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Proposal of a Incremental Modal Pushover Analysis (IMPA)

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Abstract. Existing reinforced concrete frame buildings designed for vertical loads could only suffer severe damage during earthquakes. In recent years, many research activities were undertaken to develop a reliable and practical analysis procedure to identify the safety level of existing structures. The Incremental Dynamic Analysis (IDA) is considered to be one of the most accurate methods to estimate the seismic demand and capacity of structures. However, the executions of many nonlinear response history analyses (NL_RHA) are required to describe the entire range of structural response. The research discussed in this paper deals with the proposal of an efficient Incremental Modal Pushover Analysis (IMPA) to obtain capacity curves by replacing the nonlinear response history analysis of the IDA procedure with Modal Pushover Analysis (MPA). Firstly, In this work, the MPA is examined and extended to three-dimensional asymmetric structures and then it is incorporated into the proposed procedure (IMPA) to estimate the structure's seismic response and capacity for given seismic actions. This new procedure, which accounts for higher mode effects, does not require the execution of complex NL-RHA, but only a series of nonlinear static analysis. Finally, the extended MPA and IMPA were applied to an existing irregular framed building.

Keywords: modal pushover analysis; existing building; capacity curve, incremental dynamic analysis

1. Introduction

The aim of this research is to develop an innovative pushover based methodology, simple in application and interpretation, to evaluate structural capacity of buildings.

The idea arises from the need to extend the pushover, a well-known method, also contemplating the two following important aspects that are relevant in the field of seismic construction: A) correlate intensity of the demand and structural response and therefore it is necessary to consider incremental steps of the seismic demand; B) existing structures are frequently higher modes sensitive and therefore traditional pushover techniques are often inapplicable.

Over the years, many experiences have been made during research courses dealing with seismic assessment of buildings existing in areas affected by seismic events (Inel and Meral 2016, Braga *et al*. 2015, Vanzi *et al*. 2015, Nuti *et al*. 2009, Nuti *et al*. 2010), and many studies aimed to define possible alternative methodologies (Fajfar 2000, Antoniu and Pinho 2004, Bento *et al*. 2010, Bhatt and Bento 2010a, b) to what is probably considered nowadays the most reliable approach: the Incremental Dynamic Analysis (IDA).

IDA is a method for estimating the seismic response and capacity of structures over the entire range of structural responses, from elastic behavior to global dynamic instability, performing a set of nonlinear response history analysis (NL_RHA) on a detailed three-dimensional (3D) mathematical model of the structure (Colapietro *et al*. 2015). IDA requires the execution of NL_RHA for a set of ground motions, each scaled for various intensity levels, selected to cover a wide range of structural responses (Vamvatsikos 2002, Vamvatsikos and Cornell 2002, Bayati and Soltani 2016). As recognised even by the developers of IDA, the computation of many NL_RHAs can result computationally extremely demanding and therefore the definition of a simplified method can be useful (Vamvatsikos and Cornell 2005).

This simplification can be done by developing a method based on Nonlinear Static Procedures (NSPs) and therefore more practical and of faster to be implemented (Fiore *et al*. 2016, Resta *et al*. 2013). Of course the selection of the specific NSP technique is relevant. In literature different NSPs have been developed: the Capacity Spectrum Method (CSM) (Freeman 1998), the Modal Pushover Analysis (MPA) (Chopra and Goel 2002), the N2 method (Fajfar *et al*. 2005), the Adaptive Capacity Spectrum Method (ACSM) (Casarotti and Pinho 2007) etc.

Among all the proposed methods the MPA (Goel and Chopra 2004) has been selected in this research since it has been developed to take into account the higher mode contributions to the total response and also because it yields better results compared to a traditional pushover analysis (Goel and Chopra 2004, Chintanapakdee and Chopra 2004, Han and Chopra 2006).

Moreover, MPA has been also extended to execute nonlinear analysis of tall buildings and multi-story unsymmetric-plan buildings (Reyes and Chopra 2011, Kalkan and Chopra 2012, Reyes *et al*. 2015).

Therefore the idea of an incremental pushover based

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analysis requires to replace IDA's NL_RHAs with a set of MPAs. Remembering that MPA procedure maintains the conceptual simplicity and computational attractiveness of standard pushover procedures the simplification obtained is an immediate consequence.

The contribution of this paper is the presentation of a procedure, that is a modal pushover based incremental analysis, named Incremental Modal Pushover Analysis (IMPA) suitable for complex 3D structures. With IMPA structural response will be represented by a multimodal capacity curve describing the multimodal performance for a desired range of seismic intensity.

The IMPA, already introduced in previous papers of Bergami *et al*. (2015) and differently to previous approaches finalised to obtain correlation between seismic demand and a damage index (e.g., spectral acceleration-interstory drifts) (Han and Chopra 2006, Vamvatsikos and Cornell 2002, Bobadilla *et al*. 2008), allow an immediate evaluation of the building state as performing a common traditional pushover (direct correlation among demand intensity-displacements/forces). This approach can be easily used to perform structural analysis of existing structures or performing a displacement based design procedure for both new structures or retrofitting systems for existing structures (Priestley *et al*. 2005, Priestley *et al*. 2007, Bergami *et al*. 2013, 2014, Kim and Choi 2004).

In the following, the MPA is discussed with reference to asymmetric structures and the basic concept of IMPA is presented; then a step-by-step computational procedure is summariszed.

Finally the application of IMPA to an existing building, presenting both vertical and plan irregularities, is discussed; results are compared with IDA and standard pushover.

