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MODAL PUSHOVER ANALYSIS AS A MEANS FOR THE SEISMIC ASSESSMENT OF BRIDGE STRUCTURES

A. J. KAPPOS
Professor, Lab. of Concrete and Masonry Structures, Department of Civil Engineering, Aristotle University Thessaloniki, 54124 Greece

T.S. PARASKEVA
Graduate Student, Lab. of Concrete and Masonry Structures, Department of Civil Engineering, Aristotle University Thessaloniki, Greece

A.G. SEXTOS
Lecturer, Structural Division, Department of Civil Engineering, Aristotle University Thessaloniki, Greece

ABSTRACT

Nonlinear static (pushover) analysis has become a popular tool during the last decade for the assessment of buildings. Nevertheless, its main advantage of lower computational cost compared to nonlinear dynamic time-history analysis is balanced by the inherent restriction that essentially only the fundamental mode is utilized. Extension of the pushover approach to consider higher modes effects has attracted attention, the effort being to match as closely as possible the results of nonlinear time history analysis. So far such work has primarily focused on buildings, while corresponding work for bridges has been very limited. Hence, the aim of this study is to investigate the extension of the modal pushover method to bridges, and the investigation of its applicability in the case of complex bridges. To this effect, a real, long and curved bridge is chosen, designed according to current seismic codes; this bridge is assessed using the aforementioned three nonlinear analysis methods. Comparative evaluation of the calculated response of the bridge illustrates the applicability and potential of the modal pushover method for bridges, and quantifies its relative accuracy compared to that obtained through the 'standard' pushover approach.

INTRODUCTION

Although elastic analysis provides a useful overview of the expected dynamic response of a bridge, in general it cannot predict the failure mechanisms or the redistribution of forces that follow plastic hinge development during strong ground shaking. Nonlinear pushover analysis on the other hand, is a widely used analytical tool for the evaluation of the structural behaviour in the inelastic range and the identification of the locations of structural weaknesses as well as of failure mechanisms [1], [2]. Nevertheless, the method is limited by the assumption that the response of the structure is controlled by its fundamental mode. In particular, the structure is subjected to monotonically increasing lateral forces with an invariant spatial distribution until a predetermined target displacement is reached at a monitoring point. As a result, both the invariant force distributions and the target displacement, do not account for higher mode contribution, which can affect both, particularly in the inelastic range, thus limiting the application of the approach to cases where the fundamental mode is dominant. Extension of the 'standard' pushover analysis (SPA) to consider higher modes effects has attracted attention, the effort being to match as closely as possible the results of nonlinear time history analysis. In an early effort [3], the multi-mode pushover procedure was used to identify the effects of higher modes in pushover analysis of buildings by appropriately extending the Capacity Spectrum Method (CSM), which is a means to directly compare building capacity to earthquake demand. A series of adaptive multi-mode pushover analysis methods followed [4], [5], [6] involving redefinition of the loading pattern, which is determined by modal combination rules (e.g. SRSS of modal loads), at each stage of the response during which the dynamic characteristics of the structure change (usually at each step when a new plastic hinge forms).

While in all previous methods modal superposition is carried out at the level of loading, in the Modal Pushover Analysis (MPA) proposed by Chopra and Goel [7], subsequently improved by Goel and Chopra [8], pushover analyses are carried our separately for each significant mode, and the contributions from individual modes to calculated response quantities (displacements, drifts, etc.) are combined using an appropriate combination rule (SRSS or CQC). Although, theoretically, superposition of modal responses does not apply in the inelastic range of the response (modes are not uncoupled anymore), Goel and Chopra [8] have shown that the error, taking the results of inelastic time-history analysis as the benchmark, is typically smaller than in the case that superposition is carried out at the level of loading (with fixed loading pattern), as recommended in the FEMA356 Guidelines [9]; these guidelines adopt the Nonlinear Static Procedure (NSP), i.e. pushover analysis, with two different load patterns, one based on first mode loading ('triangular' distribution) and one with 'modal' distribution (SRSS combination of elastic modal loads). In another recent development, Aydinoglu [10] has proposed the so-called "incremental response spectrum analysis (IRSA)", wherein each time a new hinge forms in a structure, elastic modal spectrum analysis is performed, taking into account the changes in the dynamic properties of the structure.

So far, most of the work performed in the direction of extending the applicability of pushover analysis to structures with more complex dynamic characteristics was focused on buildings. Bridges, on the other hand, are structures where higher modes usually play a more critical role than in buildings; hence developing a modal pushover procedure for such structures is even more of a challenge than in the case of buildings. From the previously mentioned studies, only in that of Aydinoglu [10], is a bridge structure tackled; the IRSA procedure is used, taking one or eight modes into account, without detailed discussion of the resulting differences. At the same time as [10] (summer 2004) two more studies appear, one by Kappos et al. [11], and one by Fischinger et al. [12]; the first one applies a multimodal pushover procedure generally similar to that of Chopra and Goel [7] to a complex actual bridge (studied in more detail herein) considering its first three transverse modes, and compares the results with those of single-mode pushover and of time-history analysis for spectrum-compatible records. In the study by Fischinger et al. [12] slightly different versions of these three methods, as well as IRSA, are used for the analysis of a hypothetical irregular, torsionally sensitive bridge, and results are compared. The need for further work is pointed out in these few studies on bridges. In view of the previous considerations, the present study first proposes a clearly defined procedure for applying the MPA in the case of bridges and then attempts to quantify the relative accuracy of the three main inelastic analysis methods (i.e. SPA, MPA and nonlinear time history) by focusing on the realistic case of a complex, long and curved, actual bridge.

PROPOSED PROCEDURE FOR MODAL PUSHOVER ANALYSIS OF BRIDGES

Modal Pushover Analysis is in fact an extension of the 'standard' pushover analysis. According to this procedure, standard pushover analysis is performed for each mode independently, wherein invariant seismic load patterns are defined according to the elastic modal forces. Modal pushover curves are then plotted and can be converted to SDF capacity diagrams using modal conversion parameters based on the same shapes. 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 presented by Chopra and Goel [7], but a set of additional assumptions and decisions regarding alternative procedures that can be used have to be made in order to apply the method in the case of bridges. The proposed steps are summarized below:

Step 1: Compute the natural periods, T_n and modes, ϕ_n, for linearly elastic vibration of the structure. It is noted that in the case of bridges the number of modes that have to be considered is significantly higher than in the case of buildings; in fact, in order to capture all modes whose masses contribute to at least 90% of the total mass of a complex bridge structure (a criterion commonly used in seismic codes) one might need up to a few hundred modes; this issue is further discussed under point 7.

