Incremental modal pushover analysis (IMPA) for bridges

A.V. Bergami*, D. Lavorato, C. Nuti & G. Fiorentino

University of Roma Tre – Department of Architecture Largo Marzi, 10 - 00153 Rome (Italy)

C. Nuti

Visiting Professor, University of Fuzhou, College of Civil Engineering, Fuzhou (China)

**corresponding author: alessandro.bergami@uniroma3.it*

ABSTRACT:
In recent years, many research activities were undertaken to develop a reliable and practical analysis procedure to identify the safety level of existing structure. There are many literatures available on the seismic evaluation procedures of buildings using nonlinear static analysis (pushover) instead of performing, as in the well known incremental dynamic analysis (IDA), complex non-linear dynamic analysis (RHA). Conversely, only a few studies are available in literature regarding seismic evaluation of existing bridges, although bridges are strategic infrastructures in every country. The aim of this work is to carry out a seismic evaluation case study for an existing RC bridge, with reinforcement affected by corrosion, using nonlinear static (pushover) analysis. The work starts from an efficient pushover proposal named Incremental Modal Pushover Analysis (IMPA). This procedure has been developed for higher modes sensitive structures; bridge are commonly characterized by a very high participating mass ratio for the higher modes and therefore the application of IMPA to bridges appears as a promising procedure. The procedure, together with some results of the application on one case study, are presented herein.

1 INTRODUCTION

1.1 *Aim of the work*

Over the past two decades, there has been an increasing attention to Nonlinear Static Procedures (NSPs) for the seismic assessment and evaluation of buildings. More recently, this attention has been extended to bridges. In these methods, a pushover analysis is carried out to estimate the inelastic capacity of the structure as well as its response to different levels of seismic demand. The NSPs have the simplicity of the linear static methods but the accuracy cannot be considered comparable to nonlinear dynamic analyses without a specific discussion for bridges; undoubtedly the pushover procedures have been predominantly developed and tested for buildings and their extrapolation to other structural systems, like bridges, may not be straightforward. This issue, together with the application and discussion of the IMPA procedure (Bergami et al., 2017), an incremental multimodal pushover-based procedure proposed by the same authors for buildings, are addressed in the paper. The proposed method is herein applied to an irregular bridge; inelastic response history analyses RHA, incremental dynamic analyses IDA, standard pushover SPA (load pattern proportional to the dominant mode) and uniform pushover UPA (load pattern proportional to lumped masses) were used as a reference method for comparison.

The cited procedures have been mainly tested for buildings and few results are available regarding their use for the analyses of bridges; for this reason the contribution of this work is addressed to contribute to the discussion, about pushover-based methods for bridges, ongoing in the scientific community (T. Isaković and M. Fischinger, 2006) and in particular to validate the IMPA procedure for bridges. More detailed data and the extension to other case study will be published in a forthcoming work (Bergami et al., in prep.)

As regards the analysis conducted in this work, general results are presented in Section 3 and 4.

1.2 *State of the art of non-linear static analysis*

NSA is a very effective alternative to nonlinear response history analysis RHA, but it is strongly influenced by the choice of lateral force distribution; existing guidelines for load pattern do not cover all possible cases and the specific case of bridges is not considered.

However, pushover analysis can be grossly inaccurate for buildings with irregularities, where the contributions from higher modes are significant. Therefore, the scientific community, in the last two decades, is intensively working on this topic and demonstrated that "traditional" pushover analysis can be an extremely useful tool if some conditions are respected (e.g. a single dominant modal shape, the fundamental mode does not vary significantly in nonlinear stage). More recently, many studies aimed to define innovative methodologies alternative to what is nowadays considered the most reliable approach: the Incremental Dynamic Analysis (IDA).

