AN IMPROVED MULTIMODAL PROCEDURE FOR DERIVING PUSHOVER CURVES FOR BRIDGES

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ABSTRACT :

A modal pushover analysis (MPA) procedure for bridges is first proposed, that involves two additional steps compared to the initial proposal of the writers (Paraskeva et al, 2006), the key idea being that the inelastic deformed shape of the structure subjected to the considered earthquake level is used in lieu of the elastic mode shape. The concept of a multimodal pushover curve (derived using the MPA procedure) in terms of the base shear vs. deck maximum displacement is then put forward. The improved MPA procedure is used to assess the seismic performance of two actual bridges. The first structure is the Krystallopigi bridge, a 638m-long multi-span bridge, with significant curvature in plan, unequal pier heights, and different types of pier-to-deck connections. The second structure is a 100m-long three-span overcross bridge, typical in modern motorway construction in Europe, which is found to exhibit a rather unsymmetric response in the transverse direction, mainly due to torsional irregularity. It is found that the multimodal pushover curve reasonably matches the dynamic pushover curve derived from the more rigorous NL-RHA procedure. On the other hand, standard pushover curves based on single modal patterns, underestimated the total base shear and stiffness of the studied bridge.

KEYWORDS: bridges, seismic design, pushover analysis, inelastic response

1. INTRODUCTION

Nonlinear static (pushover) analysis is a widely used analytical tool for the evaluation of the behaviour of structures in the inelastic range and the identification of the locations of structural weaknesses, as well as of failure mechanisms (Krawinkler & Seneviratna 1998). The key outcome of this method is the pushover (or resistance) curve that provides a quantitative description of both the strength and the available ductility of the structure; such curves have been derived for a number of different structures and were used for their seismic assessment using different approaches and criteria.

To overcome the inherent limitation of the 'standard' (i.e. single-mode based) pushover analysis (SPA), its extension to consider higher mode effects has been attempted in a number of studies, initially focussing to buildings. Arguably, the most successful method was the Modal Pushover Analysis proposed by Chopra and Goel (2002), subsequently improved by the same researchers (Goel & Chopra 2004), wherein pushover analyses are carried out separately for each significant mode, and the contributions from individual modes to calculated response quantities are combined using a statistical combination rule. In this method pushover curves are drawn separately for each mode without attempting any 'synthesis' of them. Another promising category of methods are adaptive multi-mode pushover analysis procedures (Gupta & Kunnath 2000, Antoniou et al. 2004) involving redefinition of the loading pattern, which is determined by modal combination rules (i.e. SRSS of modal loads), at each stage of the response during which the dynamic characteristics of the structure change. Recently the writers (Kappos et al. 2004, Paraskeva et al. 2006, Kappos and Paraskeva, 2008) extended the multimodal pushover procedure proposed by Chopra and Goel for buildings to the case of bridges, and applied it to a complex actual bridge, comparing the results with those of single-mode pushover and of response history analysis (RHA) for spectrum-compatible records, while Isakovic & Fischinger (2006) used slightly different versions of the aforementioned pushover methods for the analysis of hypothetical irregular, torsionally sensitive bridges, and compared results. Recently, Casarotti & Pinho (2007) have extended the displacement-based version of adaptive pushover analysis to bridges and analysed a suite of idealised continuous multi-span bridges, while Lupoi et al. (2007) have applied MPA to an irregular old bridge and found that it worked well even when the bridge responds inelastically.

In view of the previous considerations, the present study introduces the new concept of a multi-modal pushover curve, wherein contributions from modal curves are appropriately synthesised. Alternative definitions of the 'dynamic pushover curves' are also explored, derived from the more rigorous NL-RHA, and comparisons are made for two actual reinforced concrete (R/C) bridges of different configuration, with promising results. The multi-modal pushover curves are derived using an improved modal pushover analysis procedure for bridges that involves two additional steps compared to the initial proposal of the writers (Paraskeva et al. 2006), the key idea being that the deformed shape of the structure subjected to the considered earthquake level is used in lieu of the elastic mode shape (Kappos and Paraskeva, 2008).

