Non linear static analysis of bridges with a modal pushover based procedure

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ABSTRACT: In the last two decades many scientific works on the seismic evaluation procedures for buildings, using nonlinear static analysis (pushover), have been published. Differently there is no much effort available in literature for seismic evaluation of existing bridges, although bridges are strategic infrastructures for every country, with pushover. The aim of the present work is to asses a procedure for existing bridges, using a nonlinear static analysis; the study extends a method proposed for buildings by the same authors (Bergami et al., 2017), the incremental modal pushover analysis (IMPA), that contemplates the two following important aspects that are relevant in the field of seismic analysis of bridges: the intensity of the demand and structural response are correlated; bridges are frequently higher modes sensitive. For all these reasons IMPA for bridges appears as a promising procedure. In this paper the procedure, toghether with some results of the application on two case study, are presented and discussed.

KEYWORDS: masonry infilled frames, dissipative braces, seismic assesment, seismic sequence

1 INTRODUCTION

Many studies, in the last decade, aimed to define innovative methodologies alternative to what is nowadays considered the most reliable approach for seismic assessment of structures: the Incremental Dynamic Analysis (IDA). IDA requires to perform a set of nonlinear response history analysis and therefore this is an extremely demanding process (Vamvatsikos D. and Cornell C.A., 2002;2005). The same authors, few years ago dealing with building structures, developed the incremental modal pushover analysis (Bergami et al., 2017); with IMPA the IDA's RHAs are replaced by a set of modal pushover analyses MPA (Han and Chopra, 2006). Moreover, with the scope of develop the use of pushover-based procedures for a wider range of applications (different structural types).

Many recent studies are oriented to extend the use of non linear static procedures (NSPs), commonly named "pushover", for the seismic assessment and evaluation of bridges although have been developed and tested mainly on buildings. This idea belongs from both the simplicity and the accuracy of this approaches. Certainly NSP cannot be considered comparable, in terms of accuracy, to nonlinear dynamic analyses without a specialization of this methodology for bridges (T. Isaković and M. Fischinger, 2006). Several authors contributed to this topic and many studies have been oriented to bridges (Pinho et. al., 2007; Kappos et al., 2006-2010; Paraskeva et al., 2006).

Therefore, in this work a pushover based procedure, applicable to bridge, is discussed; the procedure, named IMPAβ (Bergami et al., 2020), is alternative to complex analysis based on the use of inelastic response history analyses (RHA) such as the incremental dynamic analyses (IDA).

The procedure and its fundamentals are herein described together with two case study of bridges: one regular and one irregular.

1.1 *The incremental modal pushover analysis for bridges: IMPAβ*

The IMPAβ procedure, presented and discussed in Bergami et al. (2020) requires the execution of several modal pushover analyses (MPA) and uniform pushover analyses (UPA): one for each incremental step considered.

The procedure requires the execution of the following steps:

1a) Modal analysis - compute the natural frequencies, ω_n and modes, ϕ_n for the linear elastic vibration of the bridge;

1b) the seismic demand is represented in terms of response spectra (RS). The RS are scaled, according to different criteria (e.g. scaling the returning period or the p.g.a.) in order to have a set of RSs for the entire range of intensities that will be explored.

1c) perform a pushover analysis according to a uniform loading pattern (UPA)
2) for each intensity and for each capacity curve (using the MPA several pushover are executed: one for each relevant mode) a P.P. is derived; the P.P.s (one for each modal shape considered) of each intensity *i*, are combined to obtain the "multimodal performance point" ($P.P._{m,i}$) being $P.P._{m,i}$ in terms of displacement u_{rmmi} of a selected point, named "monitoring point", and base shear $V_{b,i,}$: In this work the P.Ps have been according to the Capacity Spectrum Method (ATC 40) and P.P.s have been combined with Square Root of the Sum of Squares rule to obtain the $P.P._{mi}$

$$u_{rmmi} = ((\sum_n u_{rni}^2)^{1/2}) \qquad (3)$$

3) to each intensity level *i* (corresponding to a peak ground motion acceleration) a performance point (P.P.), considering both MPA*1 and UPA can be determined using e.g. the CSM. The envelope from MPA and UPA is considered.

