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FURTHER DEVELOPMENT OF A MULTIMODAL PUSHOVER ANALYSIS PROCEDURE FOR SEISMIC ASSESSMENT OF BRIDGES

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SUMMARY

An improvement is first suggested to the modal pushover analysis (MPA) procedure for bridges initially proposed by the writers [1], the key idea being that the deformed shape of the structure responding inelastically to the considered earthquake level is used in lieu of the elastic mode shape. The proposed MPA procedure is then verified by applying it to 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 overpass bridge, typical in modern motorway construction in Europe, which, although ostensibly a regular structure, is found to exhibit a rather unsymmetric response in the transverse direction, mainly due to torsional irregularity. The bridges are assessed using response spectrum, 'standard' pushover (SPA), and modal pushover analysis, and finally using non-linear response history analysis (NL-RHA) for a number of spectrum-compatible motions. The MPA provided a good estimate of the maximum inelastic deck displacement for several earthquake intensities. The SPA on the other hand could not predict well the inelastic deck displacements along the bridge, because of the low contribution of the first mode to the total response of the bridge.

KEYWORDS: bridges, seismic assessment, pushover analysis, inelastic response, reinforced concrete, higher mode effects

INTRODUCTION

Extension of the 'standard', fundamental mode based, pushover analysis (SPA), to consider higher mode effects has attracted attention over the last decade. Several efforts made in this direction are briefly reviewed in a previous paper by Paraskeva et al. [1] wherein the Modal Pushover Analysis (MPA) proposed by Chopra and Goel [2] was extended to the case of bridges; the procedure was applied to a rather complex actual bridge, and results were compared with those from single-mode pushover and response-history analysis.

The first part of the present study identifies a weakness in the aforementioned procedure [1] and proposes an improvement to it, the key idea being that in the

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calculation of displacement demand 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. It is worth pointing out that the idea of using the inelastic deflection shape in deriving the properties of an equivalent single-degree-of-freedom (SDOF) system has been previously suggested for buildings [3]. Also, in the FEMA 273 document [4] which was the first to provide a solid basis for the practical application of pushover analysis, the C_0 factor that relates spectral displacement to the roof displacement can be (optionally) calculated as the modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement. To the authors' best knowledge, the foregoing idea has not been used for bridges, certainly not in the framework of MPA.

In the second part of the paper, the proposed MPA procedure, with the improvement introduced wherever necessary, is used to assess the seismic performance of an overpass bridge, typical in modern motorway construction in Europe, and a 638m-long multi-span bridge, with significant curvature in plan, unequal pier heights, and different types of pier-to-deck connections; Results are compared with those from 'standard', and modal pushover analysis, and finally non-linear response history analysis for a number of spectrum-compatible motions.

PROPOSED METHODOLOGY

Problems encountered in the initially proposed 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. Modal pushover curves are then plotted and can be converted to capacity diagrams using modal conversion parameters (other options for estimating target displacements are also available in the literature). Response quantities are separately estimated for each individual mode, and then superimposed using an appropriate modal combination rule. The basic steps of the method have been first presented by Chopra and Goel [2], and the method has been subsequently improved by the same authors [4]. Additional issues, assumptions and decisions regarding alternative procedures that are needed in order to apply the method in the case of bridges have been presented by Paraskeva et al [1].

In developing the MPA procedure for bridges [1], wherein higher modes usually play a critical role, it was found that both the target displacement and the bridge response quantities were dependent on the selected monitoring (or control) point; case-studies illustrating this point are given in the next section. To overcome this problem, which is associated with the inelastic range of the modal pushover curves for higher modes, an improved MPA is first proposed herein, involving an additional step compared to the initial one. To investigate the applicability of the improved MPA procedure for bridges, a number of actual bridge structures were studied, two of which are reported herein.

Proposed improved procedure

For the sake of completeness (and the benefit of the reader) all steps of the modified MPA procedure (including those that are the same as in the Chopra and Goel method) are briefly summarized in the following.

Step 1: Compute the natural periods, T_n, and mode shapes, ϕ_n, for linearly elastic vibration of the structure.

Step2: Carry out separate pushover analyses for force distribution $\mathbf{s}_n^* = \mathbf{m}\cdot\phi_n$, for each significant mode of the bridge and construct the base shear vs. displacement of the 'control' or 'monitoring' point (V_{bn} vs. u_{cn}) pushover curve for each mode.

Step3: The pushover curve must be idealized as a bilinear curve so that a yield point and ductility factor can be defined and then be used to appropriately reduce the elastic response spectra representing the seismic action considered for assessment. This idealization can be done in a number of ways, some more involved than others; the remaining steps of the proposed methodology can be applied regardless of the method used for producing a bilinear curve.