Step 2: Carry out separate pushover analyses for force distribution, $\mathbf{s}_n^* = \mathbf{m}\phi_n$, where $\mathbf{m}$ is the mass matrix of the structure, for each significant mode of the bridge and construct the base shear vs. displacement of the monitoring point (V_{bn}- u_{rn}) pushover curve for each mode. The selection of an appropriate monitoring point for bridges (in buildings it is always the roof) is further discussed in the following (Step 5). Gravity loads are applied before each modal pushover analysis, and P-Δ effects are included. The value of the lateral deck displacement due to gravity loads, u_{rg}, is very small for a bridge with nearly symmetrically distributed gravity loading.

Step 3: A critical issue in MPA is the way that response quantities individually calculated for each mode are superimposed, in the sense that modal contributions should correspond to the same earthquake intensity. Most of the currently available procedures [9], [13] developed for SPA require that the pushover curve be idealized as a bilinear curve (Fig.1), and is then used to appropriately reduce the elastic response spectra representing the seismic action used for assessment. This idealization can be done in a number of ways, some more involved than others; it is suggested to do this once (as opposed, for instance, to the ATC procedure) using the full pushover curve (i.e. analysis up to 'failure' of the structure) and the equal energy absorption rule (equal areas under the actual and the bilinear curve). However, the remaining steps of the MPA procedure can be applied even if a different method for producing a bilinear curve is used.

Step 4: If the popular CSM [3], [13] is to be used for defining the target displacement for certain earthquake intensity, Step 4 consists in converting the idealized V_{bn} - u_{rn} pushover curve of the Multi-Degree-Of-Freedom (MDOF) system (calculated in Step 3) to a 'capacity diagram', as shown in Figure 1. The base shear forces and the corresponding displacements in each pushover curve are converted to spectral accelerations (S_a) and spectral displacements (S_d) of an equivalent Single-Degree-Of-Freedom (SDOF) system, respectively, using the relationships [14].

$$S_{\mathrm{a}} = \frac{V_{bn}}{M_n^*} \tag{1}$$

$$S_d = \frac{u_{rn}}{\Gamma_n \phi_{rn}} \tag{2}$$

wherein $M_n^* = \dfrac{L_n}{M_n}$, is the effective modal mass, ϕ_{rn} is the value of ϕ_{rn} at the monitoring point, and $\Gamma_n = \phi_n^T \mathbf{m1} / \phi_n^T \mathbf{m} \phi_n$, $L_n = \phi_n^T m1$, $M_n = \phi_n^T m \phi_n$, is the generalized mass for the n^th^ natural mode. For elastic behaviour, the intersection of the capacity spectrum and the demand spectrum for each mode provides an estimate of the acceleration (strength) and displacement demand (D_n) for the particular ground motion. The procedure used in the present study is based on the use of inelastic spectra for estimating the target displacement at the monitoring point; this is equally simple, more consistent, and more accurate than the ATC [13] procedure based on reducing the elastic spectra with ductility –dependent damping factors, as shown in a number of studies [14], [15]. For an inelastic SDOF system with a bilinear force-deformation relationship, the inelastic acceleration spectrum (S_a) and displacement spectrum (S_d) are determined as [14]:

$$S_{\mathrm{a}} = \frac{S_{\mathrm{a}el}}{R_\mu} \tag{3}$$

$$S_d = \frac{\mu}{R_\mu} S_{d\,el} = \mu \cdot \left(\frac{T}{2\pi} \right)^2 S_{\mathrm{a}} \tag{4}$$

in which μ is the ductility factor defined as the ratio between the maximum and the yield displacement, and R_μ is the force reduction factor due to ductility. Several proposals have been made for the relationship between the reduction factor R_μ and the ductility factor μ. In this paper the formula proposed by Miranda [16], was used.

$$S_d = C_\mu \cdot S_{d\,el} \tag{5a}$$

wherein:

$$C_\mu = \left[1 + \left(\frac{1}{\mu} - 1 \right) \cdot e^{-12T\mu^{-0.8}} \right]^{-1} \tag{5b}$$

but several other options might also do. It is recalled that the relevant intersection point (of the demand and the capacity diagrams) is the one for which the ductility factor calculated from the capacity diagram matches the ductility value associated with the intersecting demand curve (inelastic spectrum). In many bridges the dominant modes correspond to long fundamental natural periods, and for those the equal displacement rule applies, hence there is no need for iterating to arrive at inelastic spectra consistent with the ductility demand; however, in general some iteration is required, at least for higher modes. An alternative, computationally more demanding, procedure was used (for buildings) by Chopra and Goel, [7], wherein for a SDOF system with known T_n and ζ_n, the displacement demand, D_n, can be calculated from a nonlinear time-history analysis for the given motion.

Step 5: Since the target displacements calculated in Step 4 refer to SDOF systems (with periods equal to those of the corresponding modes), an important step is to correlate these displacements to those of the actual bridge. Hence, Step 5 consists in converting the displacement demand of the n^{th} mode inelastic SDOF system to the peak displacement of the monitoring point, u_{rn} of the bridge using the relationship:

$$u_{rn} = \Gamma_n \phi_{rn} D_n \tag{6}$$

Natural choices for the monitoring point in a bridge are the deck mass centre, or the top of the nearest to it pier. More consistently, it can be selected as the point of the deck which corresponds to the location of the equivalent SDOF system. The position (x_n^*) of the equivalent SDOF system along the longitudinal axis of the bridge can be calculated from the properties of the MDOF system using the following equation [17]:

$$x_n^* = \frac{\sum_{j=1}^{N} x_j m_j \phi_{jn}}{\sum_{j=1}^{N} m_j \phi_{jn}} \tag{7}$$

in which, x_j is the distance of the j^{th} mass from the (selected) end of the MDOF system, and ϕ_{jn} is the value of ϕ_n at the j^{th} mass. It is noted that x_n^* is essentially independent of the way the mode is normalized. The monitoring point of the bridge may also lie at the point of the deck where the displacement is maximum, or the top of the pier which exhibits the most critical plastic rotation (θ_p), and it doesn't have to be the same for all individual analyses (i.e. for all modes). An initial analysis of the structure for each mode is required in this case, to define the corresponding most critical location, but even this extra effort is not always enough, since the location of the critical point might change as the bridge enters the inelastic range and the contribution of each mode possibly changes.

Step 6: The response quantities of interest (e.g. displacements or rotations) 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 reference point is equal to u_{rn} (equation 6).