2. PROPOSED METHODOLOGY

2.1 Proposed improved MPA procedure

According to the Modal Pushover Analysis procedure, standard pushover analysis is performed for each mode independently, wherein the elastic modal forces are applied as invariant seismic load patterns. The basic steps of the method have been presented by Chopra and Goel (2002), while a set of additional assumptions and decisions regarding alternative procedures that can be used in order to apply the method in the case of bridges were proposed by Paraskeva et al. (2006). In developing the MPA procedure for bridges, it was found that both the target displacement and the bridge response quantities were dependent on the selected monitoring point. To overcome this problem, which is associated with the inelastic range of the modal pushover curves for higher modes, an improved MPA was presented recently (Kappos and Paraskeva 2008), involving two additional steps compared to the initial one, the key idea being that the deformed shape of the structure subjected to the considered earthquake level (to which it may respond inelastically) is used in lieu of the elastic mode shape. This improved procedure is summarised in the following.

At first, the natural periods, and mode shapes, for linearly elastic vibration of the structure are computed. Separate pushover analyses for force distribution $\mathbf{s}_n^* = \mathbf{m}\cdot\phi_n$, for each significant mode of the bridge are carried out and pushover curves (V_{bn} vs. u_{bn}) are drawn. The earthquake displacement and deformation demand associated with each of the pushover curves is then estimated. Herein, a modified version of the Capacity spectrum method based on inelastic demand spectra (Fajfar 1999) is used for defining the displacement demand for a given earthquake intensity, hence the base shear forces and the corresponding displacements in each pushover curve are converted to spectral acceleration (S_a) and spectral displacements (S_d) respectively, of an equivalent single-degree-of-freedom (SDOF) system, using the relationships (ATC 1996, Chopra & Goel 2002):

$$S_a = \frac{V_{bn}}{M_n^*}, S_d = \frac{u_{cn}}{\Gamma_n \cdot \phi_{cn}} \qquad (2.1)$$

wherein ϕ_{cn} is the value of ϕ_n at the control (or 'monitoring') point, $M_n^* = L_n \cdot \Gamma_n$ is the effective modal mass, with $L_n = \phi_n^T \mathbf{m} \cdot \mathbf{1}$, $\Gamma_n = L_n / M_n$ is a mass participation factor, and $M_n = \phi_n^T \mathbf{m} \cdot \phi_n$ is the generalized mass, for the nth natural mode. The corresponding displacement of the control point of the structure is then calculated using Eqn. 2.1. If the structure remains elastic or close to the yield point, the MPA procedure suggested by Paraskeva et al. (2006) is used to estimate seismic demands for the bridge. However, a correction of the displacement of the control point is necessary in cases that significant inelasticity develops in the structure. The response displacements of the structure are evaluated by extracting from the database of the individual pushover analyses the values of the desired responses at which the displacement at the control point is equal to u_{cn} (see eqn. 2.1). These displacements are then applied to derive a new vector ϕ_n', which is the deformed shape (affected by inelastic effects) of the bridge subjected to the given modal load pattern. The target displacement at the monitoring point for each pushover analysis is calculated again with the use of ϕ_n', according to:

$$u_{cn}' = \Gamma_n \cdot \phi_{cn}' \cdot S_{dn} \qquad (2.2)$$

wherein S_{dn} is the displacement of the SDOF system and Γ_n is recalculated using ϕ_n'. The response quantities of interest are evaluated by extracting from the database of the individual pushover analyses the values of the desired responses r_n, for the analysis step at which the displacement at the control point is equal to u_{cn}' (see eqn. 2.2). The total value for any desired response quantity (and each level of earthquake intensity considered) can be determined by combining the peak 'modal' responses, r_{no} using an appropriate statistical combination rule.

2.2 Derivation of multi-modal and dynamic pushover curves

In seismic assessment of structures the pushover (or better, the resistance) curve is a key tool, in the sense that it provides a good description of both the strength and the available ductility of the structure, both combined in a single diagram. Clearly, when the loading pattern used for producing such a curve is inadequate (i.e. when a single mode pattern is not sufficient for identifying the salient features of the response) one cannot expect to obtain a reliable pushover curve. In view of these remarks the concept of multimodal pushover curve (as part of the MPA procedure) in terms of base shear vs. deck displacement is presented herein for the case of bridges. The concept is also valid for buildings and indeed, to the authors' best knowledge this is the first time that such a 'composite' pushover curve is introduced for any type of structure. Han and Chopra (2006) have used the MPA procedure for producing Incremental Dynamic Analysis (IDA) curves, i.e. plots of an earthquake intensity measure vs. an engineering demand parameter; these curves are different from the multimodal pushover curves presented herein. The procedure proposed here is also different from the 'equivalent SDOF adaptive capacity curve' used by Casarotti & Pinho (2007).

