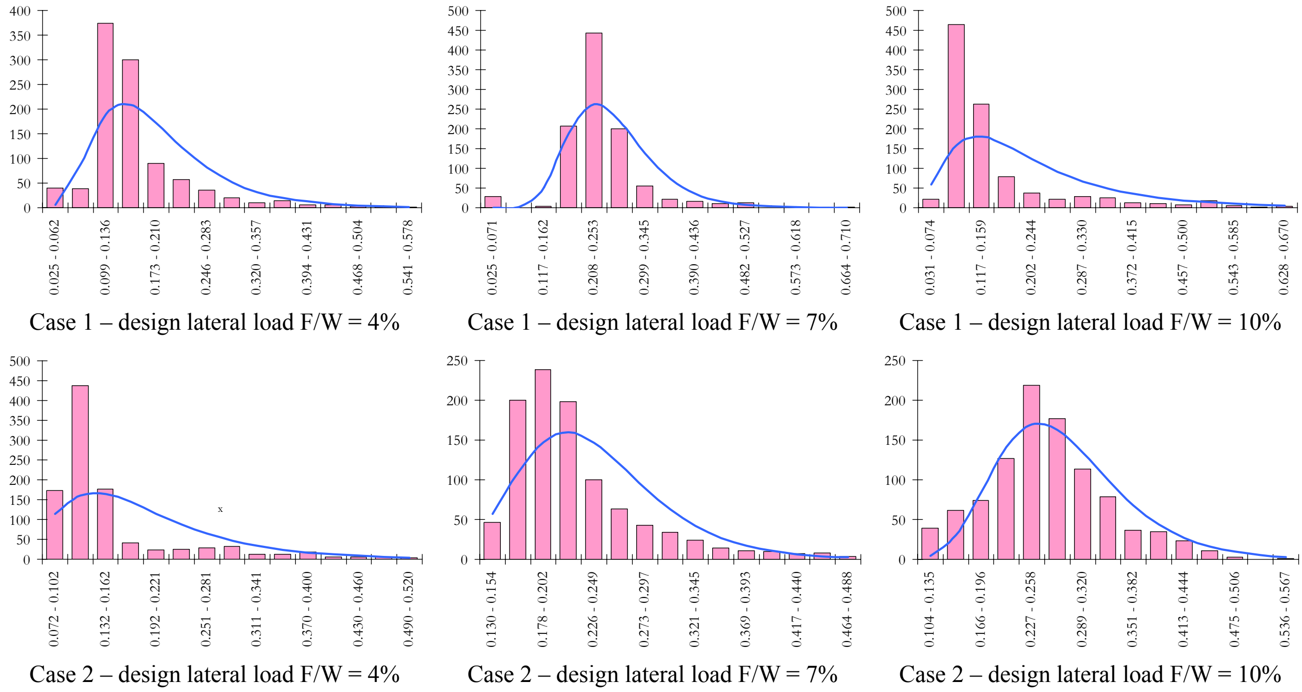


Figure 6.1: Distribution of the collapse multiplier λ of the cases 1 (top) and 2 (bottom) along the principal *x*-direction for different levels of design lateral force (F/W = 4%, 7%, 10%).

The failure of the connections was particularly evident in the Case 4, where about 80% of the cases collapsed before the maximum capacity of the columns was reached for F/W = 4%. For the Case 1 and the Case 3, designed for lateral loads F/W = 10%, such values reduce to 10% and 40%, respectively. The Case 2 was not particularly affected by the capacity of the connections (less than 5% of connection failures independently of the lateral load level). The probability of exceedance of the limit states Δ_{LSi} is not substantially affected by the design value of lateral forces, but strongly depends on the behaviour of the connections. In the Figure 6.2, in fact, it can be observed that the probability related to the attainment of the LS3 limit state is notably lower than the LS1 limit state (left, F/W = 4%), but it increases if the failure of the connections occurs (right, F/W = 10%). This result is more evident if only the Case 4 is examined (see Figure 6.3(left)): if the connection failures govern the response, the corresponding curves of the collapse limit state LS3 are very close to the curves related to the light damage limit state LS1 (80% of the cases for F/W = 7% and 10%); otherwise, the probability of exceedance of LS3 is very low, compared to LS1 (F/W = 4%). In addition, if the Figure 6.2(left) and Figure 6.2(right) are compared, it can be stated that the capacity to demand ratio not necessarily increases with the design lateral load level, since this parameter depends on the changes in the dynamic characteristics of the structures and on the characteristics of the spectrum, representative of the seismic input, used to generate the fragility curves.