2. Incremental modal pushover analysis (IMPA)

2.1 Application of MPA to three-dimensional structures

The IMPA procedure to determine the capacity curves uses MPA procedures rather than NL_RHA to estimate seismic demands for each intensity level of earthquake motions. The MPA procedure is described in a convenient step-by-step form (Chopra and Goel 2002). The application of this approximate procedure, first to 9 and 20 story RC-SMRF buildings (Bobadilla *et al*. 2008), and then to vertically 'regular' and vertically 'irregular' generic frames ranging from 3 to 18 stories is used to estimate seismic demands with a satisfactory degree of accuracy (Geol and Chopra 2004, Chintanapakdee and Chopra 2004). Chopra and Goel (2004) extended the MPA procedure to estimate seismic demands of plan-asymmetric buildings. In the extended MPA procedure by Chopra, the seismic demand due to individual terms in the modal expansion of the effective earthquake forces is determined by a non-linear static analysis using the inertia force distribution s_n for each mode, which for asymmetric buildings includes two lateral forces and torque at each floor level

$$s_n = \Gamma_n M \phi_n = \Gamma_n \begin{Bmatrix} m\phi_{xn} \\ m\phi_{yn} \\ I_0\phi_{\theta n} \end{Bmatrix} \tag{1}$$

$$\Gamma_n = \frac{L_n}{M_n} \quad M_n = \phi_n^T M \phi_n$$
$$L_n = \begin{cases} \phi_{xn}^T m1 & \text{for direction X} \\ \phi_{yn}^T m1 & \text{for direction Y} \end{cases} \tag{2}$$

where Γ_n is the nth modal participation factor; M is a diagonal mass matrix of order $3N$, including three diagonal submatrices m, ϕ and I_0: m is a diagonal matrix with $m_{jj}=m_j$, the mass lumped at jth floor diaphragm and I_0 is a diagonal matrix with $I_{0jj}=I_{0j}$ the polar moment of inertia of jth floor diaphragm about a vertical axis through the center of mass (CM); φ_n is the nth natural vibration mode of the structure consisting of three subvectors: φ_{xn}, φ_{yn} and $\varphi_{\theta n}$; the $N\times 1$ vector $\boldsymbol{1}$ is equal to unit.

2.2 The IMPA procedure

The incremental modal pushover analysis (IMPA) proposed, is a pushover based procedure that requires the execution of MPA and an evaluation of structural performance within a range of different seismic actions and intensity. Data resulting from MPA application within an identified range of seismic intensity provides all necessary information to estimate the seismic response from different intensity levels. Differently from MPA, this approach is finalised to develop a multimodal capacity curve in terms of base shear and top displacement: the MPA has been developed and used to analyze displacements and drift distribution. Therefore, dealing with MPA, the evaluation of drifts must be related with another damage index in order to evaluate the structural performance. With IMPA ,the author's want to develop a new pushover procedure useful for the same targets of other pushover methods but more suitable for buildings sensitive to higher order modes.

In the procedure, for each seismic intensity level, the corresponding Performance Point (P.P.) for the multi-degree-of-freedom (MDOF) system, in terms of roof displacement and corresponding base shear, can be obtained by combining the P.Ps determined through the application of one of the existing procedures (in this paper the Capacity Spectrum Method (CSM) has been used but other approaches could be evaluated if considered more suitable); P.Ps obtained for each significant modal shape will be combined through the Square Root of the Sum of Squares (SRSS) rule. It is thus possible to obtain a range of multimodal performance points (P.P.mm), each one corresponding to a specific seismic intensity level: CSM (or other approaches as well) are applied using Response Spectrums (RS) for all the intensity level considered (the RS will be scaled up to obtain a range of intensities such as in IDA with the time histories). By connecting all the P.P.mm a curve can be obtained: this curve is called the "Multimodal Capacity Curve" (MCC). The flowchart of the procedure follows together with a detailed step-by-step description of the IMPA procedure.

1. Compute the natural frequencies, w_n and modes, φ_n for the linear elastic vibration of the building;

2. Select the ground motions and the RS for a range of intensity levels;

3. For the intensity level i, represented by Peak Ground

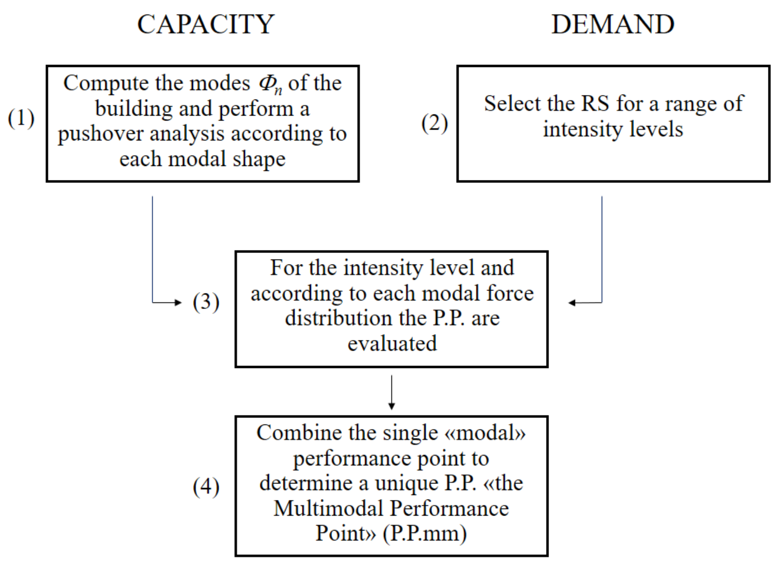


Fig. 1 Flowchart of IMPA procedure

Motion Acceleration (PGA), CSM is adopted to find the P.P. for the selected (predominant) modes: for each mode the capacity curve, which is defined in terms of base shear and roof displacement, is converted into capacity spectrum format and RS into an Acceleration Displacement Response spectrum (ADRS) format. Thus, the curves are plotted on the same chart. Their intersection is taken as the P.P., as shown in Fig. 2(a). Obtain the corresponding P.P. from the capacity curve, as shown in Fig. 2(b). It is important to note that, for the nth mode, if the structure enters a nonlinear plastic stage, then the demand spectrum should be reduced by the spectral reduction factor which depends on the effective viscous damping of structure ξ_{ni}