Step 7: Steps 3-6 are repeated for as many modes as required for sufficient accuracy. Judging the latter is far from straightforward in the case of bridges. As mentioned in Step 1, capturing all modes whose masses add up to 90% of the total mass of a complex bridge structure might need considering up to a few hundred modes (as an example, modal spectral analysis of the 1036 m long, 8-span, Arachthos bridge, to be constructed in Heperus, Greece, involved consideration of 450 modes, to capture 90% or more of the total mass in all directions, including the vertical one). On the other hand, work carried out within the present study (partly reported herein) has shown that there is little merit in adding modes whose participation factor is very low (say less than 1%), and less rigid rules than the 90% one (calibrated only for buildings) could be adopted.

Step 8: The total response for any desired quantity is determine by combining the peak 'modal' responses, r_{no} using an appropriate modal combination rule, e.g. the SRSS rule combination rule:

$$r_o = \left(\sum_{n=1}^{N} r_{no}{}^2 \right)^{1/2} \tag{8}$$

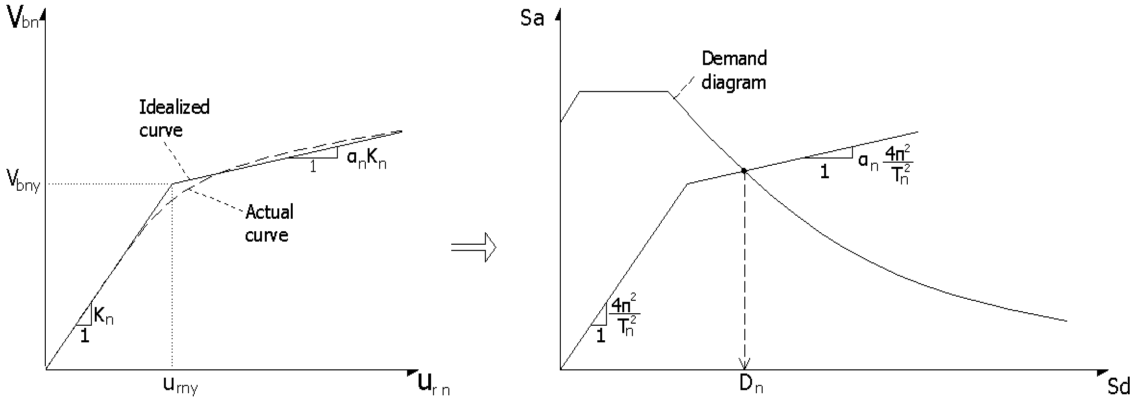


Figure 1: Idealized pushover curve of the n^{th} mode of the MDOF system, and corresponding capacity curve for the n^{th} mode of the equivalent inelastic SDOF system

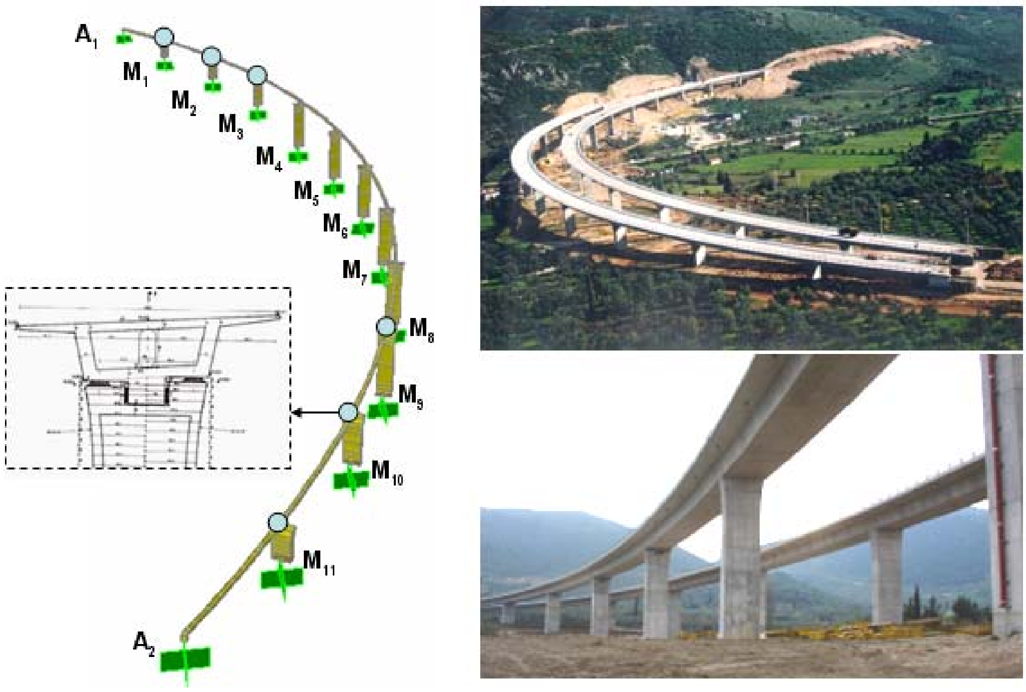


Figure 2: Layout of the bridge configuration and finite element modelling

SEISMIC ASSESSMENT OF A COMPLEX BRIDGE USING ALTERNATIVE INELASTIC ANALYSIS PROCEDURES

Description of structure studied

In order to investigate the accuracy and also the practicality of the proposed procedure it was deemed appropriate to apply it on an actual bridge structure, whose complexity hints to increased contribution of higher modes. For this purpose, the Krystallopigi bridge was selected, a twelve span structure of 638m total length (Fig. 2) that crosses a valley in northern Greece. The curvature in plan (radius equal to 488m) of the bridge adds to the expected complexity of its dynamic behaviour. The deck consists of a 13m wide prestressed concrete box girder section (see insert in Fig. 2). The concrete grade of the deck is B45 (characteristic cylinder strength f_{ck}=35 MPa) and the prestressing steel grade

1570/1770 (yield stress f_y =1570 MPa). Piers are rectangular hollow reinforced concrete members (concrete grade B35 with f_{ck}=27.5MPa and steel grade BSt500s of f_y=500 MPa). For abutments and foundations B25 (f_{ck}=20 MPa) and BSt500s are used.

The Greek Seismic Code - EAK2000 design spectrum [18] scaled to 0.24g was used for seismic design. The design spectrum corresponded to soil conditions category 'B' of the Greek Seismic Code, which can be deemed equivalent to subsoil class 'B' of the ENV version of Eurocode 8 [19]. A behaviour factor of 3.0 was adopted for design, i.e. the bridge was designed as a ductile structure (plastic hinges expected in the piers). As is common in valley-crossing viaducts, the pier height varies along the length. In particular, the structure is supported on 11 piers of heights between 11 and 27m. For the end piers M1, M2, M3, M9, M10, M11 (see Fig. 2) a bearing-type pier-to-deck connection is adopted, while the interior (taller) piers are monolithically connected to the deck. It is noted that for practical reasons (i.e. anchorage of the prestressing cables) the initial 0.50x0.20m pier section is widened to 0.70x0.20m at the pier top. The piers are supported on pile groups of length and configuration that differs between supports due to the change of the soil profile along the bridge axis.