Step4: The earthquake displacement demand for a given earthquake intensity associated with each of the pushover curves derived in Step 3 is estimated using the capacity and demand 'spectra' [6-8] approach. Hence Step 4 consists in converting the idealized $V_{bn} - u_{cn}$ pushover curve (base shear vs. displacement of control point) of the multi-degree-of freedom (MDOF) system to a 'capacity diagram', in terms of spectral acceleration (S_a) and spectral displacements (S_d) using well-known relationships [2, 6]. For *inelastic* behaviour, the procedure used here for estimating the displacement demand at the monitoring point is based on the use of inelastic spectra [7, 8]; this is equally simple, more consistent, and generally more accurate than the 'standard' capacity spectrum method (CSM) adopted by ATC [6] that is based on reducing the elastic spectra with ductility–dependent damping factors.

Step 5: Conversion of the displacement demand of the n^{th} mode inelastic SDOF system to the peak displacement of the monitoring point, u_{cn} of the bridge, using equation

$$S_d = \frac{u_{cn}}{\Gamma_n \cdot \phi_{cn}} \tag{1}$$

wherein ϕ_{cn} is the value of ϕ_n at the control point, $\Gamma_n=L_n/M_n$ is a mass participation factor, where $L_n=\phi_n^T\mathbf{m}\cdot\mathbf{1}$, and $M_n=\phi_n^T\mathbf{m}\cdot\phi_n$ is the generalized mass, for the n^{th} natural mode.

Step 6: In this step, a correction is made to the displacement of the monitoring point of the bridge, which was calculated at the previous steps 4 and 5. The correction is necessary only for cases that significant inelasticity develops in the structure. If the structure remains elastic or close to the yield point, the MPA procedure suggested by Paraskeva et al. [1] is used to estimate seismic demands for the bridge. 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} (from equation 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

$$u'_{cn} = \Gamma'_n \cdot \varphi'_n \cdot S'_{dn} \tag{2}$$

wherein Γ'_n is Γ_n recalculated using ϕ'_n, and S'_{dn} is the displacement of the equivalent SDOF system (which generally differs from S_{dn}). Application of (2) to the case-studies presented in the next section has shown that S'_{dn} *is very close to* S_{dn} (whereas this is not the case with u'_{cn} and u_{cn}); hence, in practical application it suffices to repeat only Step 5 (and not 4) during Step 6.

Step 7: The response quantities of interest (displacements, plastic hinge rotations, forces in the piers) are evaluated by extracting from the database of the individual pushover analyses the values of the desired responses r_n, due to the combined effects of gravity and lateral loads for the analysis step at which the displacement at the control point is equal to u_{cn} (see equation 2).

Step 8: Steps *3* to *7* are repeated for as many modes as required for sufficient accuracy. It was found [1] that there is little merit in adding modes whose participation factor is very low (say less than 1%), and application of the method to a number of bridges shows that it is not necessary to assure that the considered modes contribute to 90% of the total mass.

Step 9: The total value for any desired response quantity (and each level of earthquake intensity considered) is determined by combining the peak 'modal' responses, r_{no} using an appropriate modal combination rule, e.g. the SRSS combination rule, or the CQC rule. This simple procedure was used for both displacements and plastic hinge rotations in the present study, which were the main quantities used for assessing the bridges analysed (whose response to service gravity loading was, of course, elastic). If inelastic member (e.g. pier) forces have to be determined accurately, a more involved procedure of combining modal responses should be used. Such a procedure was suggested by Goel and Chopra [4] for buildings, consisting essentially in correcting the bending moments at member ends (whenever yield values were exceeded) on the basis of the relevant moment – rotation diagram and the value of the calculated plastic hinge rotation. This procedure, which blends well with the capabilities of currently available software, has also been used in the case studies presented in the next section.

CASE STUDIES

Description of studied bridges

To investigate the accuracy and also the practicality of the proposed procedure it was deemed appropriate to apply it to a number of actual bridges, two of which are presented in some detail here, while a brief reference to a third one is also made. The Krystallopigi bridge, a twelve-span structure of 638m total length and substantial curvature in plan, was presented in detail in [1]. Piers are rectangular hollow reinforced concrete members, while the height of the 11 piers varies between 11 and 27m. For the end piers (P1 to P3 and P9 to P11) 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 overpass (overcrossing) bridge with three spans and total length equal to 100m, typical in modern motorway construction in Europe (Fig. 1). Piers have a cylindrical cross section, a common choice for bridges both in Europe and in other areas, while the pier heights are unequal (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 is closed.

The Greek Seismic Code (EAK) design spectrum scaled to 0.24g for the first bridge and to 0.16g for the second one (different seismic zones), 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 of the Code [8]). Both bridges were designed as ductile structures (plastic hinges expected in the piers), with behaviour (i.e. force reduction) factors q=3.0 and q=2.4, respectively.

The bridges were assessed using standard pushover analysis (first mode loading), pushover analysis for a 'uniform' loading pattern (as required by Eurocode 8 [8] and other codes), modal pushover analysis as proposed in Paraskeva et al. [1], 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, described in [1]. Details of the inelastic modelling procedure (using the SAP2000 software package) of the Krystallopigi bridge are given in [1]; the same modelling approach was adopted for the overpass bridge (details given in [9]).