$$\xi_{ni} = \xi_0 + k\frac{1}{4\pi}\frac{E_{dni}}{E_{S0ni}} = \xi_0 + k\xi_{eqni} \quad (3)$$

where ξ_{ni} is the effective damping for nth mode, ξ_0 is the inherent damping of the elastic structure, about 5% for reinforced concrete structures; E_{dni} is the energy dissipated in an ideal hysteretic cycle, which corresponds to the area enclosed by the hysteresis loop; E_{s0ni} is the maximum strain energy dissipated by the structure corresponding to the area of the hatched triangle. The term k is the damping modification factor that is an adjustment factor to approximately account for changes in hysteretic behavior in reinforced concrete structures. ATC-40 proposes three equivalent damping levels (Type A, B and C) that change according to the hysteretic behavior of the system. Type A hysteretic behavior denotes structures with reasonably full hysteretic loops, like the elastic perfectly plastic oscillator. Type C hysteretic behavior represents severely degraded hysteretic loops resulting in the smaller equivalent damping ratios; in the case study discussed herein this structural type has been adopted as since it is an existing old fashioned r.c. frame building. Type B hysteretic behavior is an intermediate hysteretic behavior between types A and C. The value of k $h$as been derived according to ATC-40 indications Type C hysteretic behavior has been assumed k=0.33.

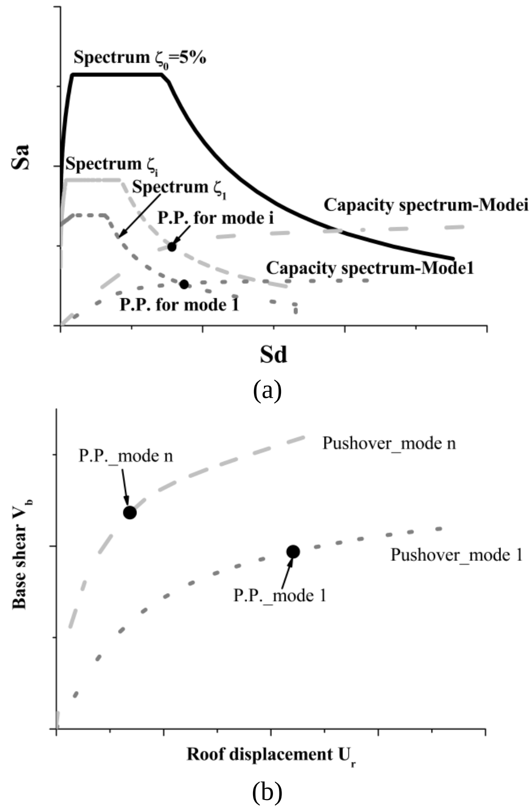


Fig. 2 Evaluation of the performance points (P.P.) for each capacity curve that belongs to the pushover analysis with the selected load distributions: proportional to Mode1. Mode n. (a) for each capacity curve the P.P. is determined via C.S.M. (b) P.P. can be plotted in the V-U plane

4. Determine multimodal performance point (P.P.mm) in terms of multimodal base shear V_{bmmi} and multimodal roof displacement u_{rmmi}.

The value of the roof displacement of the selected control point is determined, for each level of earthquake intensity considered, by combining the modal displacements of the control point u_{rni} using the SRSS rule.

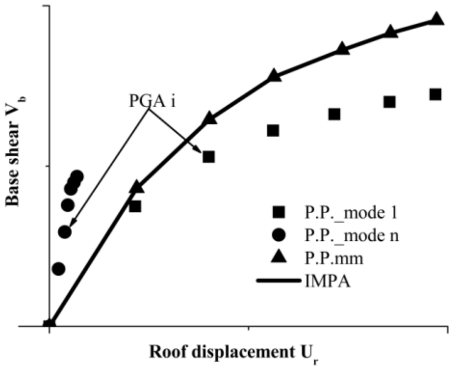


Fig. 3 Construction multimodal capacity curve (MCC) from the IMPA procedure. By applying the SRSS rule with the P.P. obtained with each load distribution (Mode1,..., Mode n) and for each intensity level (the response spectrum is scaled from lower to higher intensity levels) the MCC can be obtained

$$u_{rmmi} = ((\sum_n u_{rni}^2)^{1/2}) \tag{4}$$

Instead for the considered earthquake intensity level, to derive the base shear, the procedure adopted follows:

a) **if the structure remains elastic** the value of the base shear of the structure is determined using the same procedure

$$V_{bmmi,el} = ((\sum_n V_{bni}^2)^{1/2}) \tag{5}$$

b) **if the structure enters the inelastic range** a different procedure is used:

step 1) the total value of the plastic hinge rotation, θ_{cb} at column end of the first level is estimated as the SRSS combination of the values θ_{cbi} obtained with the pushovers with each modal load distribution.

step 2) the corresponding bending moments in the columns are estimated through the relevant moment-rotation diagram at the value of the plastic hinge rotation calculated from the SRSS combination

step 3) shears in the columns are calculated using the corrected bending moments, and the base shear is calculated as the sum of the column shears.

$$V_{bmmi,y} = V(\theta_{cb}); \quad \theta_{cb} = (\sum_n \theta_{cbi}^2)^{1/2} \tag{6}$$

5. Repeat steps 2-4 for as many intensity levels needed to form the IMPA curve, as shown in Fig. 3 where P.P. are the performance points obtained with a lateral load distribution proportional to modal shape 1,..,*n*.; according to MPA approach the multimodal performance point (P.P.mm) can be obtained to trace the multimodal capacity curve (MCC).

3. Case study

3.1 Building description

The building selected as a case study is an existing nine story RC framed building located in Italy, designed for gravity loads only and built in the 1970s. The building consists of a ground floor, an eight-story elevation and a roof terrace: the plan and elevation layout of the existing building are shown in Fig. 4 and Fig. 5, respectively.