Performing this procedure within the identified range of seismic intensity, IMPA allows to develop a multimodal capacity curve relating a control parameter with the seismic demand intensity. With IMPAβ the selection of the RS can be executed according to many criteria. In this work, for the applications discussed, the median spectrum of a set of GM, compatible with the design RS according to Eurocode 8, was used. The RS has been linearly scaled using a scale factor (SF) from 0.5 to 2.0.

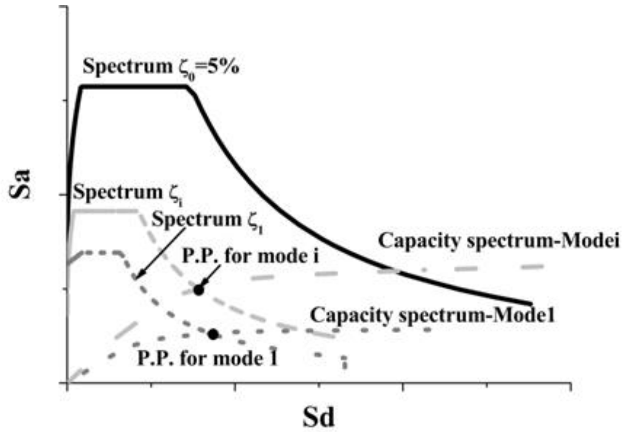

Figure 1. Evaluation of the performance points (P.P.) for each capacity curve that belongs to the pushover analysis with the selected load distributions: proportional to Mode 1..Mode n.