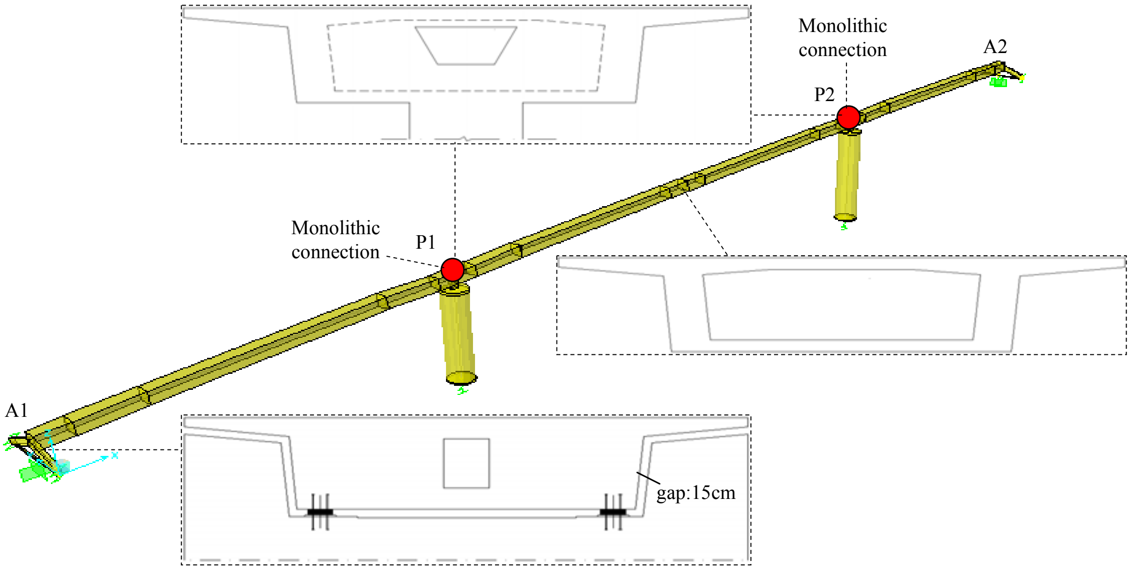
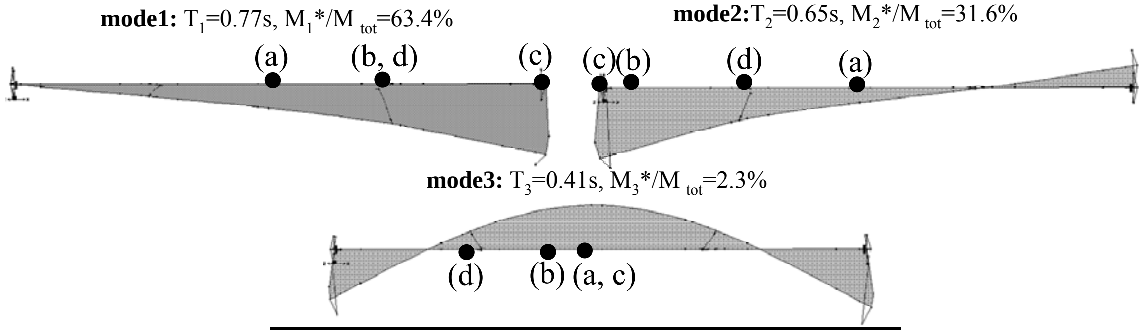


Figure 1: Layout of the overpass bridge finite element modelling

Non linear static analyses

Fundamental mode-based ('standard'), as well as 'uniform' loading, pushover analyses were first performed for assessing the inelastic response of the selected bridges; results of these analyses (reported only briefly herein, due to space limitations) were presented in detail in previous studies by the writers [1, 9].

The dynamic characteristics of the bridges, required within the context of the MPA approach, were determined using standard eigenvalue analysis. Figure 2 illustrates the first three transverse mode shapes of the overpass bridge, together with the corresponding participation factors and mass ratios, as well as the locations of monitoring points for each mode; similar information for the other bridge is given in [1]. Consideration of the modes shown in Figure 2 assures that more than 90% of the total mass in the transverse direction is considered. Applying the modal load pattern of the n^{th} mode in the transverse direction of the bridge, the corresponding pushover curve was constructed and then idealized as a bilinear curve. Figures 3 and 4 illustrate the deck displacements of the selected bridges derived using pushover analysis for each mode independently, as well as the MPA procedure initially proposed in Paraskeva et al [1]. If the structure remains elastic for the given earthquake intensity, both spectral displacement S_d and the product $\Gamma_n \cdot \phi_n$ will be independent of the selection of the control (monitoring) point; this means that deck displacements are independent of the location of the 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.