A multimodal pushover curve cannot be derived from a simple combination of the individual curves derived for each modal pattern; it is essential that the earthquake demand for each stage of the bridge's response is properly accounted for, so that forces and displacements from each modal curve can indeed be combined (in a statistical sense, of course). The procedure for the derivation of a multimodal pushover curve is summarized in the following.

Response quantities of interest are calculated for each mode using the improved MPA procedure (Kappos and Paraskeva, 2008); this is repeated for as many modes as required for sufficient accuracy. The value of the deck 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_{cn} (or u_{cn}'), using a statistical combination rule. If the structure remains elastic for the considered earthquake intensity level, then the value of the base shear of the structure is determined using the same procedure. However, if the structure enters the inelastic range for the considered earthquake intensity level, a more involved procedure of combining modal shear forces in the piers is used. First, the total value of the plastic hinge rotation, θ_{pj} at each pier end is estimated as the SRSS combination of the modal values θ_{pjn}. The corresponding bending moments in the piers are estimated through the relevant moment-rotation diagram at the value of the plastic hinge rotation calculated from the SRSS combination. Then, shears in the piers are calculated using the corrected bending moments, and the base shear is calculated as the sum of the pier shears. The above procedure is repeated for as many earthquake intensity levels as required for drawing a representative multimodal pushover curve. The multimodal pushover curve is derived by plotting the total value of deck displacement of the selected control point against the corresponding value of the base shear of the structure, for each earthquake intensity level.

Validation of the aforementioned multimodal pushover curve can be made by comparing it to 'dynamic' pushover curves, derived from NL-RHA, which should not be confused with an IDA curve. A dynamic pushover curve can be derived by extracting from the 'database' of response-history analysis the values of the desired response quantities, i.e. base shear of the structure and displacements of the selected control point. The deck displacement of the control point is the average of the maximum displacements of the selected point recorded in the structure during the response history analyses for a number of accelerograms. Three different combinations of base shear and maximum displacement of the monitoring point were used here in order to derive dynamic pushover curves:

- Maximum displacement (u_{max}) at the control point vs. simultaneous base shear of the bridge ($V_b(t)$).
- Maximum displacement (u_{max}) at the control point vs. the base shear ($V_b(t-\Delta t)$) recorded at the previous step of that when u_{max} was recorded; or the base shear recorded after the step of the maximum displacement at the control point ($V_b(t+\Delta t)$)
- Maximum displacement (u_{max}) at the control point vs. maximum base shear of the bridge (V_{bmax}). This is considered only as an upper bound, since, obviously, these response quantities are not simultaneous.

It is noted that, in order to compare a 'standard' pushover curve (derived by SPA), to a multimodal pushover curve (derived by the MPA procedure or the improved MPA procedure wherever necessary), or to a dynamic pushover curve (derived by NL-RHA), the selected monitoring point of deck displacements has to be common for all curves.

3. CASE STUDIES

3.1. Description of studied bridges

The improved MPA procedure is applied to two actual bridge structures. The first bridge is the Krystallopigi bridge, a twelve span structure of 638m total length (Fig.1-left). The curvature in plan (radius equal to 488m) of the bridge adds to the expected complexity of its dynamic behaviour. Piers are rectangular hollow reinforced concrete members, while the height of the 11 piers varies between 11 and 27m. For the end piers M1 to M3 and M9 to M11 (Fig.1) a bearing type pier to deck connection is adopted, while the interior (taller) piers are monolithically connected to the deck. The second structure is an overcross (overpass) bridge with three spans and total length equal to 100m, typical in modern motorway construction in Europe (Fig. 1-right). Piers have a cylindrical cross section, while the pier heights are 8m and 10m. The deck is monolithically connected to the piers, while it rests on its two abutments through elastomeric bearings; movement in both the longitudinal and the transverse direction is initially allowed at the abutments, but transverse displacements are restrained whenever the 15cm gap (shown in the insert in Figure 1-right) is closed.