The bridge was assessed using inelastic 'standard' and modal pushover analysis (the target spectrum for both pushover analyses was the design one, or multiples of it) as well as non-linear time history analysis, for artificial records closely matching the design spectrum. Analyses were carried out using the SAP2000 program [20]; additional verification of results was made with extra analyses using the general purpose FE program ANSYS [21]. The reference FE model involved 220 non-prismatic 3D beam elements, while appropriate nonlinear links and hinges were employed for time history and static inelastic analyses, respectively. For the piers connected to the deck through bearings, the movement along the longitudinal axis, as well as the rotation around both the longitudinal and transverse axis, were unrestrained. On the contrary, the presence of shear keys (Fig. 2) resulted in the restraint of transverse displacements, as well as displacement along, and rotation about, the vertical axis. For the pushover analyses, the inelastic behaviour was simulated through software built-in plastic hinges, whereas for the case of time-history analysis, a compatible lumped plasticity model (i.e. nonlinear links) was employed [22].

In the analyses presented in the following, the focus is on the transverse response of the bridge, as it is well known (e.g. [12]), that this is the response most affected by higher modes; additional analyses in the longitudinal direction are also briefly presented. Soil-structure interaction was accounted for in previous studies [23], through the use of an appropriate foundation stiffness matrix. Due to the relatively stiff soil formations underneath the structure, SSI was found to little affect the response (no more than 15%), hence it was considered that the relative evaluation of the three types of nonlinear analysis is not affected by SSI effects, which are subsequently ignored herein.

'Standard' pushover analysis (SPA)

A fundamental mode-based ('standard') pushover analysis was first performed, to serve as the reference (i.e. the least demanding procedure) for assessing the inelastic response of the bridge studied. It is worth noting that unlike the case of buildings, wherein the pushover curve is generally defined in terms of base shear vs. top displacement (in the direction under consideration), in bridges the shape of the pushover curve depends on the pier on which the monitoring point is located (particularly when piers are of unequal height, as in the bridge studied). The displacement of the monitoring point is used not only as a parameter of the capacity curve, but also to establish the seismic demand along the structure at the estimated peak displacement (Step 5). In the case of Krystallopigi bridge, the monitoring point was initially selected as the upper joint of the central pier M6 of the bridge, which practically coincided with the mass centre of the structure.

The typical pushover curve that was calculated by applying the modal load pattern of the 1st mode in the transverse direction of the bridge is shown in Fig. 3, referring to the central pier M6; a similar curve was also derived for the longitudinal direction. As discussed in more detail elsewhere [23], the overall performance of the bridge was very satisfactory, since neither local nor global failure was predicted, even under seismic actions that far exceed the design level. The sequence of plastic hinge formation (along with the force (V/W) - displacement (δ/H) curves), was also derived for both the longitudinal and transverse direction of the bridge. It is noted that plastic hinge formation is much closer to being simultaneous in the longitudinal direction in which the bridge behaves much more like a SDOF, than in the transverse one, wherein hinging is also affected by higher modes and takes places at distinctly different stages of the response, something which is also manifested by the much steeper slope of the yielding branch of the bilinear curve ('strain-hardening') in this direction. In addition to the first mode loading pattern, wherein the force at each node is proportional to the modal displacement and the corresponding nodal concentrated mass (and acts in the direction of the modal displacement), for comparison purposes, the alternative pattern of uniform loading was also used in SPA; according to this pattern, the force acting at each node is proportional to the nodal mass in the direction of the control displacement. This pattern is usually required by codes for pushover analysis of buildings [9], mainly as a (rough) means of identifying critical combinations of shear and flexure (V/M).

Results indicated that the adoption of a particular loading model for the SPA plays indeed an important role with respect to the inelastic response of the bridge. In particular, when the uniform loading pattern was applied, the overall strength of the system appeared to be higher (i.e. yielding occurred at a higher level of base shear). This trend was found to be clearer in the transverse direction. The more pronounced effect in the transverse direction should be attributed to the fact that, due to the shape of the first transverse mode, the largest displacement corresponds to the middle pier (M6), hence the modal force is higher at that particular pier compared to the rest, whereas in the uniform pattern, forces at all piers are about the same (since masses are similar). As a result, for the same target displacement, higher forces are developed in the latter case. It is interesting to note that a similar overestimation of strength is found in pushover analysis of buildings, but in that case the key reason is the distribution of the lateral loading along the height of the structure (which results in higher overturning moment at the base in the case of modal loading). The above observation is an indication that consideration of higher modes in the MPA procedure could possibly highlight aspects of the inelastic response of the bridge that would be otherwise 'hidden' in 'standard' pushover analysis.

In terms of the overall assessment obtained through the reference approach presented herein, it can be also concluded that displacement ductility was found to be high in both principal directions (i.e. 6.1 in the longitudinal and 4.6 in the transverse direction) due to the significant available curvature ductility at the critical locations of potential plastic hinging.

Modal pushover analysis (MPA)

After obtaining a clear overview of the main aspects of the expected inelastic response of the Krystallopigi bridge using the 'standard' pushover analysis, the MPA method described in the previous section was implemented. The dynamic characteristics required within the context of the MPA approach, were determined using standard eigenvalue analysis. Fig. 4 illustrates the first four transverse mode shapes of the bridge (note that the lowest mode is a longitudinal one, with T=1.46s), together with the corresponding participation factors and mass ratios, as well as the locations (from equation 7) of the equivalent SDOF systems for each mode. It is seen that the modal mass participation factors of higher transverse modes are much lower than that of the fundamental transverse mode, a fact that could be primarily attributed to the curvature of the bridge in plan. Consideration of these four modes assures that more than 90% of the total mass is considered. Applying the modal load pattern of the n^{th} mode in the transverse direction of the bridge, the corresponding pushover curve, involving the displacement of the central pier (M6) top was constructed and then idealized as a bilinear curve. These curves were then converted to capacity curves (shown in Fig. 5 for the four transverse modes) using the procedure described earlier as Step 4. It has to be noted that the capacity curves of Fig. 5, derived using the displacement of the central pier M6 as monitoring point, are not necessarily representative of the actual response of all structural members of the bridge. For instance, the capacity curve (total base shear vs. central pier top displacement) corresponding to modes 3 and 4 is purely linear, hence conveying the impression that the bridge does not enter the inelastic range when subjected to the 3rd or the 4th mode load pattern, even for very high accelerations. In reality, it is only the central pier that responds elastically in those cases, whereas the edge piers *do* enter the inelastic range; this is clearly due to the form of the load pattern of these two higher modes (see lower row of Fig. 4), which is obviously not critical for the central pier.