		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 overpass bridge

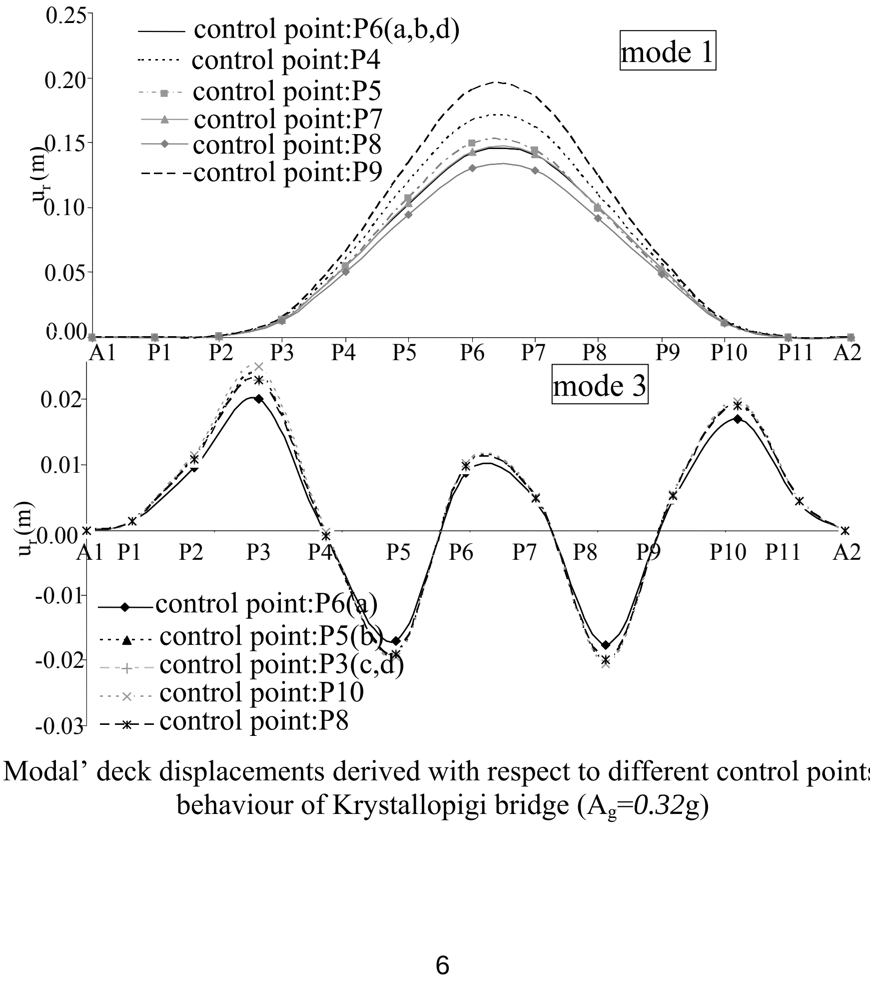


Figure 3: 'Modal' deck displacements derived with respect to different control points – *inelastic* behaviour of Krystallopigi bridge (A_g=*0.32*g)

For inelastic behaviour, equation (1) gives a different value of u_{cn}, not only because of the deviation of the elastic mode shape ϕ_n from the actual deformed shape of the structure, but also due to the fact that the spectral displacement S_d is dependent on the selection of monitoring point if the structure exhibits inelastic behaviour (due to the bilinearization of the capacity curve). An improved target displacement of the monitoring point is calculated (from eq. 2) using ϕ_n', the actual deformed shape of the structure (see figures 3 and 4), while the spectral displacement can be kept the same as noted earlier. The response quantities of interest are evaluated for the analysis step at which the displacement at the control point is equal to u_{cn}' (the improved estimate of u_{cn} derived on the basis of ϕ_n').