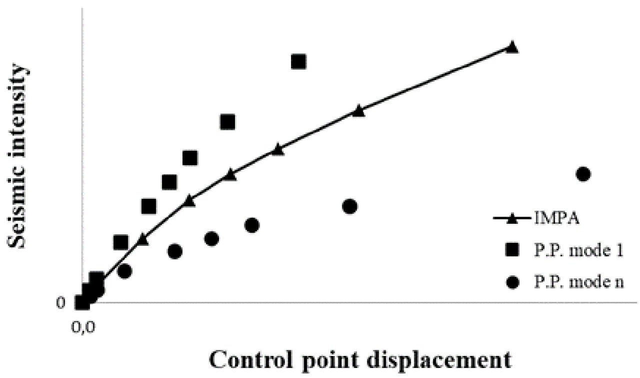

Figure 2. capacity curve obtained combining the P.P.m.i

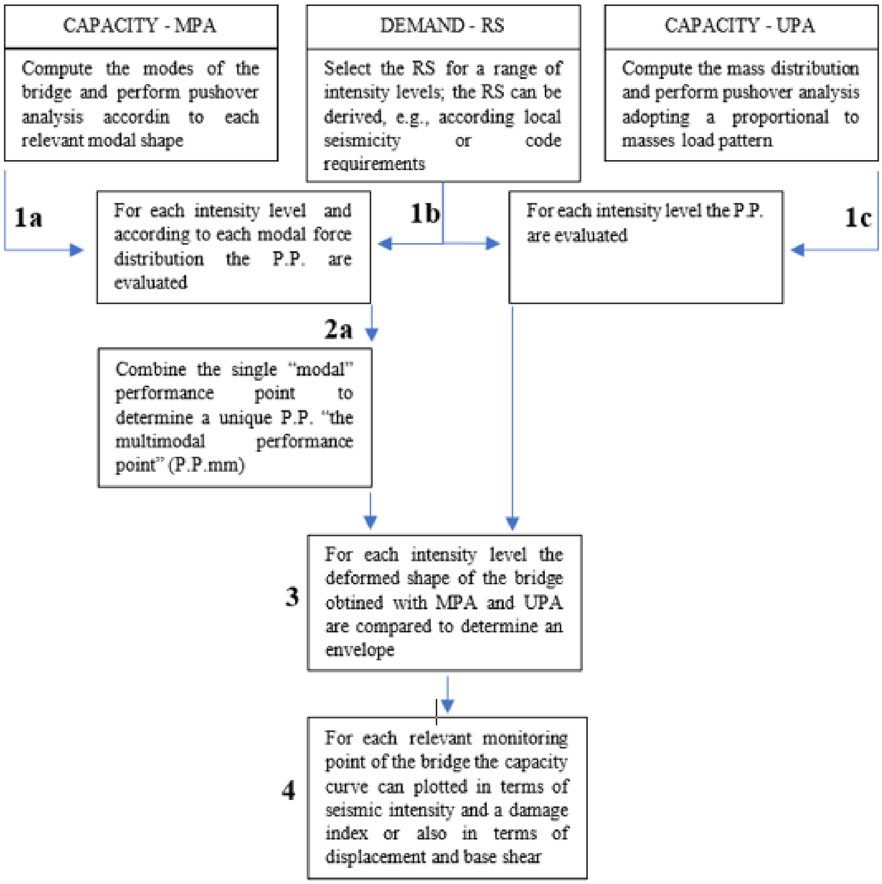

Figure 3. Flowchart of IMPAβ procedure

2 CASE STUDY: REGULAR AND IRREGULAR BRIDGE

The case study selected are two bridges: one regular (RB) and the other irregular (IB) according to the definition given by EC8 and NTC 2018. Both RB and IB have four equal spans of 50m each and a continuous deck. The bridge has been designed according to Eurocode 8 (EC8): the design peak ground acceleration was 0.35g (Reggio Calabria, Italy).
For the RB, the piers are unequal in height: the highest pier is the middle one (P2=21m), and the others at the extremities (P1=14m and P3=14m).
For the IB, the piers are unequal in height: the shortest pier is located in the middle (P1=7m), and the others at the extremities (P1=14m and P3=21m).

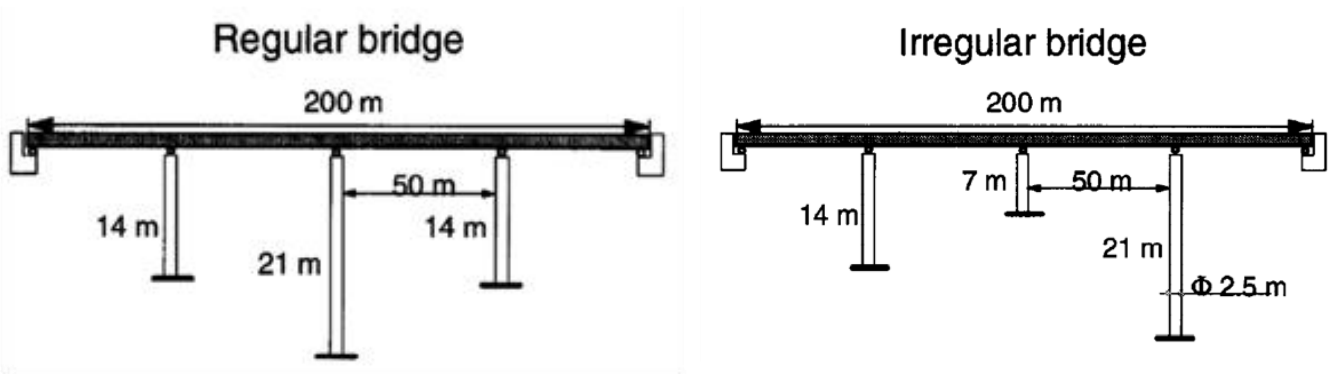

Figure 4. Case study (RB and IB): layout of the bridges [m]

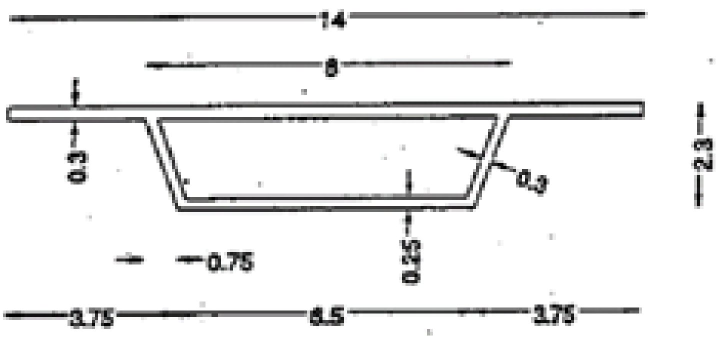

Figure 5. Case study (RB and IB): geometry of the deck [m]