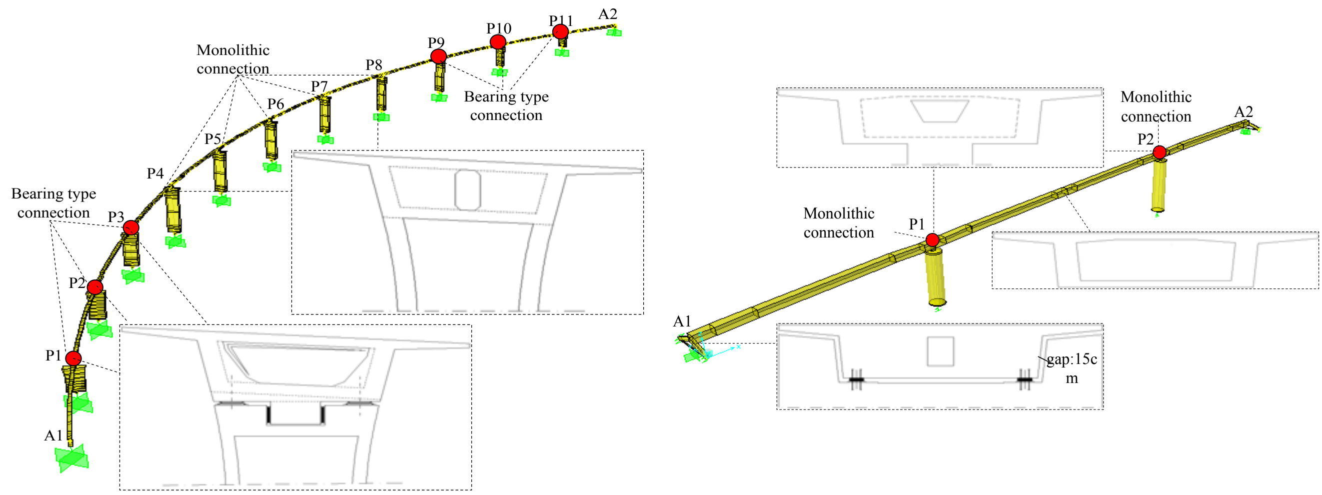


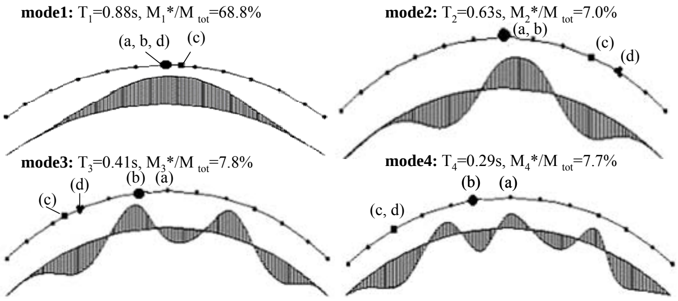
Figure 1 Layout of the Krystallopigi bridge and overcross bridge finite element modelling

The Greek Seismic Code (2000) design spectrum scaled to 0.32g for the first bridge and to 0.16g for the second one (different seismic zones and importance class), was used for seismic design. The design spectrum corresponded to ground category 'B' of EAK (same as in the ENV version of Eurocode 8, closer to 'C' in the final version (CEN 2004)). Both bridges were designed as ductile structures (plastic hinges expected in the piers); behaviour factors q=3.0 and q=2.4, respectively, were adopted for design. The bridges were assessed using standard pushover analysis (first mode loading), pushover analysis for a 'uniform' loading pattern (as required by Eurocode 8 (CEN 2004) and by ASCE Standard 41-07 (ASCE/SEI 2007)), modal pushover analysis as proposed in Paraskeva et al (2006), and improved modal pushover analysis as proposed herein; the demand spectrum in all analyses was the design one or multiples of it. The bridges were subsequently assessed using NL-RHA, for artificial records closely matching the demand spectrum. All analyses were carried out using the SAP2000 software package (Computers and Structures Inc, 1999).