IDA requires to perform a set of nonlinear response history analysis (RHA), on a detailed numerical model of the structure, for a set of ground motions (GM), each scaled for various intensity levels, selected to cover a wide range of structural responses; this can result as an extremely demanding process (Vamvatsikos D. and Cornell C.A., 2002;2005). According to the scope of this work with IMPA the IDA's RHAs are replaced by a set of modal pushover analyses (MPA) keeping the conceptual simplicity and computational attractiveness of standard NSA procedures (Han and Chopra, 2006). IMPA has been developed for buildings and it is mainly finalized to obtain correlation between seismic demand and a damage index, e.g. P.G.A. Vs base shear or deck drift. The evolution of the 'standard' non-linear static analysis that consider higher modes effects, and in particular the IMPA, suggested a well promising way for an extension of NSA to bridges that are strongly influenced by higher modes. In the last two decades several authors contributed to this topic offering useful indications that have been considered in this work. In Pinho et. al. (2007) a non-linear analysis for continuous multi-span bridge is discussed and a single-run approach is proposed. Muljati and Warnitchai (2007) applied the Modal Pushover Analysis to a bridge with a continuous deck highlighting that in this application the nonlinear and linear range have a similar tendency and therefore the modal pushover can be consider efficient. Kappos et al. (2006, 2010) applies a multimodal pushover procedure, generally similar to that of Chopra and Goel, to some case studies of bridges with continuous deck and compared results with RHA. As highlighted in all this studies to perform the NSA one monitoring point is selected and the selection of this point is a critical issue for MPA of bridges. Natural choices for the monitoring point in a bridge are the deck mass center or the top of the pier nearest to it (Eurocode 8 - Part 2; Paraskeva et al., 2006) if, as usual for monolithic or hinged pier-to-deck connections but not for sliding or flexible connections (e.g. through pot bearings or elastomeric bearings), the displacement of the two is practically the same. According to suggestions from scientific literature (Paraskeva et al, 2006), in bridges with continuous deck, the monitoring point selected does not significantly affect final results and the use of a barycentric point, according to Eurocode 8, is suggested; this advice has been respected in this work being the case study a bridge with continuous deck as detailed in section 3.

2 PROPOSAL OF THE INCREMENTAL MODAL PUSHOVER ANALYSIS FOR BRIDGES

2.1 *Description of the IMPA procedure*

The incremental modal pushover analysis (IMPA) 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. The IMPA procedure is described in a convenient step-by-step form in Bergami et al. (2017) therefore in this paper the procedure is briefly reported and some specifications for bridges are reported. Remembering that the IMPA requires the execution of several MPAs, the seismic demand due to individual terms in the modal expansion of the effective earthquake forces is determined by a nonlinear static analysis using the inertia force distribution s_n for each mode:

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

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

where Γ_n is the n^{th} modal participation factor; M is a diagonal mass matrix of order 2n, including the diagonal submatrices m, ϕ and 1: m is a diagonal matrix with $m_{jj}=m_j$, the mass lumped mass barycenter of the jth pier; φ_n is the n^{th} natural vibration mode of the structure consisting of three subvectors: ϕ_{rn} ,ϕ_{yn} and $\phi_{\theta n}$; the $n\times 1$ vector 1 is equal to unit.

The IMPA procedure can be summarized in the following steps:

1. Compute the natural frequencies, ω_n and modes, ϕ_n for the linear elastic vibration of the bridge. The modal properties of the bridge model are obtained from the linear dynamic modal analysis and the relevant modes of the bridge are selected.
2. Define the seismic demand in term of response spectra (RS) for a defined range of intensity levels.
3. For the intensity level i, represented by peak ground motion acceleration (PGA) the performance point (P.P.) for the selected (predominant) modes can be determined (Figure 1a).
4. Using a combination rule to combine the P.P. corresponding to each mode for each intensity i, the "multimodal performance point" ($P.P._{m,i}$) can be determined (Figure 1b). The $P.P._{m,i}$ is expressed in

terms of monitoring point displacement u_{rmmi}, and corresponding global base shear $V_{b,i,}$ for each intensity level considered: being u_{rni} the modal displacements of the monitoring point: in this paper the tansverse direction has been considered (Figure 2).

Data resulting from MPA application within an identified range of seismic intensity provides all necessary information to estimate the seismic response for different intensity levels. Therefore IMPA allows to develop a multimodal capacity curve (Figure 1c) relating a control parameter with the seismic demand intensity. In the procedure, for each seismic intensity level, the corresponding Performance Point (P.P.) for the multi-degree-of-freedom (MDOF) is determined and the corresponding deformed configuration of the bridge is derived (the deformed configuration of the bridge is the deformed configuration of each monitored station (usually the top of the piers) at the P.P. determined: $u_{r1i},\ldots,u_{rni}$).