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 seismic action has been defined considering the location in Reggio Calabria (Italy) and, according to NTC 2018, structural class = IV, soil type B, and a behavior factor that, as indicated in NTC 2018 [7.3.1] is:

$$q=q_0*K_R \tag{1}$$

with:

- q_0 is the maximum value of the behavior factor, that depends from the ductility class selected. A in this case (ductile structure);
- K_R is a reduction factor that is 1 in this case.

The load analysis of the bridges is summarized in Table 1.

Table 1. Design loads (characteristic value)

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

According to NTC 2018 (Table 7.9.1) $q_0 = 3.5\ \lambda$ with λ defined according to NTC 2018 [7.9.2.1] as a function of α=L/H

with:

- L is the distance of the plastic hinge from the section wherein bending moment is zero;
- H is the dimension of the inflection plane of the plastic hinge;

Therefore in this specific case $q_0 = 3.5\ \lambda = 3$ and therefore $q= q_0*K_R = 3$

The length (L_h) of the plastic zone, used to realize the non-linear model, is defined in NTC 2018 [7.9.6.2] as the maximum of two:

- the depth (D) of the pier section within the plane of bending (perpendicular to the axis of rotation of the hinge);
- the distance ($L_{\Delta M20\%}$) from the point of maximum moment to the point where the design moment is less than 80% of the value of the maximum moment.

The dimension of the plastic hinges is detailed in Table 2.

Table 2: plastic hinge length

Pier	dimension	[m]
Pier 1= 14m	D	2.5
	$L_{\Delta M20\%}$	2.8
	L_h	2.8
Pier 2= 7m	D	2.5
	$L_{\Delta M20\%}$	1.5
	L_h	2.5
Pier 3= 21m	D	2.5
	$L_{\Delta M20\%}$	4.2
	L_h	4.2

The bridge was modelled using the software SAP2000 NL, v. 21; the 3D numerical model is shown in Figure 6.

The model ideally represent the mass distribution, strength, stiffness and deformability. The piers are modelled as 3D frame elements (circular cross section with D=2.5m) and the deck, realized as prefabricated concrete-girder (concrete class C32/40) is modelled with “section designer”. As permanent action only the self-weight of the elements has been considered together with the vehicle variable load according to NTC 2018 (chapter 5).

The deck is modelled physically, therefore it’s dead load is automatically considered; consequently the corresponding distributed mass is considered for modal analysis. To correctly model the connection between the deck and the piers, shear and moment releases have been introduced on the top of the piers that, at the lower extremity, are considered as fixed. All the pier elements are modelled with non-linear properties at the possible yield locations defined as to the plastic region according to NTC 2018. Pier hinges have been modelled as fiber non-linear hinges P-M2-M3. For the piers the design concrete class used was C20/25 (characteristic compressive cylinder strength fck=20MPa) while B450C steel (design characteristic yield strength fyk = 450MPa) reinforcement was used throughout the structure.

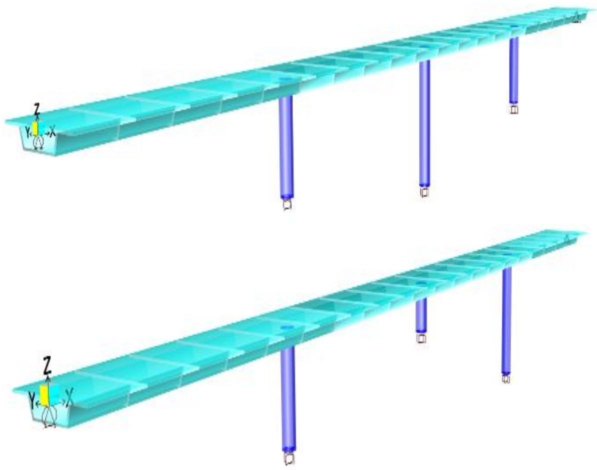

Figure 6. Case study – RB (top), IB (bottom) - numerical models (SAP2000 NL)

The response of the bridge model is estimated through the employment of non-linear static analyses (modal pushover MPA and incremental dynamic analysis IMPA) and non-linear and incremental dynamic analysis (RHA and IDA).