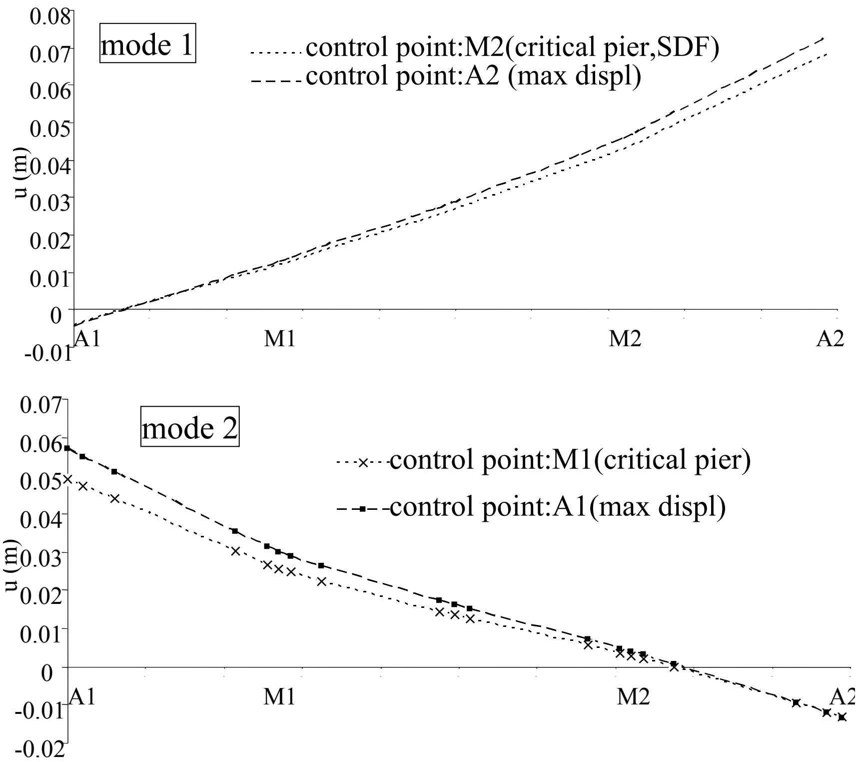


Figure 4: 'Modal' deck displacements derived with respect to different control points – *inelastic* behaviour of the overpass bridge (A_g=*0.16*g)

Figures 5 and 6 illustrate the deck displacements of Krystallopigi bridge and the overpass bridge, respectively, calculated from SPA using u_{cn}' as target displacement for each mode. It is noted that, due to the approximations involved in the capacity-demand spectra procedure, deck displacements derived with respect to different control points are not identical, but differences are significantly reduced and results are deemed acceptable for all practical purposes. From Figures 3 to 6 it is observed that the differences between deck displacements derived with respect to different control points, as well as the improvement in the prediction of deck displacements using the procedure proposed here, are more significant in the case of Krystallopigi bridge than in the overpass bridge. This is attributed to the larger length combined with the curvature in plan of the former bridge, which amplifies the complexity of its dynamic behaviour and renders more significant the contribution of higher modes (especially towards the abutments).

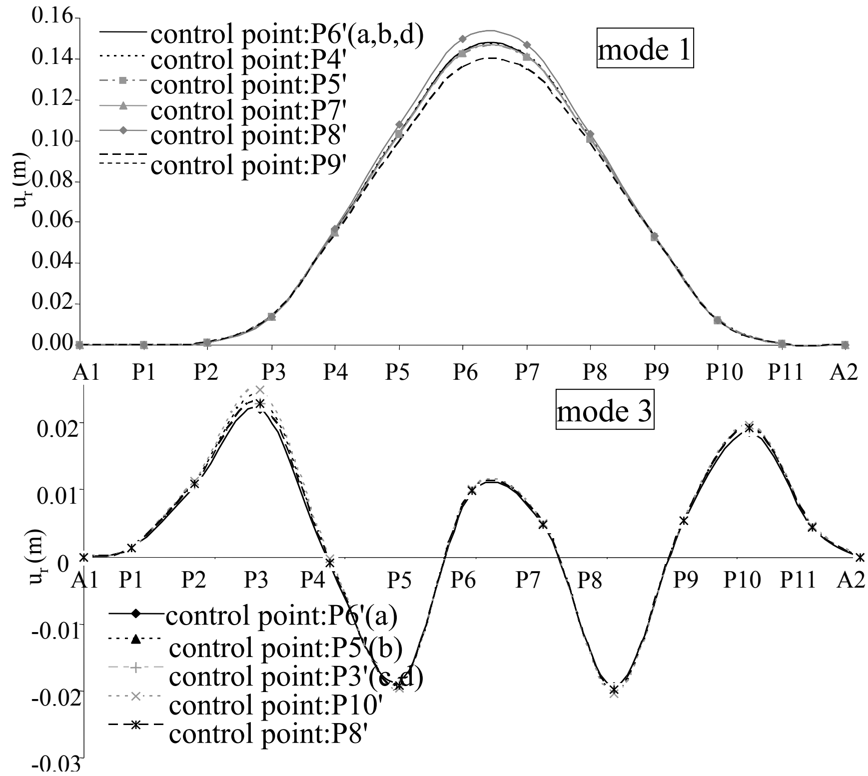


Figure 5: 'Modal' deck displacements derived with respect to different control points using u_{rn}' as target displacement according to the *improved* MPA procedure- Krystallopigi bridge (A_g=0.32g)