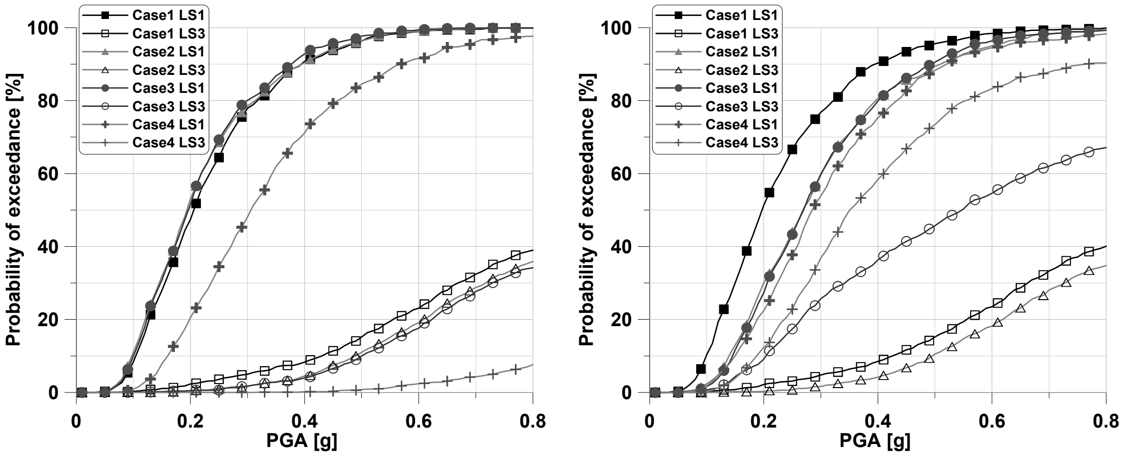


Figure 6.2: Probability of exceedance of the limit states Δ_{SLi} of the four structural types as a function of the lateral force levels F/W = 4% (left) and 10% (right).

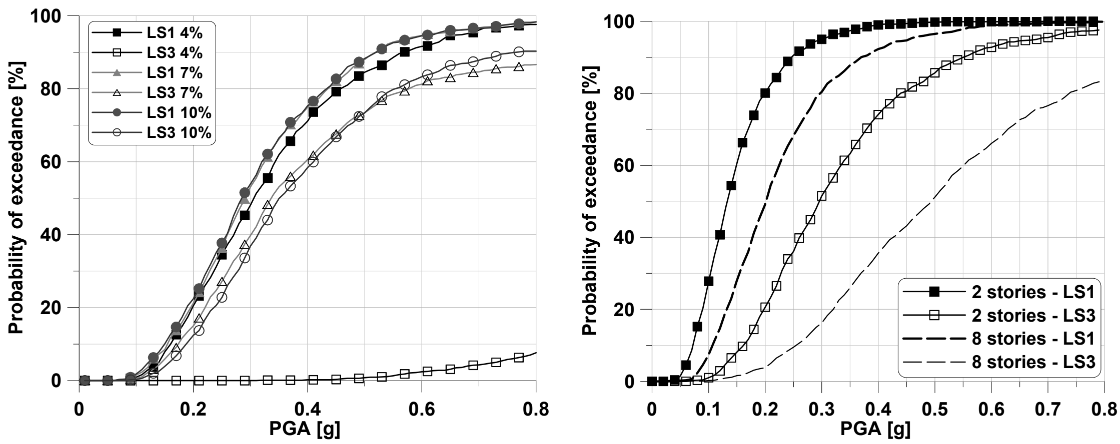


Figure 6.3: Probability of exceedance of the limit states Δ_{SLi} of the Case 4 (left) as a function of the lateral force levels F/W = 4%, 7%, 10%, compared to the case of multi-storey RC structures (right) (Borzi *et al.*, 2007).