To perform the cited analysis the local seismicity has been defined according to a set of 7 of ground motions GM generated from the response spectra used to design the bridge: therefore the RS has been defined according to the NTC 2018 being:

- Location: Reggio Calabria
- soil type: B

- return period considered: 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 (RSm) 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 3; to perform the incremental procedures the 7 ground motions selected, for the RHAs, and the corresponding RSm, for the MPA, were scaled by a factor from 0.5 to 2.

Table 3: 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

3 ANALYSES RESULTS

The response of the bridge model is estimated through the employment of non-linears static (MPA, UPA) and dynamic analyses (RHA) and incremental static (IMPAβ) and dynamic analysis (IDA).

The dynamic analyses have been performed adopting a set of 7 ground motions (GM) generated from the response spectra (RS) used to design both the bridges. Comparing results from MPA and RHA was observed that, for the case study analyzed, MPA is a well-performing approach if compared with a standard pushover procedure (SPA).

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 (RB: mode 2 and mode 4; IB: mode 1, 3 and 4) have been taken into account performing each MPA and therefore in the IMPAβ procedure (Table 4 and 5; Figure 7 and 8).

Table 4: Regular Bridge - Modal properties

Mode	Period	Participating Mass
N°	Sec	%
2	1,02	78,0
4	0,33	12,0

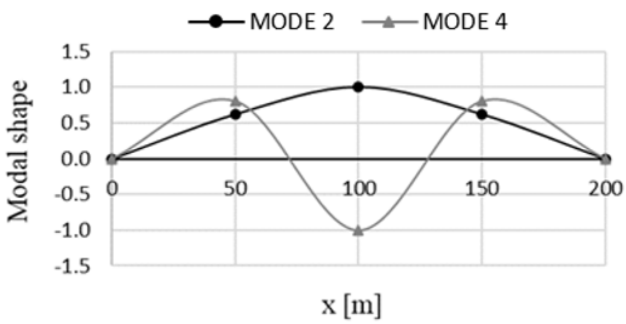

Figure 7. Modal analysis of the Irregular bridge: relevant modes

Table 5: Irregular Bridge - Modal properties

Mode	**Period**	**Participating Mass**
N°	**Sec**	**%**
1	0,65	16,9
3	0,53	71,3
4	0,13	4,5

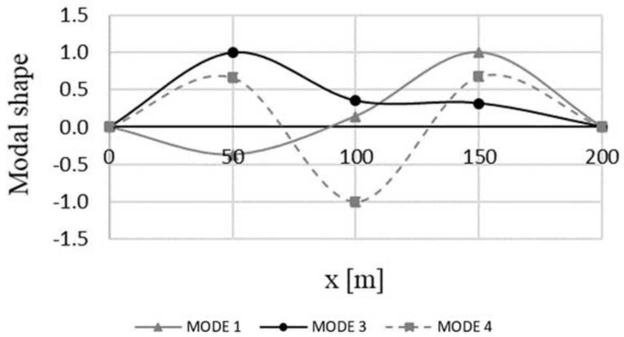

Figure 8. Modal analysis of the Irregular bridge: relevant modes

In Figure 9 and 10, for each intensity step, the curves envelope of the MPA and UPA results (MPA-UPAenv), considered in the IMPAβ, are compared with the RHA results.