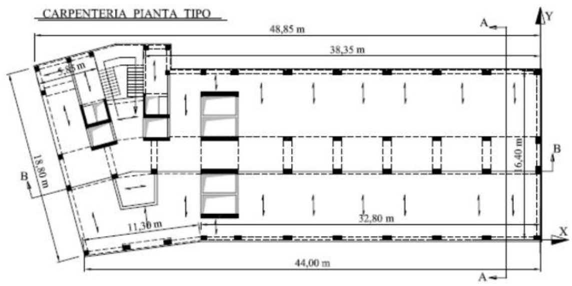
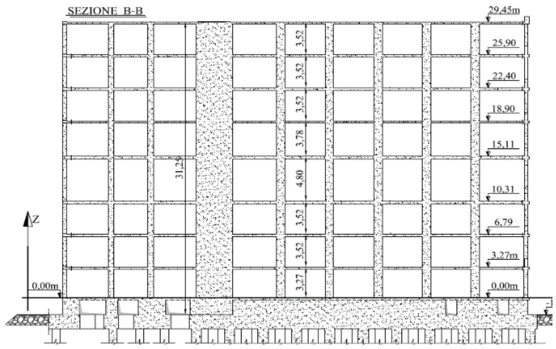
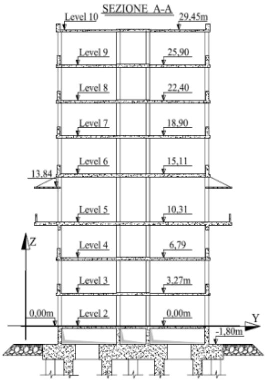


Fig. 4 Plan layout of the existing building (m)



(a)



(b)

Fig. 5 Elevation layout of the existing building, (a) elevation layout (b) section A-A (m)

From a structural point of view, the plan is an irregular polygon where the resistant elements are distributed unevenly: the concentration on one side of shear walls and the one-way beam orientation cause a strong irregularity.

The heights of the floors are irregular and go from a minimum of 3.08 m up to 4.80 m, as shown in Fig. 5.

The cross sections of columns changed from 40×40 cm to 85×40 cm. The cross sections of beams changed from 20×24 cm to 34×114 cm. Roofs are realized with ribbed slabs: each slab is 4cm and thus diaphragmatic behavior can be assumed for the sample frame.

An extensive and rigorous study of the design documents and a site survey were conducted in 2009 to estimate the mechanical properties of the concrete and steel reinforcement in this building. The mean cylinder concrete

Table 1 Material properties from extracted samples (year 2009)-concrete

f_{cm}	10,1	[MPa]	mean cylindrical compressive strength
f_{ctm}	1,40	[MPa]	mean cylindrical tensile strength
E_{cm}	26286	[MPa]	mean modulus of elasticity
v	0,30	[-]	Poisson coefficient
G	10110	[MPa]	Shear modulus

Table 2 Material properties from extracted samples (year 2009)-rebar

f_{ym}	395,8	[MPa]	mean yielding strength
f_{ud}	1,40	[MPa]	mean maximum strength
E_{sm}	209800,0	[MPa]	mean modulus of elasticity

Table 3 Dead loads-characteristic value

Structural element	Geometry (cm)	Dead load (KN/m^2)
Typical floor (concrete and masonry)	H=20+4	7,7
Level 5-6 floor (concrete and masonry)	H=24+6	9,1
Concrete slab	H=30	11,1
Concrete slab	H=24	10,7
Roof	H=20+4	10,1
External cladding (double layer masonry walls)	s=13+8	3,1

Table 4 Live loads-characteristic value

Area	Live load (KN/m^2)
Typical floor	3,5
Stairs and balcony	5,0
Storage room Liv.5-6	6,0
Storage room Liv.2	8,0

compression strength from the site survey f_{cm} is equal to 10.1 MPa. Mean yield strength of reinforcement f_{ym} in site survey is 396.8 MPa. A summary of material properties and design loads is presented in the following tables (Tables 1-4).

Since the purpose of this study is to evaluate the inelastic deformations of the structure before collapse, the permanent load, the variable load and their combination are defined for the structure according to the definition of ultimate limit state (ULS) in accordance with the regulations of the Italian Technical Code-2008 (NTC'08).

A three-dimensional finite element model was built in SAP2000 software (SAP2000 v.15.0, 2011), as shown in Fig. 6: beam and column elements are modeled as frame elements with lumped nonlinearity by defining plastic hinges at the critical sections (extremities of beams and columns): the plastic hinges are automatically defined by the software according to the geometry, the materials and the requirements of FEMA 356, Tables 6-7 for Concrete Beams and Table 6-8 for Concrete Columns. A coupled axial force and biaxial bending moment hinges (P-M2-M3 hinge) are assigned to columns whereas moment hinges (M3 hinge) are assigned to beams. Nonlinear shell elements (Shell layered nonlinear) are used to simulate walls. The foundations in this study are modeled with joint constraints.

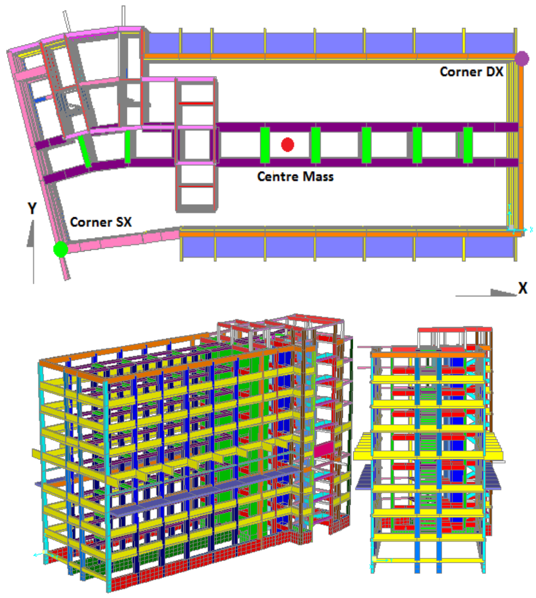


Fig. 6 Finite element model of the existing framed building

3.2 Seismic input

In this study, the seismic action is defined using both the elastic response spectrum according to NTC'08 and a set of 7 natural time histories. In both cases, seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum or by time history; the vertical component of the seismic action has been ignored.