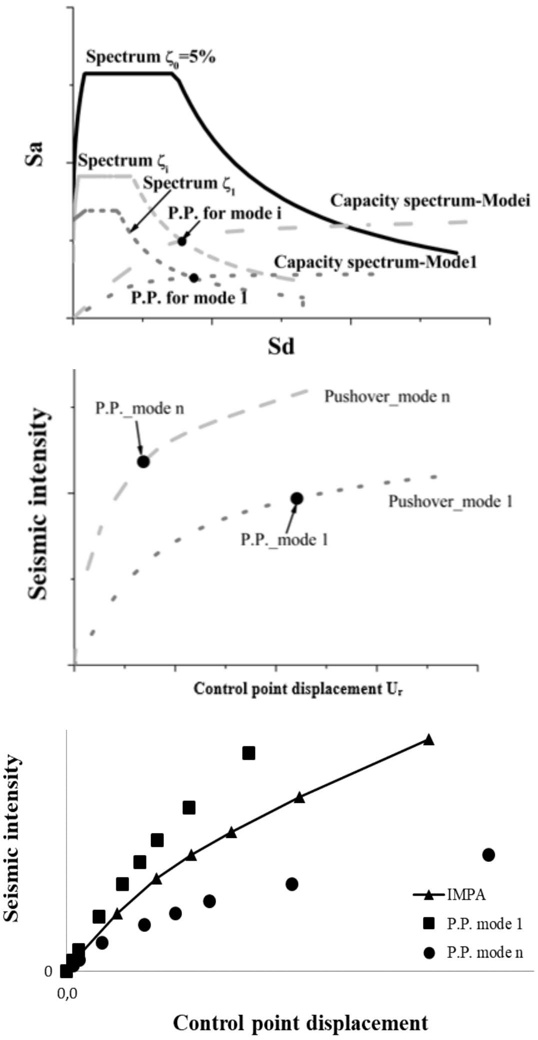

Figure 1. a) Evaluation of the performance points (P.P.) for each capacity curve that belongs to the pushover analysis with the selected load distributions: proportional to Mode 1..Mode n. b) evaluation of the P.P. for each capacity curve: e.g. via C.S.M. c) capacity curve plotted in the u_r-intensity plane (u_r is the displacement of the monitoring point along the investigated direction e.g. the transverse direction).

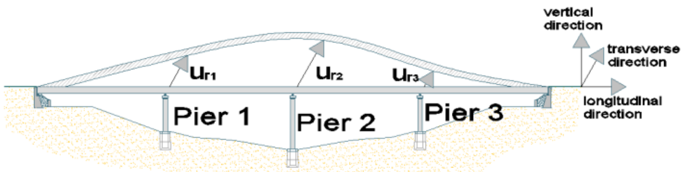

Figure 2. Degree of freedom of the bridge: performing the NSA the displacement u_r of the monitoring point is controlled. In this work u_r is the transversal displacement.

In the application presented in this paper the P.Ps have been determined through the application of the Capacity Spectrum Method (ATC 40) and P.P.s obtained for each significant modal shape have been combined using the Square Root of the Sum of Squares rule (3) to obtain a multimodal performance point ($P.P._{mi}$) for each specific seismic intensity level.

$$u_{rmmi} = ((\sum_{n} u_{rni}^{2})^{1/2}) \tag{3}$$

Performing IMPA the seismic demand is expressed in terms of Response Spectrums (RS) that are defined for all the intensity level; the RS selection can be performed according to different approaches. The RS can be selected according to the design code specification and then it can be linearly scaled to cover the desired intensity range, otherwise the scaling procedure can be performed scaling the return period (Tr) or moreover the RS considered can be derived from a set of ground motions (GM), generated or selected investigating the local seismicity, for example considering the spectrum of each GM (or the median spectrum of each set) for each intensity level. In this paper, in the applications described in section 4, the median spectrum of a set of GM was used. In the application presented herein the RS have been linearly scaled using a scale factor (SF) from 0.5 to 2.0.

3 APPLICATION TO A CASE STUDY

3.1 *Description of the bridge*

The case study selected is a straight bridge with four equal spans (span length 50m), total length qual to 200m (Figure 3) and a continuous deck.

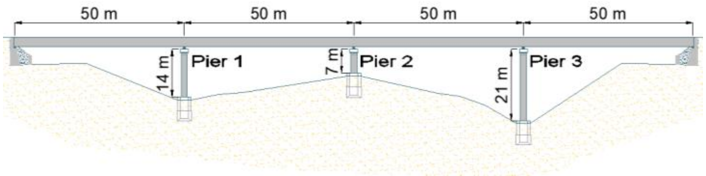

Figure 3. Case study: layout of the bridge [m].