7. CONCLUSIONS AND FURTHER DEVELOPMENTS

A simplified pushover-based method for the definition of vulnerability curves has been presented in this paper. The definition of the capacity of a structure through pushover curves is a valuable tool in order to capture the global performances of a building subject to earthquake loads with a reasonable level of computational effort. A simplified approach for the definition of pushover curves has been necessarily adopted, such that the analysis of thousands of randomly-generated samples can be performed within a reasonable time. The definition of whether or not a building survives a limit condition is based on the displacement capacity, which are known to be well correlated with the structural damage. The results obtained from the analysis performed in accordance with the assumptions of the simplified method confirmed that the connections are one of the weak points of the traditional RC precast structures in Italy. In particular, the number of connections which achieve the failure condition increases with the design lateral force level. This is essentially justified by two fundamental reasons: the connections were not properly designed until the publication of the most recent seismic design code (OPCM N° 3274, 2003) and their resistance fully depends on their shear strength capacity. This result is particularly evident for the multi-storey buildings examined, which are representative of sensitive structures subjected to large crowding. For the highest values of F/W, the fragility curve of the collapse limit state is very close to the curve related to the light damage limit state.

Due two the assumption of rigid diaphragms and to the symmetric geometry of the cross section and of the arrangement of the longitudinal re-bars in a large number of cases, the structures considered in this paper are characterized by almost similar behaviour along the two principal directions. Different responses are due to the behaviour of the beam to column connections, except the case of vertical threaded bars (Case 4).

Although the application of the simplified method has given important information, which confirmed the results of past research works on a limited number of structures (Calvi *et al.*, 2007), further development is required in order to thoroughly capture the behaviour of RC precast structures under seismic loading and refine the method. In particular, it will be necessary to account for the effect of: (i) flexible diaphragms in the case of one-storey structures; (ii) panel-structure interaction; (iii) staircases and lift shafts for multi-storey structures (not necessarily present or to be considered rigid elements); (iv) irregularities in plan and in elevation.

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BIBLIOGRAPHY

Borzi B., Pinho R., Crowley H. (2008). Simplified Pushover-Based Vulnerability Analysis for Large Scale Assessment of RC Buildings, *Engineering Structures*, **30:3**, 804-820.

Borzi B., Pinho R., Crowley H. (2007). SP-BELA: a mechanics-based method based on simplified pushover analysis for seismic vulnerability assessment, *Proceedings of the 12th Italian Conference on Earthquake Engineering*, Pisa, Italy, Paper no. 160. (in Italian)

Calvi G.M., Filippetto M., Bolognini D., Nascimbene N. (2007). Strutture prefabricate in c.a. in zona sismica: stato della pratica costruttiva Italiana e principali problematiche di progettazione, *L'Ingegneria Sismica*, **1**, 41-56. (in Italian)

Casarotti C., Pinho R. (2006). Seismic response of continuous span bridges through fibre-based finite element analysis, *Journal of Earthquake Engineering and Engineering Vibration*, **5:1**, 119-131.

Chopra A.K. (2000). *Dynamics of structures: theory and applications to earthquake engineering*, Prentice Hall.

Decreto Ministeriale 16 gennaio 1996 (1996). *Norme tecniche per le costruzioni in zone sismiche*. (in Italian)

European Organisation for Technical Approvals (EOTA) (2001). *ETAG 001 – Guidelines for European Technical Approval of Metal Anchors for Use in Concrete, Annex C: Design methods for anchorages*.

Fédération Internationale de la Précontrainte (FIP) (1994). *Planning and design handbook on precast building structures*, Special report, pp. 138.

López-Menjivar M.A. (2004). *Verification of a displacement-based Adaptive Pushover method for assessment of 2-D Reinforced Concrete Buildings*, PhD Thesis, European School for Advances Studies in Reduction of Seismic Risk (ROSE School), University of Pavia, Italy.

Ordinanza del Presidente del Consiglio dei Ministri n, 3316 del 2 Ottobre 2003, (2003). *Modifiche ed integrazioni all'ordinanza del Presidente del Consiglio dei Ministri n, 3274 del 20 marzo 2003*, GU n, 236 del 10-10-2003. (in Italian)

Seismosoft (2006). *Seismostruct, a Computer Program for Static and Dynamic nonlinear analysis of framed structures*, available from URL: www.seismosoft.com.