According to NTC'08, seismic action has been defined according to the site and return period detected: the return period depends on the limit state and the category of the existing building. The life safety (SLV) limit state has been adopted. The return period T_R of the earthquake actions is given by

$$T_R = -\frac{V_R}{\ln(1-P_{VR})}\ ;\ \ V_R = V_N C_N \tag{7}$$

where V_R is the reference design life and P_{VR} the probability of exceedance of the seismic action, expressed as a function of the limit state, V_N the nominal life, and C_N the importance coefficient.

From the site survey, the soil foundation can be classified as type B, according to the classification implemented in the NTC'08. For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions

$$\begin{aligned} 0 \le T < T_B: &\quad S_e(\mathrm{T}) = a_g \cdot S \cdot [1 + \frac{T}{T_B}(\eta \cdot 2.5 - 1)] \\ T_B \le T < T_C: &\quad S_e(\mathrm{T}) = a_g \cdot S \cdot \eta \cdot 2.5 \\ T_C \le T < T_D: &\quad S_e(\mathrm{T}) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot [\frac{T}{T_C}] \\ T_D \le T \le 4s: &\quad S_e(\mathrm{T}) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot [\frac{T_C T_D}{T^2}] \end{aligned} \tag{8}$$

where T is the vibration period of a linear single-degree-of-freedom system; a_g is the design ground acceleration on

Table 5 the parameters defining the elastic spectrum

	Parameter	Value	Unit
Independent parameters	V_N	50	year
	C_U	2	-
	V_R	100	year
	T_R	949	year
	ag	0,25	*g*
Dependent parameters	*S*	1,19	-
	η	1,00	-
	TB	0,16	*s*
	TC	0,48	*s*
	TD	2,45	*s*

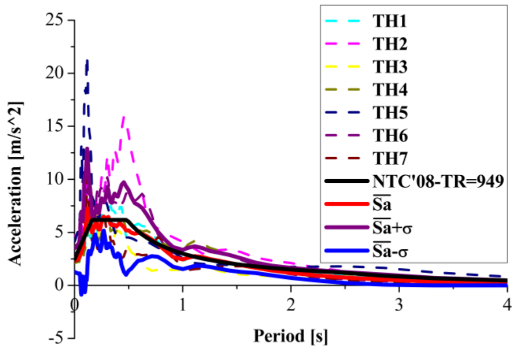


Fig. 7 Response spectra of the code-compliant set of Time Histories (TH): TH1..TH7 are the selected ground motion records, NTC'08 is the response spectra according to the Italian technical code for a returning period of T_R=949 years, S_a is the average response spectra from the set and $S_a \pm\sigma$ is the range of variance according to standard deviation

type A ground; T_B and T_C are the lower and upper limit of the period of the constant spectral acceleration branch, respectively; T_D is the value defining the beginning of the constant displacement response range of the spectrum; *S* is the soil factor; η is the damping correction factor. The values of the parameters employed for defining the spectrum of the horizontal acceleration are listed in Table 5.

According to the elastic response spectrum previous described, a set of 7 natural time histories are defined using Rexel software (Iervolino *et al*. 2010). They are named TH1~TH7. In Fig. 6 the elastic response spectrum defined by NTC'08 is shown with the response spectrum of each time history record.

The mean and standard deviation of the natural records' response spectrums can be calculated with the following equations

$$\overline{S_a}(T_i) = \frac{1}{N-1}\sum_{j=1}^{N} S_{aj}(T_i); \sigma(T_i) = \sqrt{\frac{1}{N}\sum_{j=1}^{N}(S_{aj}(T_i) - \overline{S_a}(T_i))^2} \quad (9)$$

Where $\sigma(T_i)$ represents the standard deviation of the response spectrums of the natural records in correspondence with the period T_i, $S_{aj}(T_i)$ is the pseudo-acceleration of the *j*th spectrum, $\overline{S_a}(T_i)$ is the mean pseudo-acceleration, *N* is the number of the natural records.

For each period T_i, assumes a normal distribution of the natural records' response spectrums, the mean response spectrum of 7 natural records $\overline{S_a}(T_i)$ and a plus and minus one standard deviation (σ) in correspondence with the period T_i to achieve a 68% confidence interval, as shown in Fig. 7.

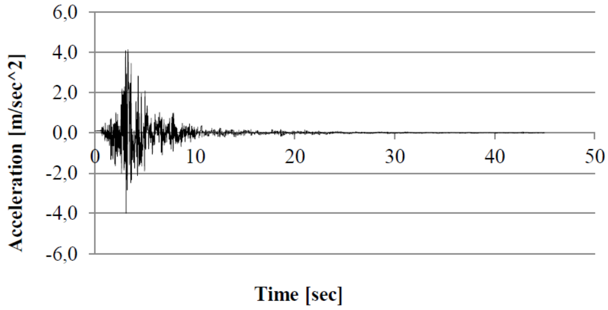


Fig. 8 Example of Time Histories (TH): TH5

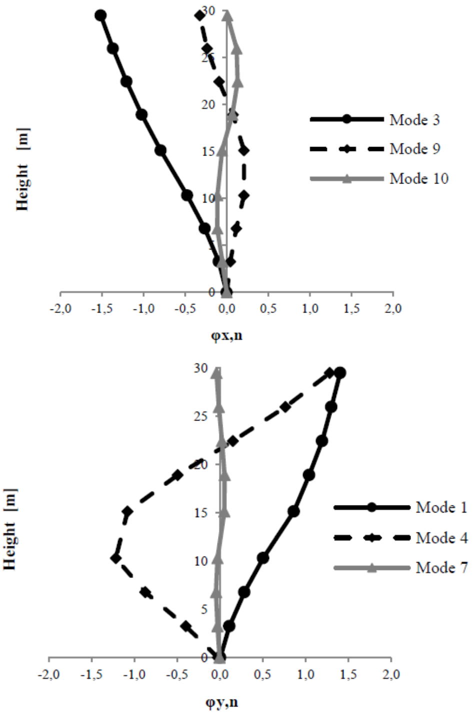


Fig. 9 Modal shapes: main three modes in terms of participating mass along *X* (left) and *Y* (right)

In the the figure is above figure (Fig. 8), as an example, one of the time histories selected is presented; performing the analysis the time lapse 0-10s of each time history has been considered.