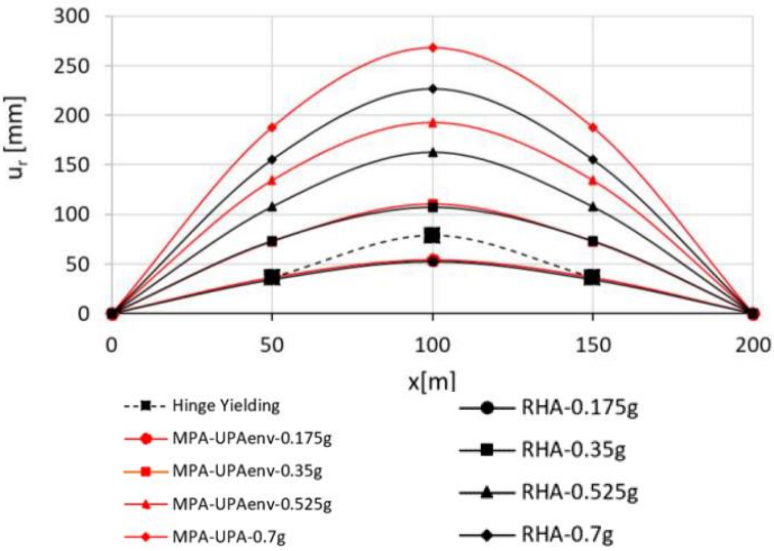

Figure 9. Deformed shape of the bridge RB – u_r is the transverse displacement of each control

For the RB (for brevity only few results are presented being the IB the most interesting case study; a full description of RB is provided in Bergami et al., 2020), the use of the MPA rather than that of a SPA (single mode pushover analysis) appears to be of modest importance (the plots are overlapped) and this assessment is also confirmed by analyzing the stresses and curvatures at the plastic hinges of the piers.

The standard pushover, considering the dominant mode only, works well for the regular bridge and therefore the benefit of performing a modal pushover, even if it exists, is substantially negligible. Figure 11 demonstrates how the IMPAβ can be considered well performing if compared with IDA.

For the IB, as can be observed also from Figure 10, 12, 13 and 14, 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 10 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. It should be recognized that none of the pushover methods, even the MPA, can reflect these sudden and substantial modification.

Results of all the methods differ (SPA even qualitatively) from the results of the RHA (Figure 12 and 13). 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.

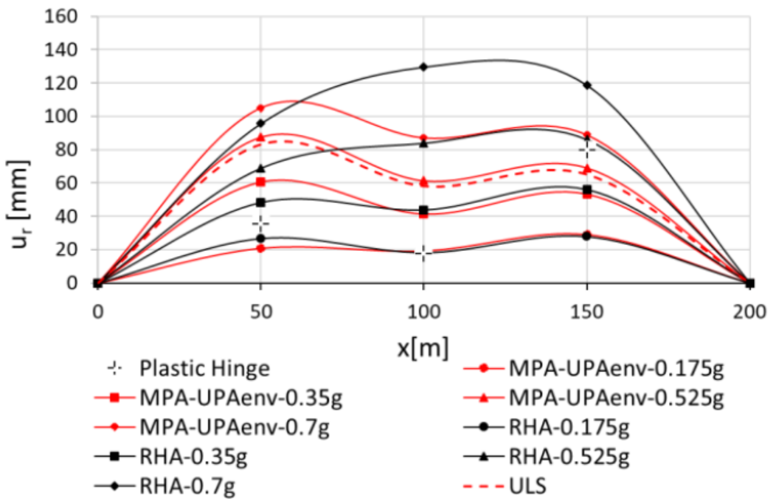

Figure 10. Deformed shape of the bridge IB – u_r is the transverse displacement of each control joint