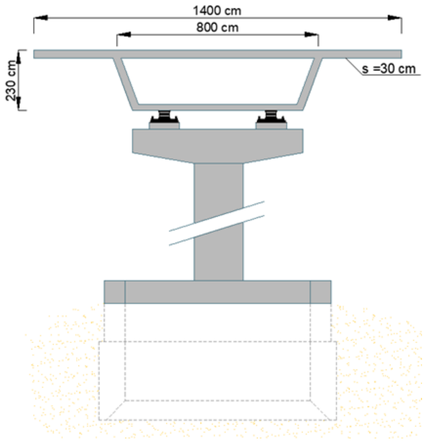

Figure 4. Case study: geometry of the deck [cm].

The deck consists in a 14m wide pre-cast concrete caisson supported by piers consisting of cylindrical cross-section of 2.5m diameter; the height of the piers (P) is variable between 7 and 21m (P_1=14m, P_2=7m, P_3=21 m).

The deck rests on its two abutments through bearings (movement in the longitudinal direction is allowed at the abutments, but transverse displacements are restrained) and it is supported on the concrete pier-head through bearings locked in the transverse direction.

The details of this bridge are described in Figure 4 and 5; the design concrete class used was C20/25 (characteristic compressive cylinder strength f_{ck}=20MPa) while B450C steel (design characteristic yield strength f_{yk} = 450MPa) reinforcement was used throughout the structure. The pile sections are considered retrofitted with new materials after being damaged by corrosion (Lavorato et al., 2018).

The bridge has been designed according to Eurocode 8: the design peak ground acceleration was 0.35g and the behavior fator "*q*" was 3.0 (irregular bridge – EC8). The design loads are summarized in Table 1.

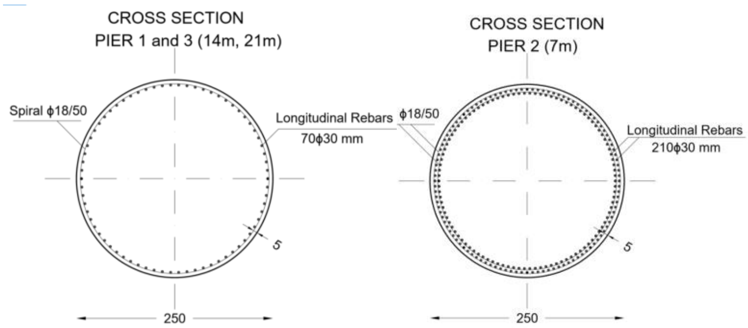

Figure 5. Case study: piles cross section.

Table 1. Design loads (characteristic value)

Load	[kN/m^2]
Dead load	from elements
SuperDead load	200.0
Live load	54.5

The response of the bridge model is estimated through the employment of non-linear static and dynamic analysis (SPA, UPA, MPA and RHA) and incremental static and dynamic analysis (ISPA, IUPA, IMPA and IDA). The dynamic analyses have been performed adopting a set of 7 GM generated from the response spectra used to design the bridge; from the set of ground motions the median response spectra (RSm) have been defined.

To generate the set of GM a spectral matching for a selected period range has been adopted; this approach is considered sufficient to describe the seismic behavior of an individual structure.

Therefore the RS has been defined according to the Italian technical code (NTC 2018) being: the location is Reggio Calabria (a region of high seismic hazard in the south of Italy), the soil is type B according to EC8 classification (very dense sand or gravel or very stiff clay, 360≤ V_{s30} ≤800) and the return period considered is T_r = 949 years (life safety limit state: P_{VR}=10%; a_g=0.35g; T_B=0.172s, T_C=0.516s, T_D=3.035s; F_0=2.464; S_T=1.0). Given this target spectrum, with the software Rexel (Iervolino et al. 2010), a set of 7 unscaled GM, which average spectrum is compatible with the target one, and considering the minimum dispersion of individual spectra, have been selected: the average response spectrum matches the target spectrum at a specified period range that includes all the periods considered relevant (modes with participating mass > 1% along the transversal direction of the bridge).