4. Results

Modal analysis is employed to identify the dynamic behavior of the existing structure and investigate the relevance of higher modes (Fig. 9). Along *Y* direction the first, fourth and seventh modes exhibit more than 83% of the participation mass and therefore these modes will be considered in the IMPA. In the same way, along *X* direction, the third, ninth and tenth modes exhibit more than 79% of the participation mass and therefore these modes will be

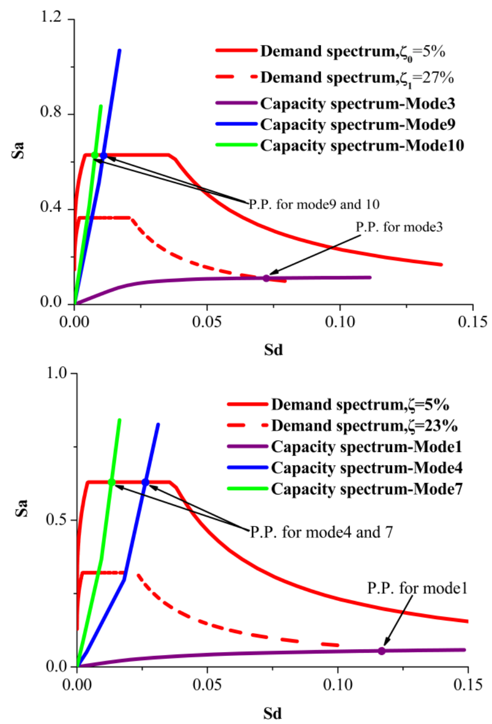


Fig. 10 Evaluation of the P.P. with CSM: in the plot (left) there is the elastic Response spectrum ζ=5% (P.P. for Mode 9 and Mode 10 are in the elastic range) and the Response spectrum reduced according to a damping of ζ=27% (23% is the equivalent viscous damping at the P.P. for Mode 3) and (right) ζ=27% (27% is the equivalent viscous damping at the P.P. for Mode 1)

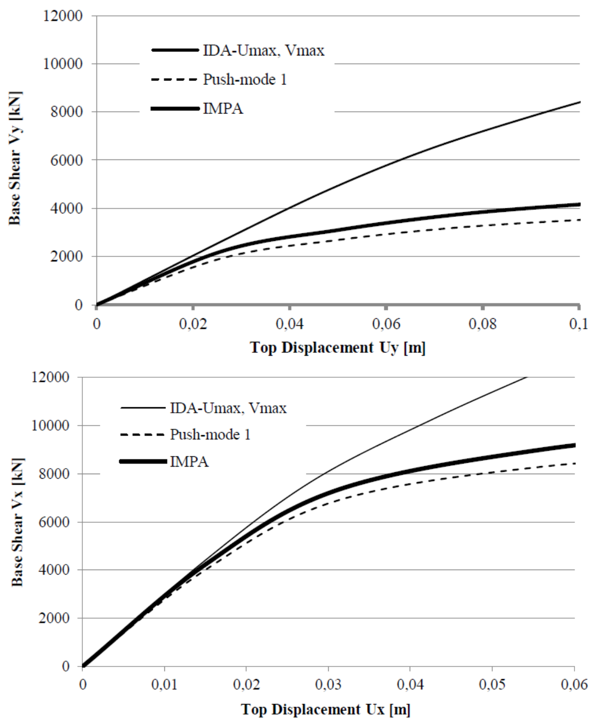


Fig. 11 Capacity curves obtained from different methods: the standard pushover analysis (for the predominate mode), IMPA method and IDA method

considered in the IMPA.

Fig. 10 shows the performance point (P.P.) obtained with the CSM for each one of the capacity curves obtained, applying a pushover analysis with load distribution proportional to the three modal shapes considered. The demand spectrum used has obtained according to NTC'08 (PGA=0.25 g). The structure enters in the nonlinear state for the first mode and linear elastic state for fourth and seventh mode. For X direction, in the same way, the structure enters in nonlinear state for the third mode, and liner elastic state for ninth and tenth mode. According to CSM, when structure enters nonlinear plastic stage, the spectral reduction factor depends on the effective viscous damping of equivalent Single Degree of Freedom (SDOF) system ξ_i.

By repeating this procedure for other intensity levels, the Multimodal Capacity Curve (MCC) has been obtained. By combining the responses considering a range of intensity levels through SRSS rule, both for roof displacements and base shear are determined. Responses are obtained applying CSM on capacity curves obtained considering each modal shape and the response spectrum is scaled from PGA=0.05 g to PGA=0.30 g so for each intensity level an SRSS combined P.P. (P.P.mm) is obtained. Connecting this sequence of P.P. the multimodal capacity curve (MCC) can be defined.

Fig. 11 shows the capacity curves obtained with a standard pushover (load profile proportional to Mode 1: Push-Mode 1) or performing IMPA (connecting the P.P.mm) and NL_RHA (in the plot named IDA-Umax-Vmax); it is underlined that maximum displacements and maximum base shear from a NL_RHA are not contemporary, therefore this curve can be considered the upper bond of the capacity curve (any other pushover based curve will be lower).