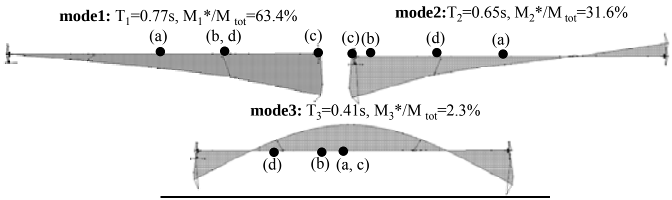
3.2. Nonlinear static analyses

Fundamental mode-based ('standard') pushover analyses as well as a 'uniform' loading pushover analysis were performed for assessing the inelastic response of the two selected bridges, results of these analyses (reported briefly herein, due to space limitations) were presented in detail in previous studies by the writers (Paraskeva et al, 2006, Kappos & Paraskeva, 2008). The dynamic characteristics required within the context of MPA approach, were determined using standard eigenvalue analysis. Figure 2 illustrates the first four transverse mode shapes of Krystallopigi bridge and the first three transverse mode shapes of the overcross bridge, together with the corresponding participation factors and mass ratios, and the locations of monitoring points of each mode. Consideration of the modes shown in

Figure 2 assures that more than 90% of the total mass in the transverse direction is considered. The deck displacements of the selected bridges derived using pushover analysis for each mode independently, as well as the MPA procedure were presented in detail in Kappos and Paraskeva (2008). It was found for both bridges that if the structure remains elastic for a given earthquake intensity both spectral displacement and the product u_{cn} will be independent of the selection of the control (monitoring) point. On the contrary it was found that deck displacements derived with respect to different control points, for inelastic behaviour of the structure are not identical but rather the estimated deformed shape of the bridge depends on the monitoring point selected for drawing the pushover curve for each mode. For this reason, an improved target displacement of the monitoring point is calculated using the improved MPA procedure in order to estimate the desired response quantities that are independent of the selection of control point.



	mode 1	mode 2	mode 3	mode 4
$x_{centre\ mass}$/(total length)	0.50	0.50	0.50	0.50
x_{SDOF}/(total length)	0.50	0.52	0.44	0.42
x_{max}/(total length)	0.54	0.80	0.19	0.14
$x_{critical\ pier}$/(total length)	0.50	0.86	0.23	0.14



		mode1	mode2	mode3
$x_{centre\ mass}$/(total length)	(a)	0.50	0.50	0.50
x_{SDOF}/(total length)	(b)	0.73	0.08	0.44
x_{max}(total length)	(c)	1.00	0.00	0.50
$x_{critical\ pier}$/(total length)	(d)	0.73	0.27	0.27

Figure 2 Modal force distribution, location of the equivalent SDOF systems and modal parameters for the main transverse modes of the Krystallopigi bridge (left) and the overcross bridge.

The deck displacements determined by the SPA and the (improved) MPA procedures were compared to those from NL-RHA, as shown in Figure 3. It is noted that the displacement demand is estimated independently in static and dynamic (time-history) inelastic analysis, whereas in some previous studies comparisons of displacement profiles are made assuming the same maximum displacement in both cases; the choice adopted here is deemed as more relevant for practical applications, as it permits an evaluation of all aspects of the proposed procedure.

In the case of Krystallopigi bridge (see Figure 3-left) it is observed that the SPA procedure predicts well the maximum transverse displacements only in the area of the central piers (an area dominated by the first mode). On the other hand the proposed (improved) MPA procedure which accounts for the other three transverse modes is much closer to RHA at the end areas of the bridge. The consideration of the higher modes with the proposed MPA scheme, significantly improves the accuracy of the predicted displacements, although its predictions are rather poor (but still better than those from SPA) in the areas close to the piers 5 and 8. From Figure 3-right it is observed that MPA predicts well the maximum transverse displacements of the overcross bridge. On the other hand, the SPA procedure underestimates the displacements of the deck at the location of the abutment A1 and the first pier of the bridge, compared to the more refined RHA approach. This is not surprising if one notes the differences between the first two mode shapes in the transverse direction (Figure 2-right), which are strongly affected by torsion (they contribute more than 90% of the torsional response, as well as over 90% of the transverse response of the bridge) due to the unrestrained transverse displacement at the abutments (until the 150mm gap closes), combined with the different stiffness of the two piers caused by their different height. What is essentially achieved by the (improved) MPA is the combination of these first two modes (the 3rd mode is not important herein), each of which dominates the response in the region of the corresponding abutment. Note that, the uniform loading pattern (also shown in Figure 3) fails to capture the increased displacements towards the abutments; nevertheless its overall prediction of the displacement profile could be deemed better than that resulting from using one single modal load pattern.