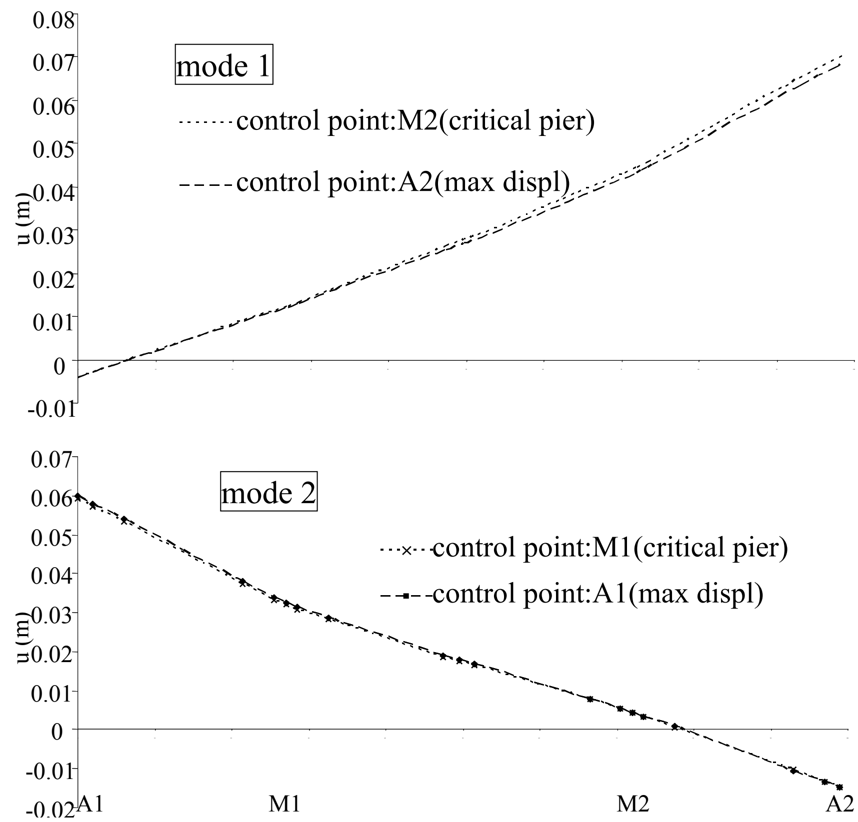


Figure 6: 'Modal' deck displacements derived with respect to different control points using u_{rn}' as target displacement according to the *improved* MPA procedure – overpass bridge (A_g =0.16g)

Evaluation of different procedures

Results of the standard and modal pushover approaches were evaluated by comparing them with those from non-linear response history analysis for 5 artificial records compatible with the design spectrum. The Newmark $\gamma=1/2$, $\beta=1/4$ integration method was used, with time step $\Delta t=0.0025$s and a total of 10,000 steps (25s of input). A uniform damping value of 5% was assumed for all modes of vibration, while hysteretic damping was accounted for through the elastoplastic behaviour of the structural members.

The displacements determined by the SPA and MPA procedures were compared to those from NL-RHA for increasing levels of earthquake excitation, as shown in Figures 7 and 8. It is noted that the deck displacements shown in the figures as the NL-RHA case are the average of the peak displacements recorded in the structure during the five response-history analyses. Besides, in all the results shown, 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.