Another important aspect is the calculation of the target displacements with the use of the Capacity Spectrum Method, according to step 4 of the methodology presented earlier and employing the top of the most critical pier for each particular mode as the reference point. The "peak" modal responses r_{no}, each determined by a pushover analysis, are then combined using an appropriate modal combination rule, to obtain an estimate of the peak value, r_o, of the total response. Figures 6, 7 and 8, illustrate the radial (i.e perpendicular to the tangent to the deck axis) displacements of the top of the piers for the four significant modal patterns along the transverse direction, as well as the total displacements as derived by applying the SRSS combination rule to the quantities corresponding to the target displacement calculated for each mode (which is, of course, different for each mode). To investigate the effect of the level of inelasticity on the calculated response, different levels of excitation were considered, i.e. the design earthquake was multiplied by a factor of 1.0, 1.5 and 2.0.

From the displacement distributions shown in Figures 6, 7 and 8, it is noted that the contribution of the 1st transverse mode to the overall response is significant for the case of the design earthquake but it is reduced as the excitation level increases, since higher modes are participating more actively. Consequently, the difference between the displacement profile calculated from SPA and that from MPA becomes more substantial as the induced level of inelasticity increases. This is a strong indication that the necessity of implementing the MPA is closely related to the magnitude of earthquake forces, as well as to the structural characteristics of the superstructure itself.

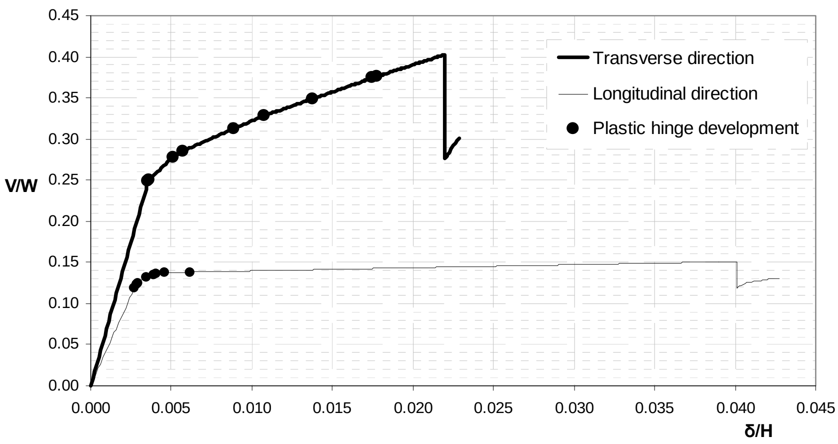
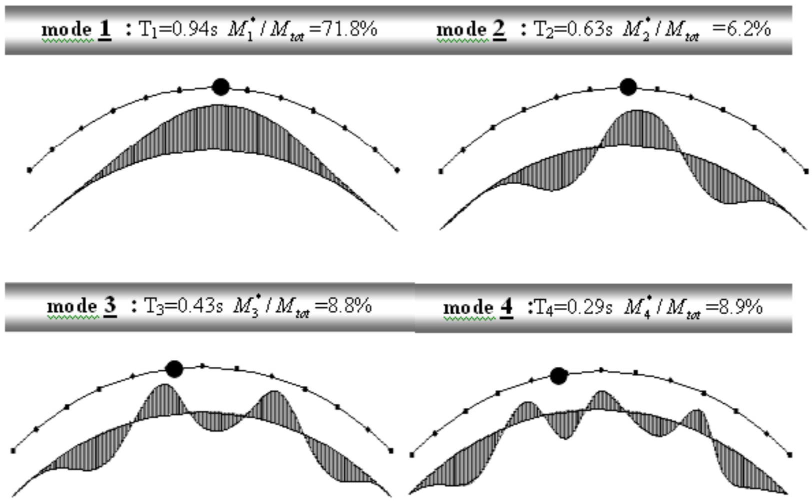


Figure 3: Base shear vs. pier top displacement, and sequence of plastic hinge formation in both directions of the bridge.



	mode **1**	mode **2**	mode **3**	mode **4**
x_n^* **/(total length)**	0.53	0.52	0.44	0.42

Figure 4: Force distribution, $\mathbf{s}_n^* = \mathbf{m}\phi_n$, location of the equivalent SDOF systems and modal parameters for the main transverse modes of the bridge

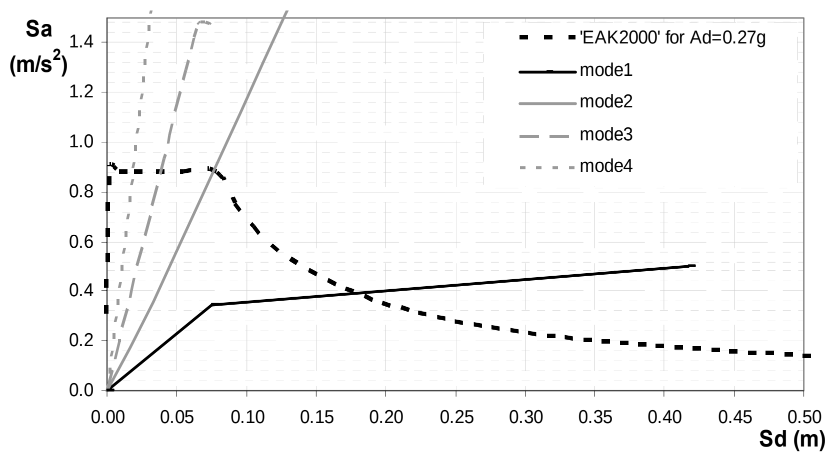


Figure 5: Capacity curves derived with respect to the central pier top for the four transverse modes (the elastic spectrum of the design earthquake is also shown)

Non-linear Time history analysis (NL-THA)

In line with most previous studies, it was deemed necessary to compare results of the ‘standard’ and modal inelastic pushover approaches with those from nonlinear Time History Analysis (NL-THA), the latter assumed to be the most rigorous procedure to compute seismic demand. To this effect, a set of NL-THA’s was performed using 5 artificial records compatible with the EAK2000 elastic spectrum and generated with the use of the computer code ASING [24]. The classical Newmark integration method was used (γ=0.5, β=0.25), with time step Δt=0.002s and a total of 10000 steps (20s of input). Since this analysis is considered as the most refined and accurate, it was of particular interest to compare the maximum displacements of the deck calculated from time-history analysis with those corresponding to the target displacement defined through the SPA and the MPA approach. A comparative evaluation of the three approaches is presented in the following.