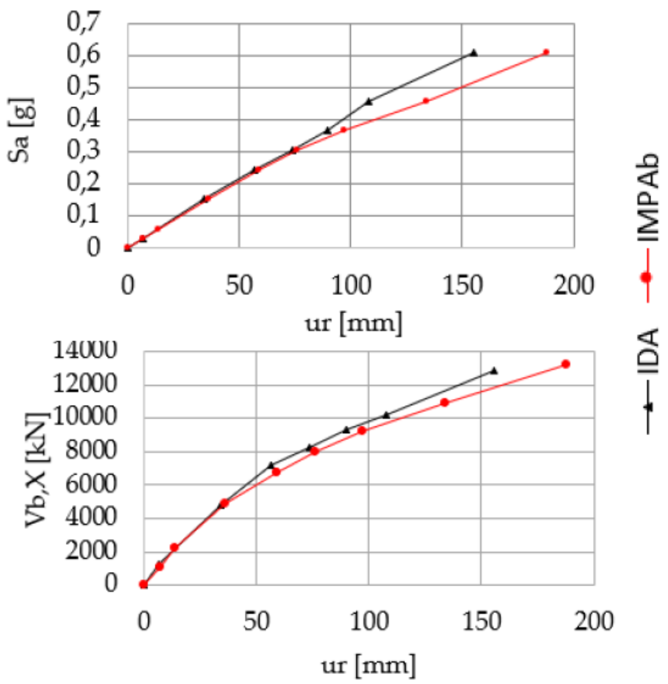

Figure 11. RB – capacity curves derived with IDA (maximum registered values of ur and Vb,x) and IMPAβ (the design PGA is 0.35g corresponding to a transversal base shear of $V_{b,x}$ ~ 5200 kN)

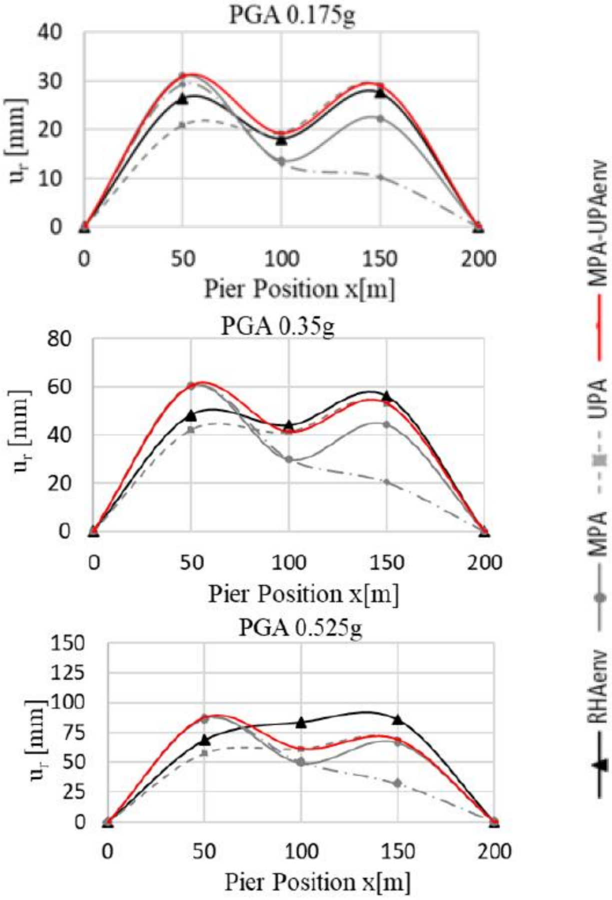

Figure 12. IB – u_r is the transverse displacement of each control joint determined according to different procedures and for all the PGA considered