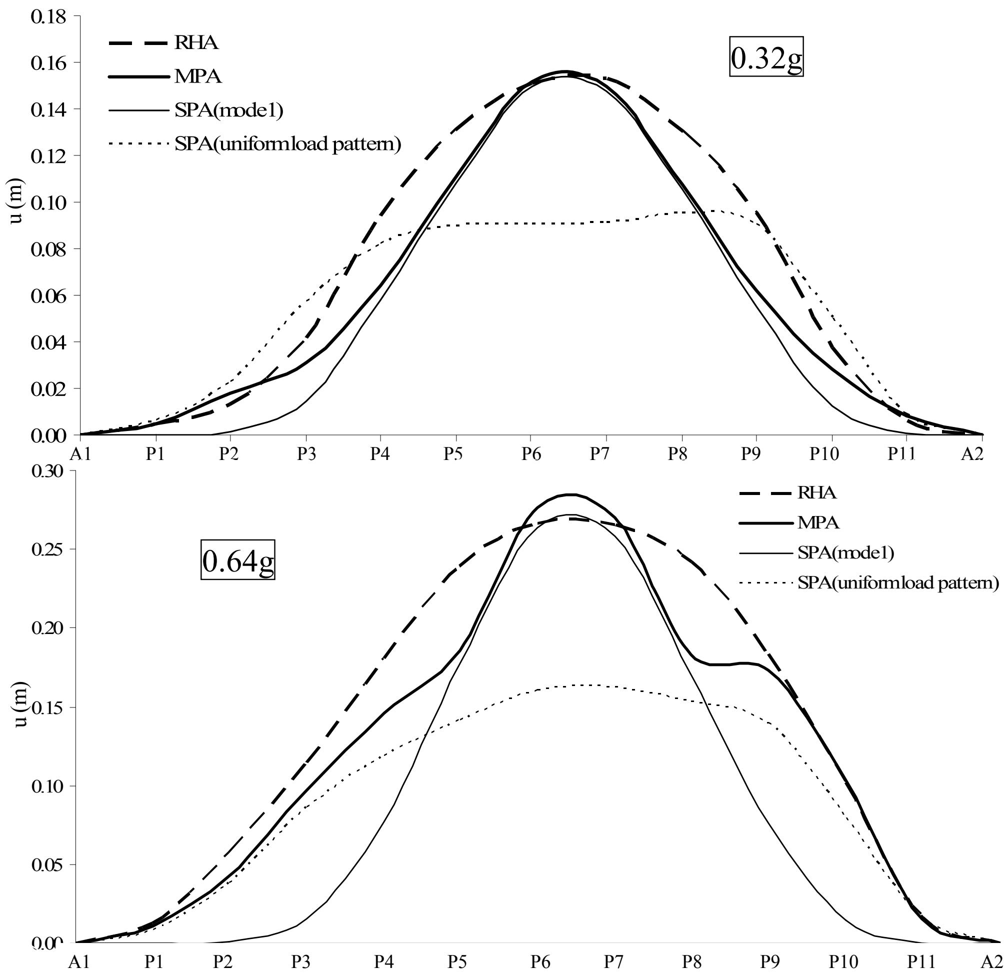


Figure 7: Response to the design earthquake (A_g =0.32g) and to twice the design earthquake (A_g =0.64g), calculated from SPA, MPA and NL-THA: deck displacements of Krystallopigi bridge

In the case of Krystallopigi bridge (Figure 7) 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 MPA procedure which accounts for the other three transverse modes is much closer to NL-RHA at the end areas of the bridge. As the level of excitation increases and higher mode contributions become more significant (without substantially altering the shape of the modes) the displacement profile derived by the MPA method tends to match that obtained by the NL-RHA, whereas predictions from SPA become less accurate as the level of inelasticity increases. The consideration of 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.

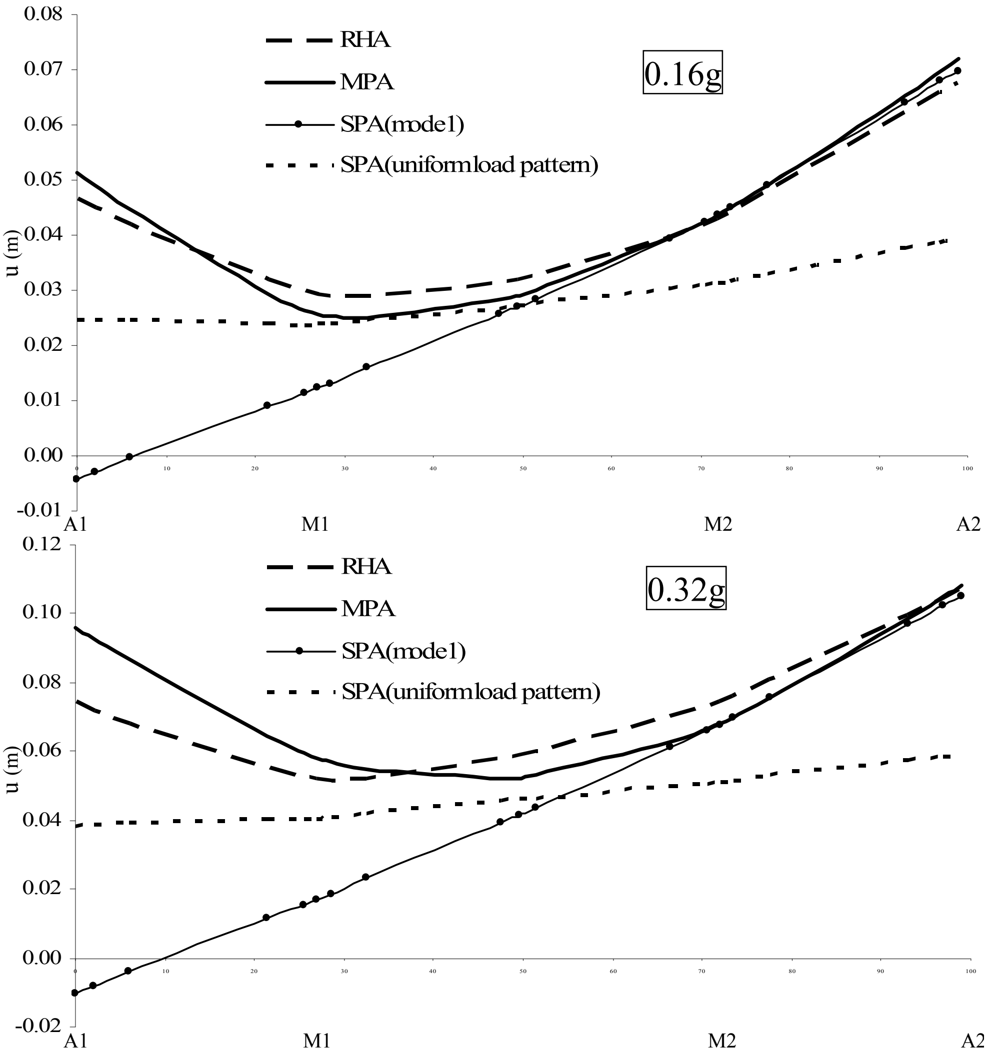


Figure 8: Response to the design earthquake (A_g =0.16g) and to twice the design earthquake (A_g =0.32g) calculated from SPA, MPA and RHA: deck displacements of the overpass bridge