The 7 ground motions selected are listed in Table 2, and in Figure 6 are shown the 5% damped response spectra of the transverse component of the ground motions; the median spectrum is taken as the design spectrum for purposes of evaluating the IMPA procedure. To perform the incremental procedures the 7 ground motions selected were scaled by a factor from 0.5 to 2 obtaining the set of ground motions for the IDA; those ground motions were used to obtain the median response spectra used in IMPA.

Table 2: list of the selected ground motion

Earthquake Name	Earthquake ID	Waveform ID	PGA (g)
South Iceland, (2000)	1635	4674-xa	3.311
South Iceland, (2000)	1635	4674-ya	3.311
Bingol, Turkey (2003)	2309	7142-xa	2.918
Bingol, Turkey (2003)	2309	7142-ya	2.918
South Iceland, (2000)	2142	6349-xa	0.822
South Iceland, (2000)	2142	6332-ya	5.570
South Iceland, (2000)	1635	6277-ya	5.083

Following Eurocode 8 recommendations, the independent damage parameter selected as reference is the displacement of the node at the center of mass of each pile: each level of intensity (corresponding to a given lateral load level or to a given input motion amplitude) is represented by the deck drift.

Results are presented in terms of the bridge capacity curve, i.e. configuration of the deck drift profile, capacity curves, base shear Vs seismic intensity.

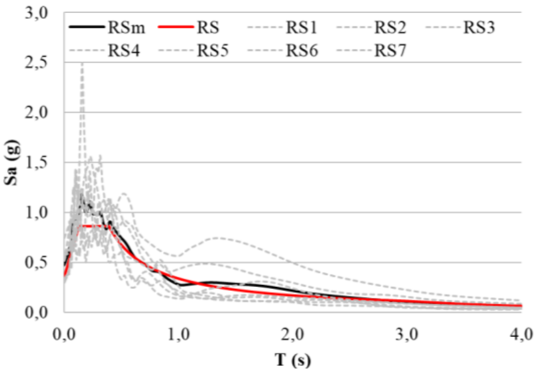

Figure 6. Individual response spectra RSi, (ζ = 5%) for the 7 unscaled ground motions and their target (RS) and median response spectrum (RSm).

3.2 *Computational model*

The bridge was modelled using the software SAP2000 NL, v. 21; the 3D numerical model is shown in Figure 7.

The model must ideally represent the mass distribution, strength, stiffness and deformability. Piers and girders supporting deck are modelled by 3D frame elements. The girder-pier joints are modelled by giving end offsets to the frame elements, to obtain the bending moments and forces at the beam and column faces. The girder-pier joints are assumed to be rigid and the pier end at foundation was considered as fixed. All the pier elements are modelled with nonlinear properties at the possible yield locations. In the present study, a lumped plasticity approach is considered for modelling nonlinearity; the plastic hinges are assumed to be concentrated at a specific portion (plastic regions according to Eurocode 8) of the pier; hinges have been modelled with fiber (P-M2-M3) hinges. Fiber hinges in this study are defined by moment-rotation curves calculated using a fiber-based model of the cross-section according to the reinforcement details at the hinge locations.

Figure 7. Case study – numerical model (SAP2000 NL).

3.3 *Modal properties*

Modal properties of the bridge model were obtained from the linear dynamic modal analysis. Table 3 and Figure 8 show the details of the relevant modes of the bridge in transverse direction. The participating mass ratio of the first three relevant modes (mass ratio ≥3%) are respectively 16.9%, 71.3% and 4.5%; the cumulative mass participating ratio for first three modes is 92.7% (the participating mass ratio of the other modes was less than 1%). Therefore, the higher mode participation in the response of the bridge is significant and the first three modal shapes in the transverse direction must be considered; the dominant mode is mode 3 (T3=0.53s).

Table 3: Case Study - Modal properties

Mode	**Period**	**Participating Mass**
N°	Sec	%
1	0.65	16.9
3	0.53	71.3
4	0.13	4.5

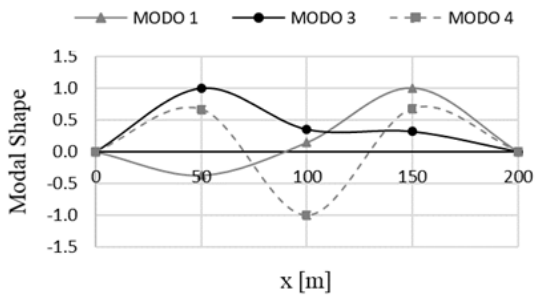

Figure 8. Case study - Modal shape (transv. direction).