Evaluation of different procedures

When assessing the feasibility of MPA, it has to be noted that the procedure is based on two principal approximations: (a) the coupling among modal coordinates associated with the modes of the corresponding linear system, arising from the yielding of the system, is essentially neglected, and (b) the estimate of the total response is obtained by combining the ‘peak’ modal responses using a statistical combination rule. In order to investigate the potential implications of each of these approximations, the bridge was first analysed elastically, using the SPA, MPA and the THA procedure, assuming elastic response in all cases, while gravity loads were included in the analyses.

The peak displacements of the top piers calculated from each analysis are shown in Fig. 9. It is observed that, consistently with the mode shapes depicted in Fig. 4, the main difference between displacements calculated from THA and those from the two static methods is towards the abutments of the bridge, with differences diminishing in the area of the central piers (an area dominated by the first mode). MPA which accounts for the other three transverse modes is much closer to THA in the end areas of the bridge, but some differences persist, possibly indicating a bias in the MPA procedure due to the estimation of the total response by using the modal combination rule (SRSS); as pointed out by Fischinger et al. [12], other rules that dully account for the sign of the response quantities (not only their magnitude) might be more appropriate in this case. The pier displacements determined by the SPA and the MPA procedures were compared to those from the nonlinear time history analysis for increasing levels of earthquake excitation, as shown in Figures 10, 12 and 13. It has to be noted that the pier top displacements were expressed as the average of the maximum pier top displacements that the structure exhibited during the 5 time history analyses (denoted as NL-THA). From Figure 10 it is observed that MPA predicts very well the maximum transverse displacement of the bridge compared to the more accurate approach of NL-THA (15.0cm, compared to 15.6 cm predicted by NL-THA). On the other hand both pushover analysis procedures under-estimate the displacements of piers M3, M4, M5 and M6 with respect to the more refined NL-THA approach. It is also interesting to note that as the level of excitation increases and higher mode contributions become more significant, the displacement profile derived by the MPA method tends to match that obtained by the NL-THA, whereas SPA’s predictions become less accurate as the level of inelasticity increases.

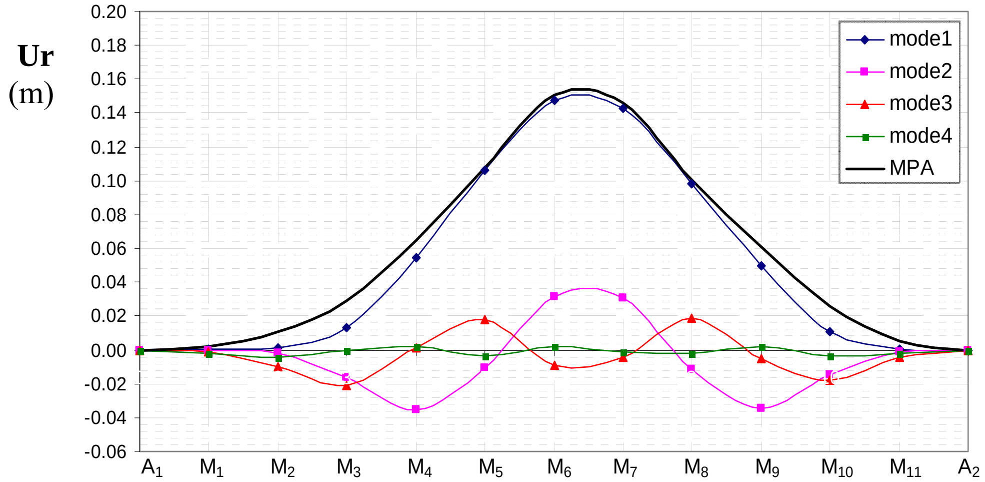


Figure 6: Pier top displacements according to MPA procedure for the design earthquake intensity (excitation in transverse direction only)

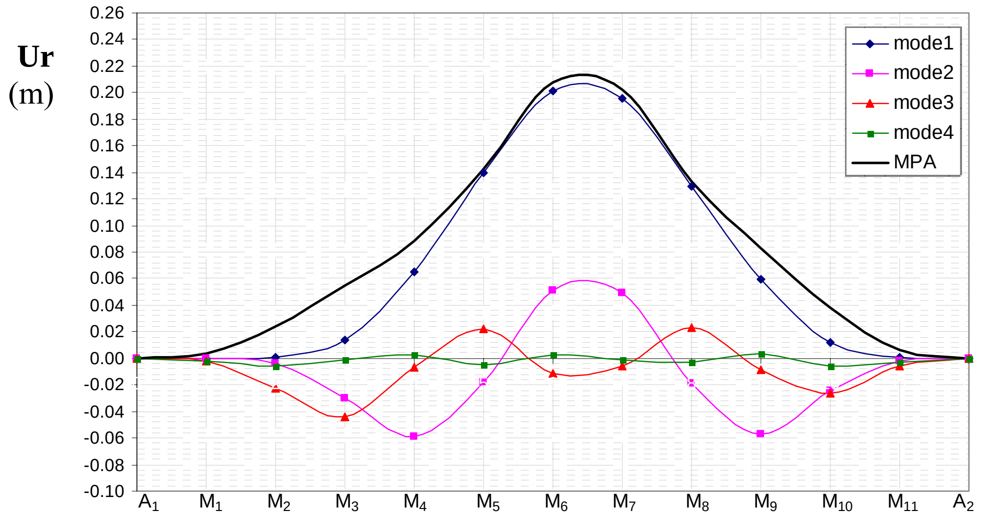


Figure 7: Pier top displacements according to MPA procedure for 1.5 times the design earthquake intensity (transverse direction only)