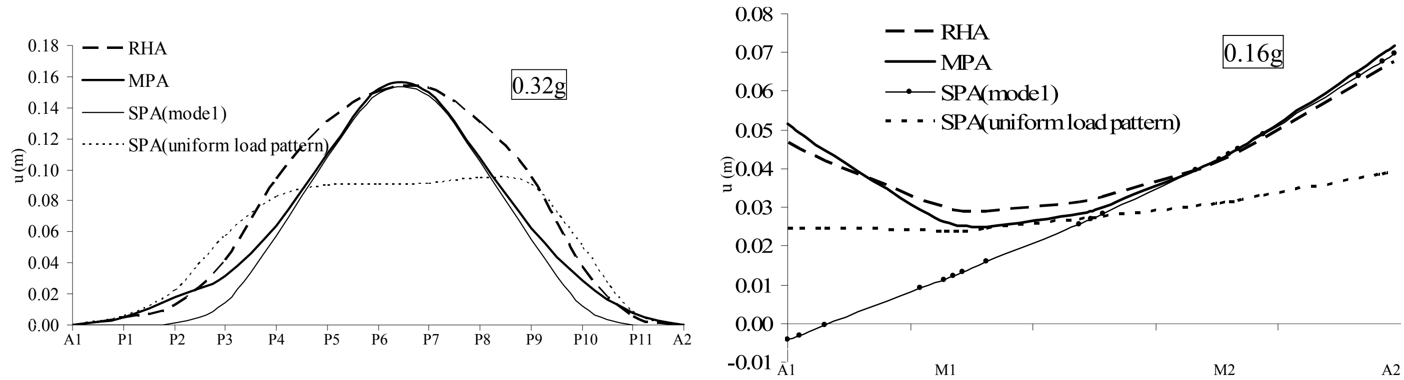


Figure 3 Response to the design earthquake calculated from SPA, MPA and RHA: deck displacements of the Krystallopigi bridge (left) and the overcross bridge (right).

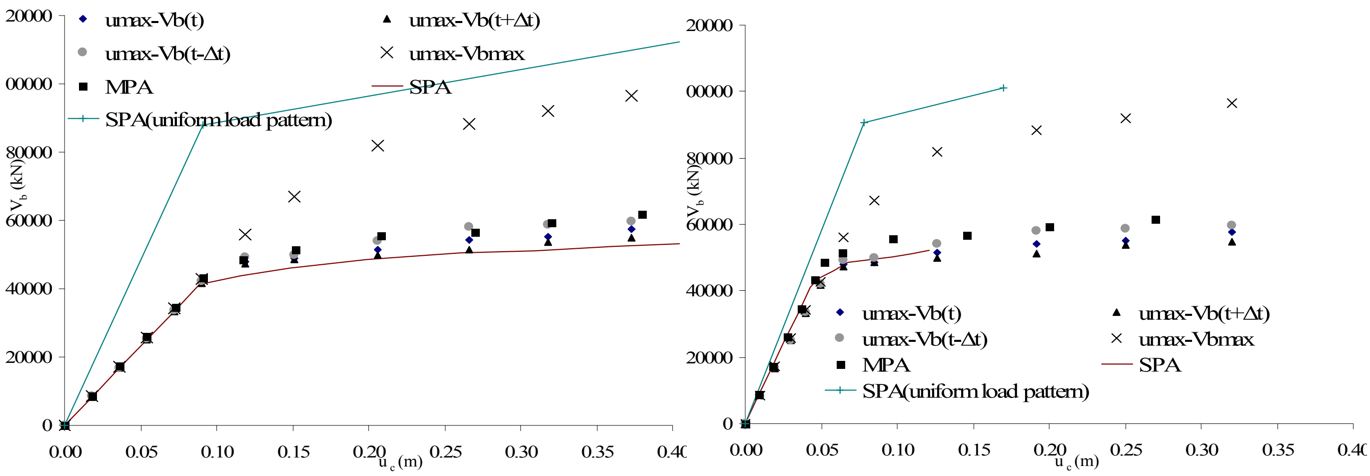


Figure 4 Pushover curves derived from SPA(first mode), SPA(uniform load), improved MPA and RHA for Krystallopigi bridge, derived with respect to the max deck displacement (deck mass centre) (left) and the deck displacement over P4