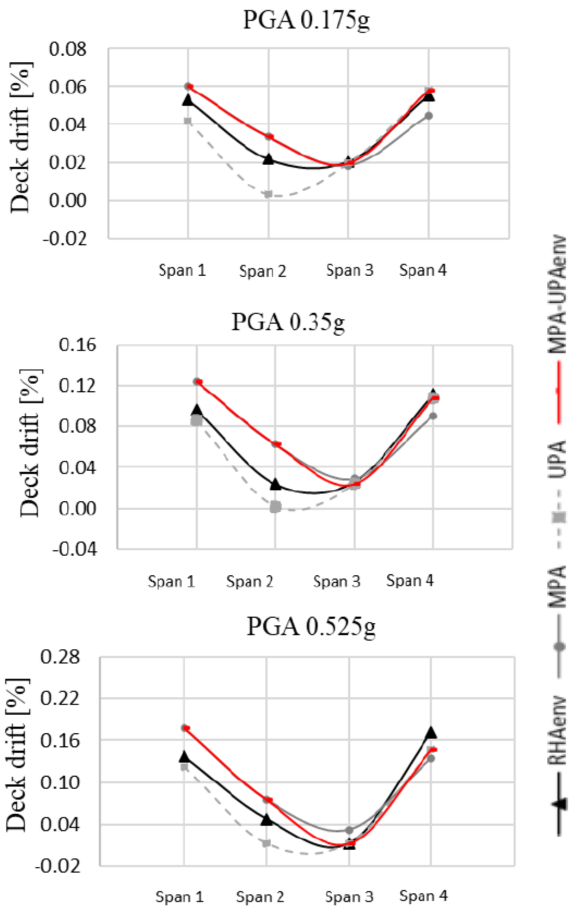

Figure 13 IB – Deck drift for different intensity level (sym. from Figure 7)

In terms of deck drift (Figure 14), for the relevant range of intensities (PGA<0.525g) previously indicated, the pushover procedures are all well performing; the MPA-UPAenv results the better solution being the most conservative up to the design intensity and well performing for higher intensities.
From the application on the IB, results (expressed in terms of capacity curve or seismic intensity vs deck displacement) obtained performing MPA only can be considered well performing up to the design intensity of the bridge whereas, for higher intensities and therefore after the activation of one or more plastic hinges, the MPA became inaccurate. Differently, if a uniform loading profile (UPA) is additionally performed and the MPA and UPA are enveloped, for each intensity level, results strongly improve: the curves derived with IMPA (incremental modal pushover analysis based on MPA) and IUPA (incremental pushover analysis based on UPA) include, as an upper and a lower bounds, the curves derived with IDA.
Therefore the IMPAβ, envelope of IMPA and IUPA, if compared with IDA, is a well performing procedure and in particular excellently performing up to the bridge design intensity.
The capacity curves of the structure have been determined performing other incremental analysis (ISPA, IUPA, IMPA) together with the proposed incremental multimodal pushover analysis for bridges IMPAβ and the incremental dynamic analysis IDA; the curves are plotted in Figure 14.

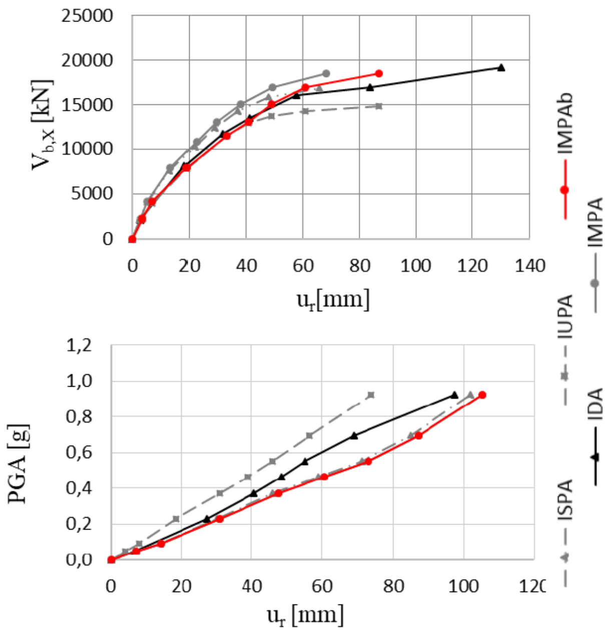

Figure 14: Case study – Comparison of the capacity curve derived with all the incremental procedure considered: IMPAβ results better performing if compared with IDA

4 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 has been tested and discussed through the application on two case study of an existing RC bridge, RB and IB. 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. IMPAβ, conservation the simplicity of a pushover method, if compared with “standard” pushover procedures results better performing. Analyzing results from the application discussed herein, the IMPAβ that is proposed as a combined use of IMPA (a procedure already proposed and widely applied on buildings) and IUPA (IMPAβ is the IMPA and IUPA envelope), seems to be the better performing procedure if compared with IDAand therefore suitable for bridges.
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.

5 ACKNOWLEDGMENTS

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