Comparing IMPA to standard single mode pushover curve, both for the *Y* and *X* direction, can be observed an increase of base shear that makes the capacity curve from IMPA stiffer than Push-Mode 3 and closer to the IDA curve (the plotted IDA curve has been derived computing the average value of the seven analyzes performed for every incremental step).

The capacity curves are almost the same, it is also another indication that the pure translation along this direction. When the structure enters inelastic state, IMPA would underestimate the base shear compared to IDA.

Comparing IMPA and IDA procedures in terms of Intensity Measure (IM), as a Peak Ground Acceleration (PGA) or Spectral Acceleration at the structure's first mode period ($S_a(T_1$, 5%)), and Damage Measure (DM), as maximum interstory drift or maximum roof displacement, the incremental curves are evidently closer.

Fig. 12 shows the incremental curves obtained with performing IMPA (connecting the P.P.mm in terms of top displacement and PGA) and NL_RHA (in the plot named IDA-Umax, PGA). IDA curve and IMPA curve have almost the same values. Whereas in the *X* direction, the difference increases progressively from PGA 0.1 g and at PGA=0.3 g the difference is about 30%. This can be attribute to the specific characteristic of the building that is characterized by a *X* extended shape that strongly emphasize the difference between a dynamic and static non-adaptive

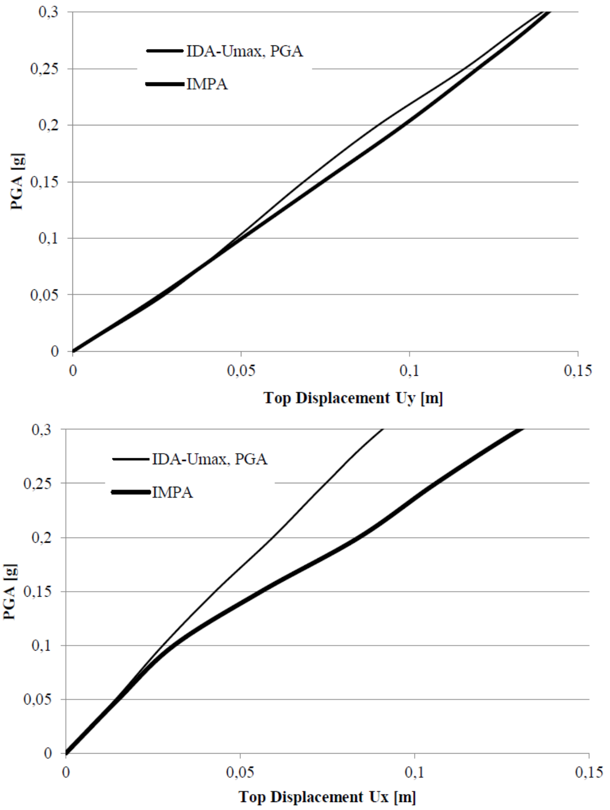


Fig. 12 Incremental curves obtained from two methods: IMPA method and IDA method

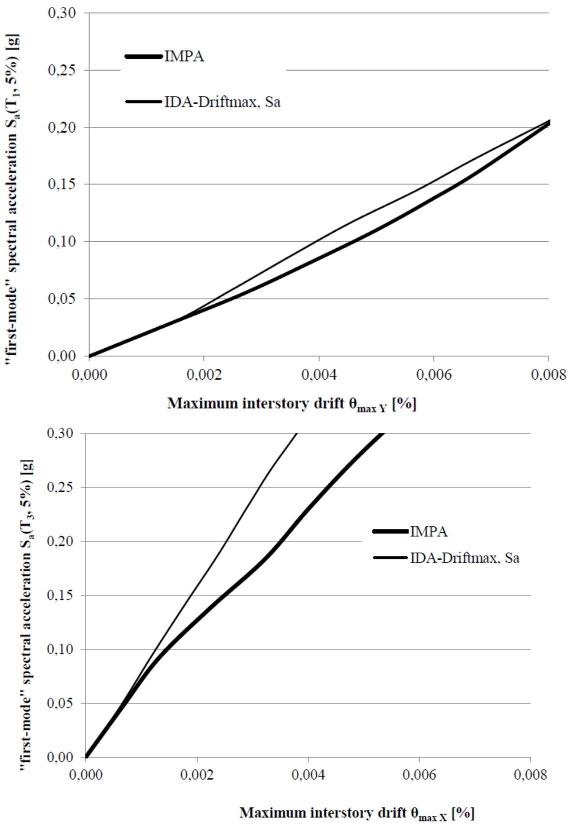


Fig. 13 Incremental curves obtained from two methods: IMPA method and IDA method, in terms of "first mode" spectral acceleration $S_a(T_1, 5\%)$) and maximum interstory drift

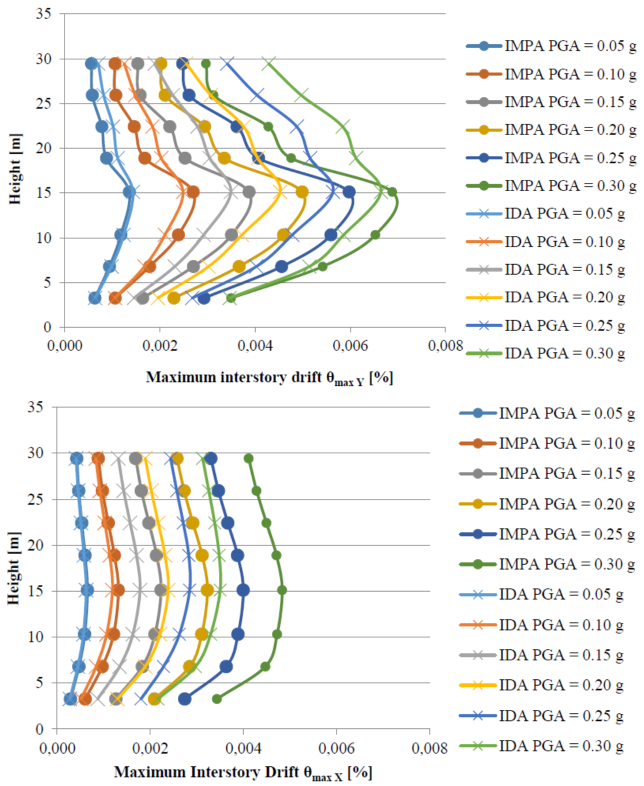


Fig. 14 Structural behavior obtained from IMPA method and IDA method

approach in the plastic range. In every case, IMPA is more accurate than standard pushover but always conservative if compared with IDA.