From Figure 8 it is observed that MPA predicts well (i.e. matches closely the values from the NL-RHA approach) the maximum transverse displacement of the overpass 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 NL-RHA approach. This is not surprising if one notes the differences between the first two mode shapes in the transverse direction (Figure 2), 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 15cm gap closes), combined with the different stiffness of the two piers caused by their different height. What is essentially achieved by the MPA is the combination of these first two modes (the 3rd transverse mode is not important in this particular bridge), each of which dominates the response in the region of the corresponding abutment. In the case of applying ground motions with twice the design earthquake intensity (also shown in Figure 8), where the structure enters deeper into the inelastic range and higher mode contributions become more significant (without substantial alteration of the mode shapes) it is noted that the displacement profile derived by the MPA method tends to match that obtained by the NL-RHA, whereas SPA's predictions remain poor. Note that, regardless of earthquake intensity, the uniform loading pattern (also shown in Figures 7 and 8) 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 any single modal load pattern.

To put MPA into the right context, an additional case-study is recalled here, involving a 247 m long, essentially regular, actual bridge, with small curvature in plan, supported on two piers that are monolithically connected to the deck, reported by the authors in [10]. Although the piers are of unequal height (36 m and 45 m) and the total length is more than twice that of the overpass bridge studied here, the fact that the tranverse displacement is blocked at the abutments leads to a much more regular configuration, without noticeable torsional effects (the 2nd transverse mode is almost symmetric and has a mass participation factor of 16%, as opposed to 66% for the 1st mode). For this bridge, appplication of SPA was found [10] to yield results very close to those from both MPA and NL-RHA for up to twice the design earthquake intensity.

CONCLUSIONS AND RECOMMENDATIONS

An improved version of the methodology initially suggested by the writers [1] for carrying out modal pushover analysis of bridges was presented herein. The improvement introduced to the MPA procedure was found to yield better results, and to make the procedure less sensitive to the selection of the control point (hence more concise), since calculated displacement profiles are not substantially affected by its selection, even when the bridge responds inelastically; final results are deemed acceptable for all practical purposes.

The feasibility and accuracy of the proposed MPA procedure (with the improvement introduced whenever necessary, as discussed in the paper) were evaluated by applying it to two different actual bridges, the 638m-long multi-span Krystallopigi bridge, and a typical overpass bridge, both designed to modern seismic practice. It was concluded that:

- In the case of *Krystallopigi* bridge, the three pushover methods yielded rather different variation of displacement along the bridge. The SPA method predicted well the displacements only in the central area of the bridge where the first mode is dominant, whereas the (improved) MPA method provided significantly more accurate estimates of the maximum displacement pattern, reasonably matching the results of the more refined NL-RHA analysis, even for increasing levels of earthquake loading that trigger increased contribution of higher modes. Carrying out pushover analysis based on a uniform loading pattern, failed to capture the maximum displacements at the central area of the structure.

- In the case of the *overpass* bridge, all three pushover methods yielded similar values of maximum inelastic deck displacement at the area of the abutment A2. However, the variation of displacement along the bridge was rather different. The SPA method 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, the improved MPA provided a significantly better estimate with respect to the maximum displacement pattern, reasonably matching the results of the more rigorous NL-RHA, even for high levels of earthquake loading (compared to the design earthquake). Results from pushover analysis based on the uniform loading pattern suffered from the same drawbacks as in the case of Krystallopigi bridge.
- The present study confirmed findings from previous studies [10, 11], which have indicated that SPA generally works reasonably well when applied to bridges of regular configuration (as opposed to irregular ones, such as those affected by torsion).

On the basis of the results obtained for the studied bridges, the improved MPA procedure appears to be a promising approach that yields generally more accurate results (for these bridges) compared to the 'standard' pushover, without requiring the higher computational cost of the NL-RHA. It is emphasised again that the extra effort involved in carrying out the additional step proposed herein is warranted only when the inelastic deformed shape is clearly different from the elastic mode shape.

More work is clearly required to further investigate the effectiveness of MPA by applying it to bridge structures with different configuration, degree of irregularity (including cases where the deformed shape of the bridge changes substantially during pushover analysis), and dynamic characteristics, since MPA is expected to be even more valuable for the assessment of the actual inelastic response of bridges with significant higher modes. Finally, at this stage of development, the improved MPA procedure is not implemented in a software package that can carry it out in a single run (a second run is necessary, using the results of the previous one stored by proper post-processing); such an implementation would substantially increase the practical value of the procedure.

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