A key goal of the study was to compare results of the multimodal pushover curve, derived as proposed earlier on, with those from NL-RHA (dynamic pushover curves). The pushover curves derived using different procedures are shown in Figures 4 and 5. For the 'dynamic' pushover curve, the base shear of the structure, as well as the transverse deck displacement at the control point, were extracted from the database of the NL-RHA results for each intensity level. The three combinations of base shear and maximum displacement of the monitoring point, described in section 2.2, were used to derive dynamic pushover curves. In the diagrams shown in Figures 4 and 5, the point of maximum deck displacement was taken as the control point (for Krystallopigi bridge it is located at the mass centre of the deck, while for the overcross bridge it is at the location of the abutment A2, where displacement is maximum in SPA). It is clear from the figures that the multimodal pushover curve reasonably matches the 'dynamic' one derived from the more rigorous NL-RHA for both structures, while the pushover curve based on SPA method is less accurate. In particular, in the case of Krystallopigi bridge (see Figure 4-left), it is observed that multimodal pushover curve is almost identical to the physically meaningful 'dynamic' pushover curves, while the pushover curve based on standard pushover analysis slightly underestimated the total base shear of the structure. In the case of the overcross bridge, it is clear from Figure 5a that the multimodal pushover curve reasonably matches the 'dynamic' one derived from the more rigorous NL-RHA. On the other hand, the pushover curve based on a single modal pattern (SPA), strongly underestimated the total base shear (and the member shears) of the studied bridge. The pushover curve based on SPA using a uniform load pattern overestimated the total base shear (and the stiffness) of both studied bridges. It is notable, however, that the two SPA curves based on the load patterns recommended in modern codes (CEN 2004, ASCE/SEI 2007) envelope the true response of both bridges (the first-mode pattern provides a lower bound and the uniform pattern provides an upper bound). It is noted that the pushover curve based on SPA (load pattern proportional to the fundamental mode) is more accurate in the case of Krystallopigi bridge than for the overcross bridge, taking the dynamic pushover curves as the benchmark. This is expected from the discussion presented in the previous section where it was shown that the first transverse mode was sufficient for describing the response of the central part of the Krystallopigi bridge (to which the multimodal pushover curve refers), whereas the overcross bridge is strongly affected by torsion, and the first mode shape does not represent a realistic pattern of the bridge deck displacements.