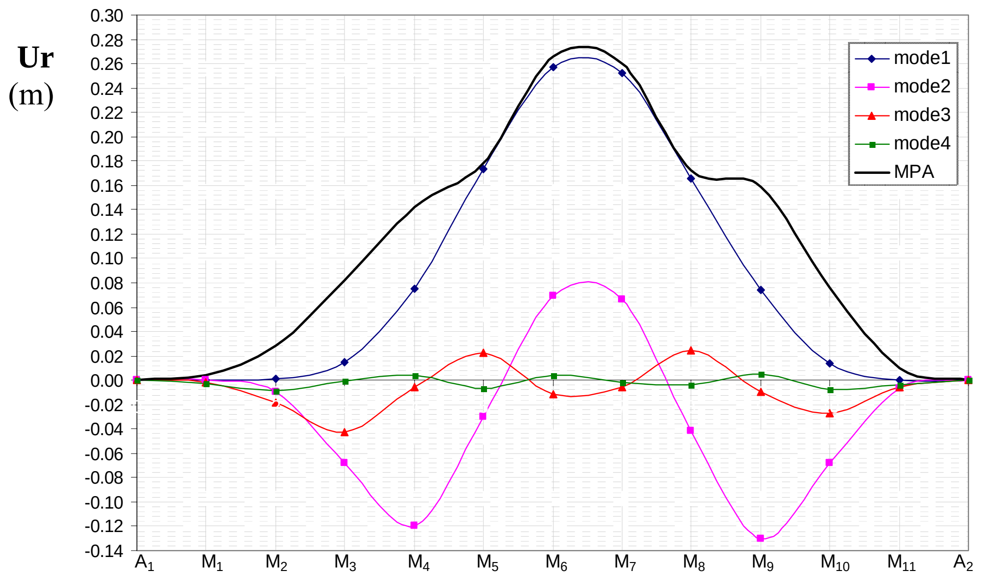


Figure 8: Pier top displacements according to MPA procedure for twice the design earthquake intensity. (transverse direction only)

Especially for. twice the design earthquake intensity case presented in Fig. 13, consideration of the higher modes with the proposed MPA scheme, significantly improves the accuracy of the predicted displacements. Another significant advantage of the MPA method is that it is able to capture the plastic hinge development (Fig. 11) at piers M2, M3, M4, M9 and M10 something the SPA failed to do, hence, the overall degree of agreement between MPA and NL-THA is deemed very satisfactory. SPA effectiveness is similar to that of the MPA method only in the case of the design earthquake where both methods capture well the inelastic behaviour of the central piers. It is noted that these are the piers (together with M8) that are connected to the deck through a bearing-type connection – a possible indication that the structural configuration plays a predominant role regarding the number of modes that should be considered in each particular case.

Notwithstanding the previous considerations, a complete matching between the two methods (pushover and time-history) is not feasible for a number of reasons. In fact, through NL-THA all structural modes (transverse, longitudinal, and vertical) are considered for the evaluation of the non-linear response, whereas only four transverse modes are used in the MPA method and just a single mode in the 'standard' pushover analysis. Moreover, it is shown that the degree of effectiveness of the MPA method is dependent on both the earthquake force level and the degree of participation of the fundamental mode to the overall response. Based on the above observations, it can be stated that the MPA method (as well as the 'standard' pushover for the particular bridge) both provide a good estimate of the nonlinear pier top displacements of the central piers in the transverse direction, whereas only the MPA predicts sufficiently accurately the overall displacement pattern of the bridge and it also provides a good estimate of the plastic hinge distribution and the corresponding energy dissipation mechanisms indicated by the NL-THA.

CONCLUSIONS

A methodology was proposed for Modal Pushover Analysis (MPA) of bridges, and its feasibility and accuracy were investigated by applying it to an actual long and curved bridge, designed to modern seismic practice. By analysing the structure using inelastic 'standard' (SPA) and modal (MPA) pushover analysis, as well nonlinear time history analysis (NL-THA), it was concluded that:

- At least for the studied structure, which is complex but properly designed, all three methods yield similar maximum pier top inelastic displacements although their pattern is rather different.
- The SPA method predicts well the displacements only in the central, first mode dominated, area of the bridge. On the contrary, MPA provides a significantly improved estimate with respect to the maximum displacement pattern, reasonably matching the results of the more refined NL-THA analysis, even for increasing levels of earthquake loading that trigger increased contribution of higher modes.
- On the basis of the results obtained for the studied bridge structure, MPA seems to be a promising approach that yields more accurate results compared to the 'standard' pushover, without requiring the high computational cost of the NL-THA, or of other proposals involving multiple eigenvalue analyses of the structure to define improved loading patterns in the inelastic range.
- Further work is clearly required, to further investigate the effectiveness of MPA by extending its application to bridge structures with different configuration, degree of irregularity and dynamic characteristics, especially in terms of higher mode significance, since MPA is expected to be even more valuable for the assessment of the actual inelastic response of bridges with significant higher modes.

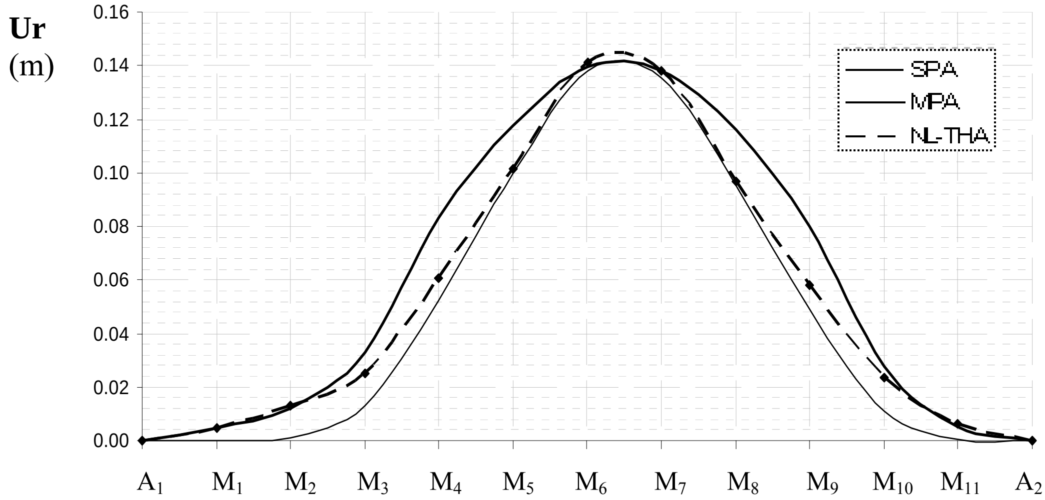


Figure 9: Pier top displacements calculated from SPA, MPA and THA, for elastic behaviour