4 BRIDGE ANALYSIS

The response of the bridge model is estimated through the employment of non-linear static and dynamic analyses (SPA, UPA, MPA and RHA) and incremental static and dynamic analysis (ISPA, IUPA, IMPA and IDA). The dynamic analyses have been performed adopting a set of 7 ground motions (GM) generated from the response spectra (RS) used to design the bridge (Figure 5). Comparing results from NSA and RHA was observed that, for the case study analyzed, MPA is a well-performing approach but better results can be achieved considering an envelope of the response derived from MPA and UPA (MPA-UPAenv). Results are reported considering the following incremental range: earthquake intensity from PGA=0.175 g to PGA=0.7g being PGA=0.35g the design level. All relevant modes (mode 1, 3 and 4) have been taken into account performing each MPA and therefore in the IMPA procedure. In Figure 9, for each intensity step, the curves MPA-UPAenv are compared with the RHA results. As can be observed also from Figure 9 and 10, MPA coincides quite well with the results of

RHA up to the design intensity (from PGA 0.175g to PGA 0.35g) but, to achieve a better estimation in the case discussed herein, the envelope of MPA and UPA should be considered. For higher intensities (scale factor 1.5 and 2.0: PGA over 0.525g) a good estimation of displacement can be observed for the control joint at Pier 1 whereas, at Pier 2 and 3, the estimation became inefficient; crucial is the occurrence of the hinge in Pier 2, first, and Pier 1, after, because at this stage the mode shapes are drastically changed as well as their sequence; in Figure 9 can be observed how the first plastic hinges are in Pier 1 and 2 (at a PGA of 0.175g that is lower than the design intensity), whereas in Pier 3 the first hinge emerges only at a very high intensity level (greater than PGA 0.525g). The predominant (translational) mode becomes the forth one (in the initial state it is the third mode for relevance) and the asymmetric mode becomes the second one; the importance of higher modes is significantly reduced and the response becomes translational.

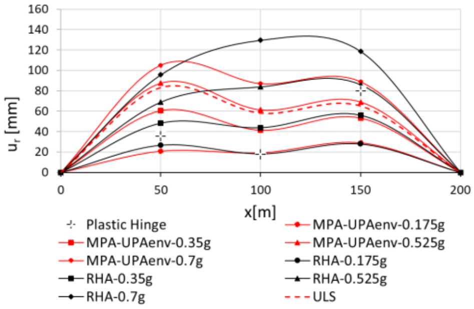

Figure 9. Case study – u_r is the transverse displacement of each control joint

It should be recognized that none of the pushover methods, even the MPA, can reflect these sudden and substantial modification. It should be observed that a limit state (ultimate limit state at the base of Pier2) is reached when the deformed shape (ULS in Figure 9) is close to PGA 0.525g: therefore for the scopes of the procedure is useless to investigate higher intensities. Results of all the methods differ (SPA even qualitatively) from the results of the RHA (Figure 10 and 11). At the Pier 2-3 side, the results depend on the level of earthquake intensity. In the case of lower ductility demand (PGA < 0,35 g) pushover procedures well-estimate the response, while in the region of higher ductility demand they under estimate results of the RHA. At the Pier 1 side, results are less influenced by the level of earthquake intensity and over-estimate the response results of the RHA.