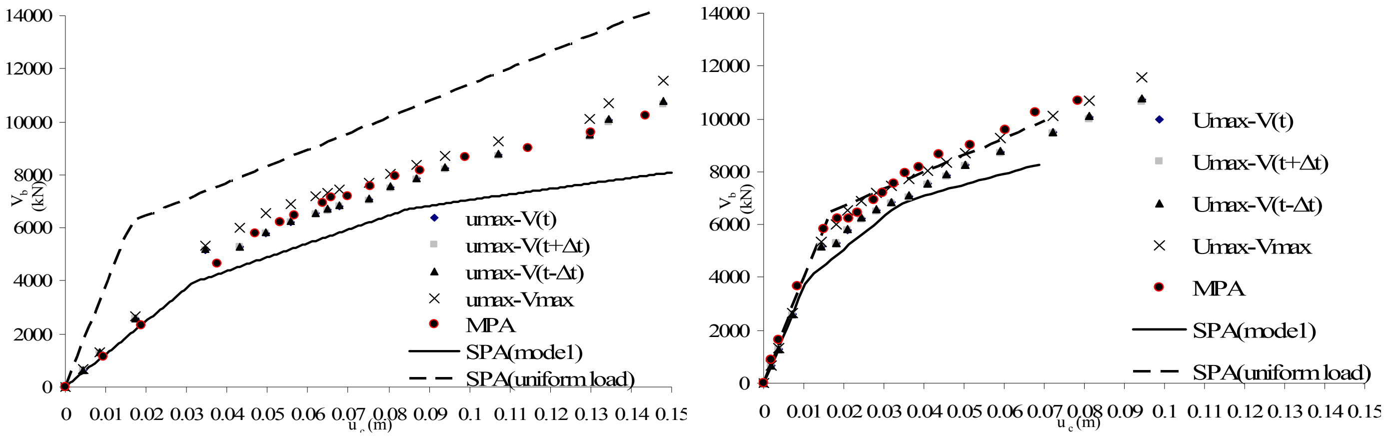


Figure 5. Pushover curves derived from SPA(first mode), SPA(uniform load), improved MPA and RHA for the overcross bridge, derived with respect to the maximum deck displacement (point A2) (left) and the deck mass centre.

In the diagrams shown on the right of Figures 4 and 5, different points on the deck of the selected bridges are selected as the monitoring point, and pushover curves are derived with respect to these points. In the case of Krystallopigi bridge, the point of the deck displacement above pier P4, (see Fig. 1) was selected as the control point, whereas the deck mass centre was selected in the case of the overcross bridge. It is observed that pushover curves based on SPA have a shorter second branch (the deck displacement of the monitoring point is strongly underestimated) for both structures. On the other hand the pushover curve based on the improved MPA reasonably matches the 'dynamic' pushover curve along the entire range of response. This is expected if one notes the differences between the displacement profile derived by SPA, MPA and NL-RHA in Figure 3, where the deck displacement of the aforementioned monitoring points, estimated by the MPA method tends to match that obtained by the NL-RHA, whereas SPA's predictions are poor, especially for increasing levels of earthquake excitation. Comparing the different curves shown in Figures 4 and 5, it is clear not only that the improved MPA procedure yields more accurate results compared to the 'standard' pushover procedure, but also that the selection of an appropriate monitoring point is a critical issue with regard to constructing realistic pushover curves.

4. CONCLUSIONS

An improved version of methodology initially suggested by the writers (Paraskeva et al. 2006) for carrying out modal pushover analysis of bridges was presented here, and its feasibility and accuracy were evaluated by applying it to two actual bridge structures, designed to modern seismic practice. By applying inelastic SPA and MPA, as well as RHA, to the selected bridge structures, it was concluded that all three pushover methods yielded similar values of maximum inelastic deck displacement, however the variation of displacements along the bridge was rather different. The SPA method predicted well the displacements only in the area of the bridge where the first mode is dominant. In the case of the overcross bridge the SPA was unable to predict a realistic pattern of deck displacements, because of the differences between the first two mode shapes in the transverse direction, which have strong torsional components and similar participation factors, and affect differently the region close to each abutment. On the contrary, MPA provided a significantly improved estimate with respect to the maximum displacement pattern, reasonably matching the results of the more rigorous RHA, for both bridges. Carrying out pushover analysis based on a uniform loading (as required by several codes), improves slightly the picture of the total response of the bridge; however, it still fails to capture the increased displacements towards the abutments. The improvement of the MPA procedure introduced herein was found to yield better results than the initial one, with displacement profiles that are not substantially affected by the selected control point when the bridge responds inellastically, hence results are deemed acceptable for all practical purposes

The new concept of a multimodal pushover curve, was introduced, and applied to the aforementioned actual bridges. It was found in both cases that the multimodal pushover curve reasonably matched the dynamic pushover curves derived from the more rigorous NL-RHA. On the other hand, standard pushover curves based on single modal patterns, underestimated the total base shear (and stiffness) of the studied bridge. On the positive side, standard pushover curves based on first-mode and uniform load patterns, were found to represent a lower and an upper bound, respectively, of the more rigorous 'dynamic' pushover curves; hence they could be used whenever simplicity is prioritised higher than accuracy. More work is clearly required to further investigate the effectiveness of MPA by applying it to bridge structures with different configuration, degree of irregularity, and dynamic characteristics (in terms of higher mode significance, in particular bridges with important anti-symmetric modes), since MPA is expected to be even more valuable for the assessment of the actual inelastic response of bridges with significant higher modes.

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