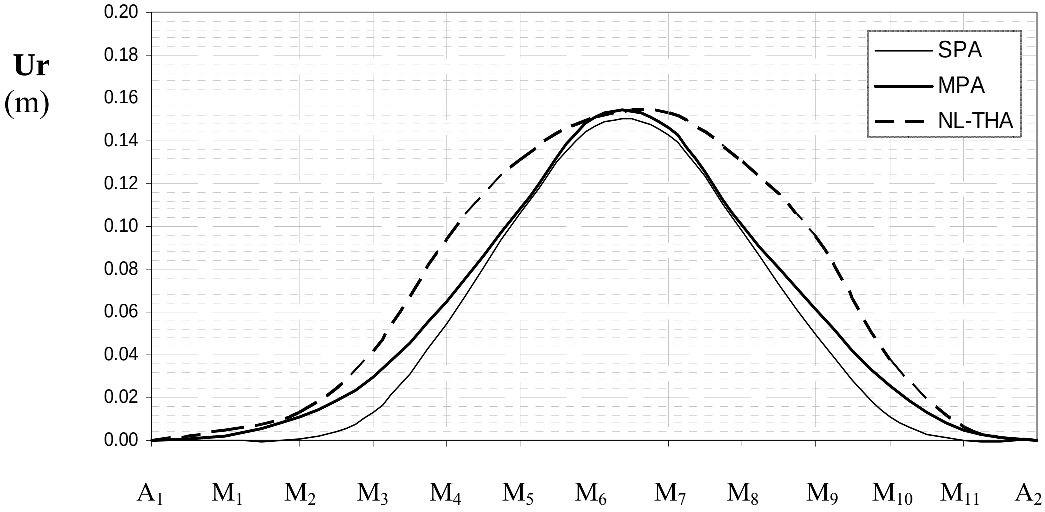


Figure 10: Pier top displacements by SPA, MPA and NL-THA. (transverse direction only), for the design earthquake intensity.

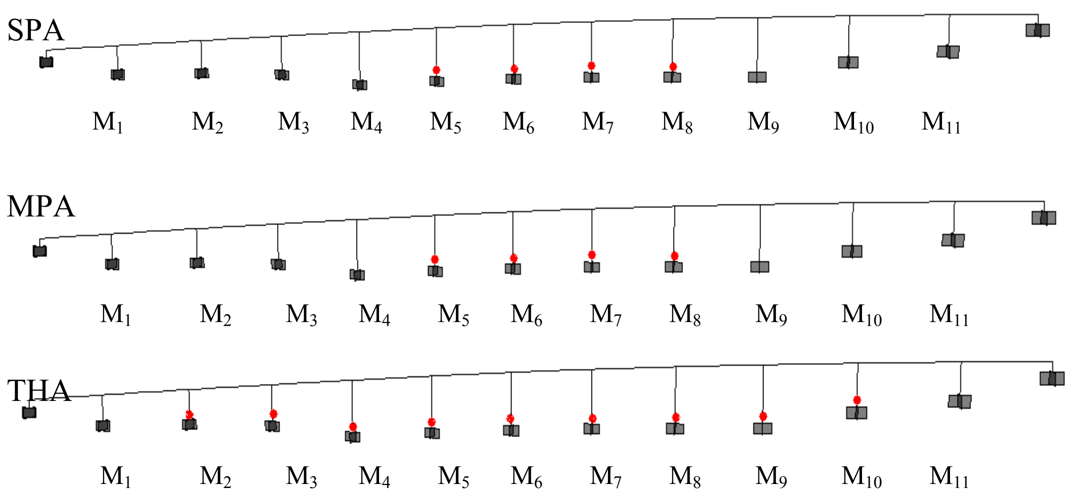


Figure 11: Location of plastic hinges determined by (a) 'standard' pushover analysis (b) MPA considering four 'modes' and (c) by non-linear time history analysis

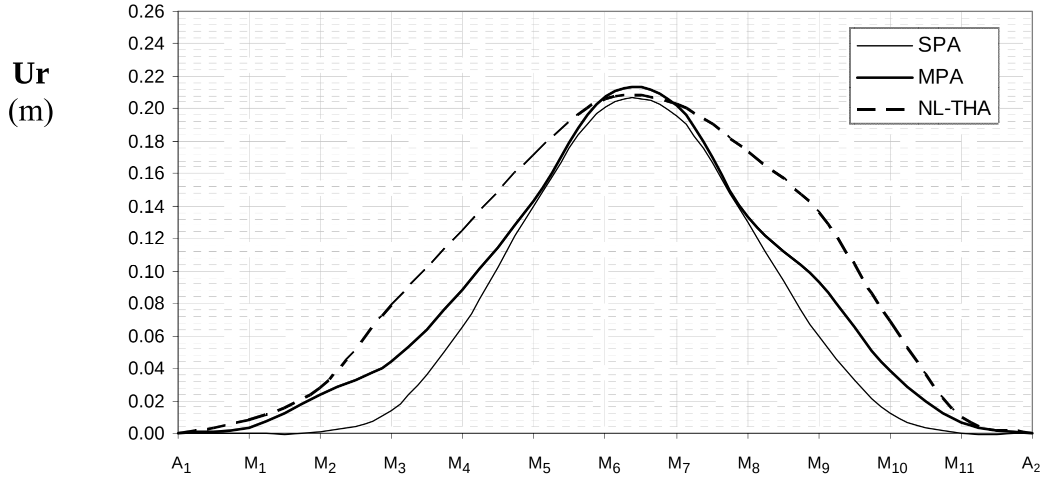


Figure 12: Pier top displacements by SPA, MPA and NL-THA. (transverse direction only), for 1.5 times the design earthquake intensity.

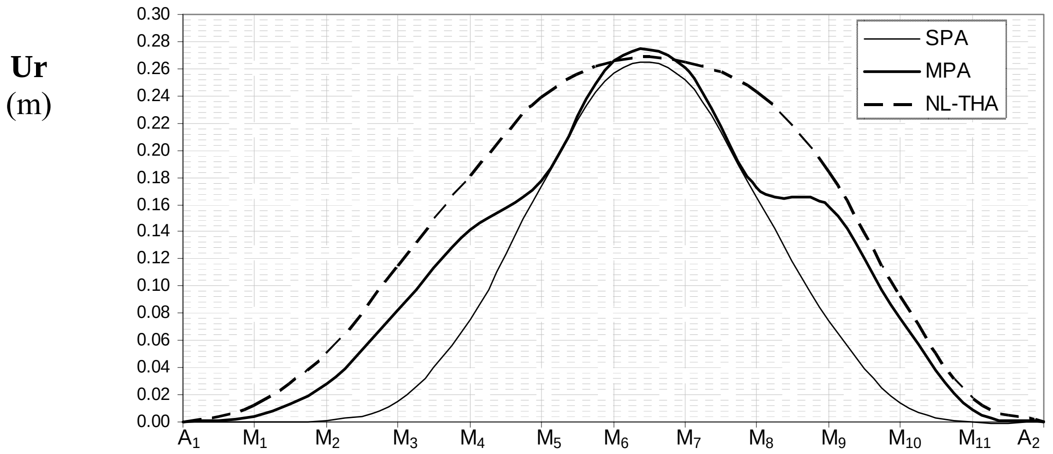


Figure 13: Pier top displacements by SPA, MPA and NL-THA (transverse direction only), for twice the design earthquake intensity.

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