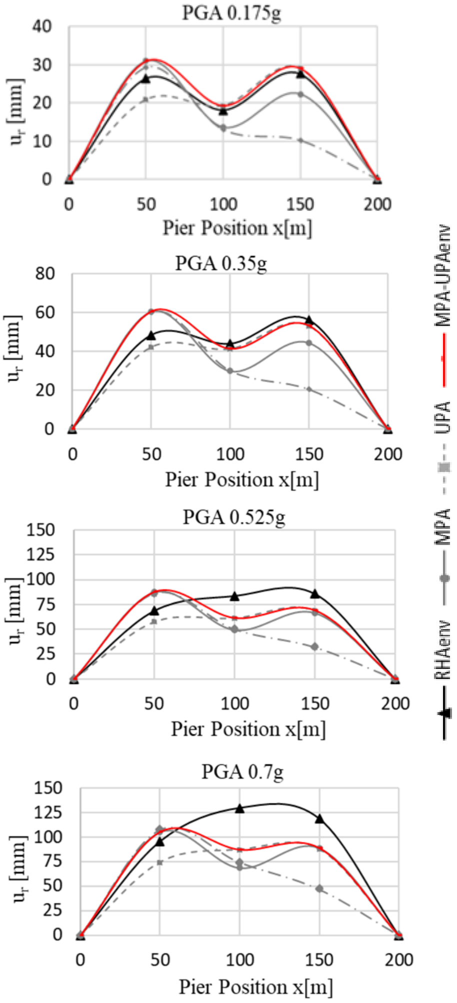

Figure 10. Case study – u_r is the transverse displacement of each control joint determined according to different procedures and for all the PGA considered.

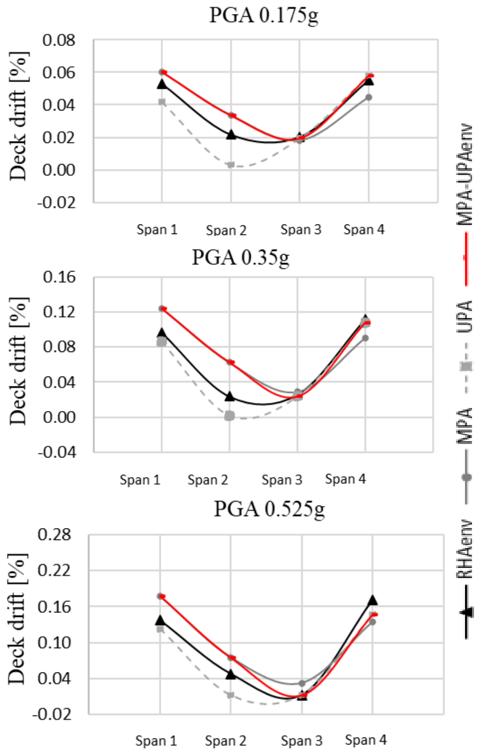

Figure 11. Case study – Deck drift for different intensity level (sym. from Figure 7).

In terms of deck drift (Figure 11), for the relevant range of intensities (PGA<0.525g) previously indicated, the pushover procedures are all well performing and the MPA-UPAenv results the better solution being the most conservative up to the design intensity and well performing for higher intensities.

The capacity curves of the structure have been determined performing both the incremental multi-modal pushover analysis IMPA and the incremental dynamic analysis IDA; the curves are plotted in Figure 12 considering different monitoring points: P1, P2 or P3. As already mentioned, according to both EC8 and the scientific literature, for this typology of bridges the recommend choice is a monitoring point close to the deck barycentric position; therefore P2 is assumed herein as the best choice.

Figure 12 demonstrates that the capacity curve, plotted considering different monitoring points, provides similar results in all cases; the IDA and IMPA curves are comparable with a moderate difference in the elastic-plastic transition.