Fig. 13 shows, in the same way, the incremental curves obtained with performing IMPA (connecting the P.P.mm in terms of maximum interstory drift and *Sa*) and NL_RHA (in the plot named IDA-Driftmax, *Sa*). The use of *Sa*(*T*) as intensity parameter is of course more reliable because is directly related with the specific structure (*T*=fundamental period of the building); PGA can't be considered directly related with structural dynamic response. The incremental curves, IDA and IMPA, are very close: IMPA method overestimates slightly maximum interstory drift values.

The structural behavior obtained from two methods is showed on Fig. 14 the story mechanism of deformation is the same of all PGA's values. In *Y* direction, the soft-story mechanism of deformation is showed both by IMPA and IDA method. However, IMPA method overestimates maximum interstory drift for the story levels lower and underestimates them for the story levels higher.

5. Conclusions

In this work a nonlinear static procedure to evaluate the seismic capacity of buildings has been discussed and applied. The procedure, named Incremental Modal Pushover Analysis (IMPA), has been developed in order to propose a simple approach to analyze the capacity of existing structures following an incremental approach.

The idea comes from the need of define a procedure suitable for existing buildings that can really constitute an operative methodology compatible with professional applications in terms of computational requirements, common professional engineer skills and a clear result interpretation.

In order to perform a comparative critical analysis of IMPA, the study includes an application, on the same case study, of IMPA, pushover and IDA to evaluate the actual advantage and disadvantage of the approach proposed.

According to the application performed, a first consideration can be made in terms of complexity of the procedures. Performing IDA, as the authors did in the case study, the execution of many nonlinear response histories that entail complex and intense computational activity (in this case we used a PC-Intel ® Core ™ i7-4770 CPU @ 3.40 GHz for 104 hours of analysis) is required. In particular, nonlinear dynamic analysis (NL_RHA) requires the preliminary definition of a set of time histories and the execution of the analysis that does not give an immediate and univocal interpretation of results.

In fact, the IDA curve, which represents the structure's nonlinear response to each of the selected and scaled time histories, can be done with different approaches: maximum displacement and maximum base shear, maximum displacement and corresponding base shear, or maximum base shear and corresponding displacement, etc.

Whereas, IMPA is performed, based on the well-known modal pushover analysis (MPA) (Chopra and Goel 2002) and therefore execution and results analysis can be performed in a very short time; the output is a capacity curve (the capacity curve is commonly obtained with a standard pushover analysis) developed also considering the higher modes effect.

In IMPA the MPA is used to estimate the seismic demand and capacity of structures over the entire range of structural responses: the demand curve (the response spectrum) is scaled from lower to higher intensity values starting from the definition of a design response spectrum. Using the capacity spectrum method (CSM), for each single mode a multimodal performance point (P.P.mm) can be defined for each intensity level: the multimodal capacity curve (MCC) is the conjunction of all the multimodal performance points obtained.

The approach was tested by applying it to an existing irregular mid-rise building, characterized by a long rectangular planar shape, emphasising that along the longitudinal direction (where a pure translation is evident) the higher modes exhibit an almost negligible contribution to the total response. Whereas, along the transverse direction, translation and rotation are strongly coupled and therefore the effect of higher modes cannot be neglected.

MPA if compared with NL_RHA can provides a good estimate for roof displacement and underestimates inter story-drift for upper floors; however, if corners are considered control joints, MPA error becomes more significant and therefore cannot be considered reliable for the estimation of drifts at the building's extremities (where torsion implies a non-negligible translation).

Comparing IMPA to standard single mode pushover curve, both for the *Y* and *X* direction, we can observe an increase of base shear that makes the capacity curve from IMPA stiffer and closer to the IDA curve; IMPA and IDA curves are almost the same in the elastic range but, when the structure enters inelastic state, IMPA underestimate base shear if compared to IDA but performs better than pushover.

Comparing IMPA and IDA procedures in terms of Intensity Measure (IM), as a Peak Ground Acceleration (PGA) or Spectral Acceleration at the structure's first mode period ($Sa(T_1$, 5%)), and Damage Measure (DM), as maximum interstory drift or maximum roof displacement, the incremental curves are evidently closer and in both directions IMPA is more accurate than standard pushover and always conservative if compared with IDA.

Furthermore, analyzing the output in terms of a damage index, in this case the interstorey drift distribution, IMPA results are accurate on describing the distribution and concentration of displacements; in terms of estimation of the damage index (maximum interstory drift) the IMPA is more accurate than the standard pushover and always conservative compared to IDA.

Moreover, it is interesting to underline that the comparison between IMPA and IDA agrees with the observation of IDA authors (Vamvatsikos and Cornell 2005) that, discussing the usefulness of deriving from IDA a capacity curve in order to describe structural response of buildings, underline how a capacity curve can be approximate starting from an IDA analysis.

In this paper, the capacity curve has been derived performing IMPA but the cited requirements have been checked and respected. In fact, IMPA and IDA curves are composed of the same number of corresponding and distinguishable segments: the elastic segment of the IMPA naturally coincides with the elastic IDA region, while the yielding and hardening of IMPA corresponds with a change of IDA curve slope.

Concluding, we can consider extremely promising the comparison executed between IMPA and IDA. Therefore the IMPA approach cannot be considered an alternative to IDA but complementary as long as the relations between Static Pushover curve and results of Incremental Dynamic Analysis are satisfied but with a reduced gap compared with what a standard pushover can provide.

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