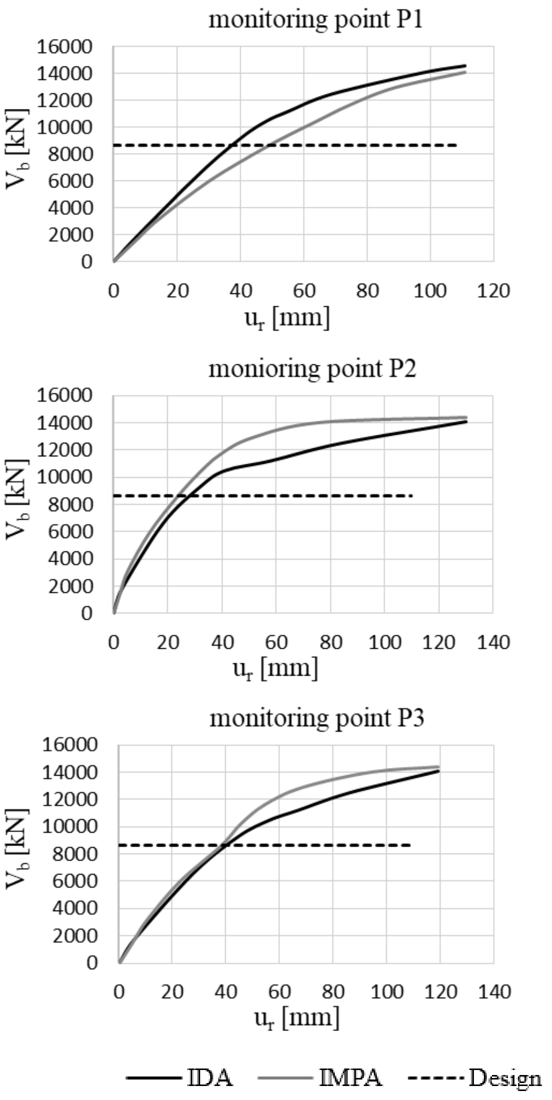

Figure 12. Case study – Comparison of the capacity curve derived with IMPA and IDA controlling three different monitoring point: P1 (Top of pier 1), P2 (Top of pier 2), P3 (Top of pier 3).

Comparing IMPA with a "traditional" pushover approach (Figure 13) based on the incremental SPA (ISPA), the reliability of IMPA is confirmed. The "real" behavior plotted with IDA is included between IMPA and IUPA; IMPA is confirmed as conservative.

In conclusion, also investigating the relation between seismic intensity and shear action, the IMPA curve can be considered well performing, also in terms of base shear (Figure14), if compared with other pushover procedures.

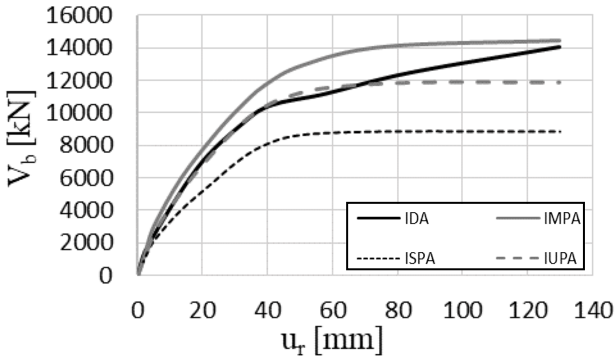

Figure 13. Case study – capacity curve from the application of IDA, IMPA, IUPA o ISPA (monitoring point: P2).

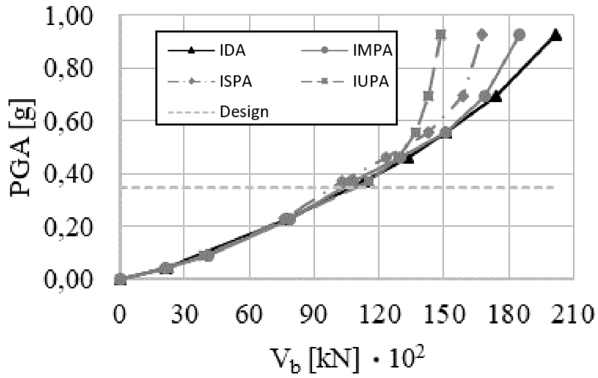

Figure 14. Case study – base shear Vs seismic intensity with IDA, IMPA, IUPA, ISPA (Design p.g.a is 0.35g).

5 CONCLUSIONS

Many research activities were undertaken to develop a reliable and practical analysis procedure to identify the safety level of existing structures and in particular, more recently, the applicability of pushover-based procedure to bridges is widely discussed.

Nonlinear static analysis (pushover), and in particular the multimodal procedures, seems to be a well promising approach for bridges in order to avoid performing, as in the well known incremental dynamic analysis (IDA), complex non-linear dynamic analysis (RHA).

In this work the applicability of a procedure named IMPA, already developed and applied on buildings, has been tested and discussed through the application on a case study of an existing RC bridge subjected to corrosion. In the work the Incremental Modal Pushover Analysis (IMPA) is discussed with reference to the peculiarities of bridges: high sensitivity to higher modes, complex identification of a monitoring point. The results obtained on the case study demonstrate that the procedure is excellently performing up to the bridge design intensity level and well performing up to a seismic intensity corresponding to the ultimate limit state. IMPA, conservation the simplicity of a pushover method, if compared with “standard” pushover procedures results better performing. Analyzing results from the application discussed herein, a combined use of IMPA and IUPA (IMPA and IUPA envelope) seems to be the most conservative strategy to be suggested; this analysis should be supported with other case study.

The activity presented in this paper is the preliminary stage of a more extensive study that will imply several applications on bridges with different configurations.

Therefore these preliminary results can be considered encouraging for this study and confirming IMPA as a well promising procedure for bridges.

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