Seismic assessment of a major bridge using modal pushover analysis and dynamic time-history analysis

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Summary

Modal Pushover Analysis has been shown to be a significant improvement compared to the pushover analysis procedures currently used for buildings. This work investigates the extension, applicability and accuracy of the method for a different case of structures, namely bridges. The structure analysed to illustrate the above is a long and curved, twelve span bridge structure, currently under construction in Greece. The bridge is designed according to current seismic codes and then assessed for motions up to twice the design earthquake intensity. Its performance in the transverse direction is herein evaluated through 'standard' and modal pushover, as well as non-linear time history, analysis. The behaviour is found to be satisfactory regardless of analysis method, but the estimated performance slightly varies depending on the analysis approach adopted.

Introduction

Non-linear static (pushover) analysis, is a widely used assessment tool that allows the evaluation of the structural behaviour in the inelastic range and the identification of failure mechanisms, while it highlights the critical points of structural weaknesses. Nevertheless, the inherent assumption is made that structural performance is controlled by the fundamental mode. In particular, the structure is subjected to monotonically increasing lateral forces having a constant pattern until a predetermined target displacement is reached. As a result, both the invariant force distributions and the target displacement, do not account for higher mode contribution, or for potential redistribution of inertia forces due to structural yielding, thus limiting the application of the approach to cases where the fundamental mode is dominant. To overcome the aforementioned limitations, a Modal Pushover Analysis (MPA) procedure has been developed ([1], [2]) wherein the higher mode effect is taken into account (in an approximate way) but despite the substantial amount of work that has been performed for buildings, the corresponding work on bridges is relatively limited ([3], [4]). It is therefore still open to research whether the non-linear performance of a bridge can be accurately assessed using 'standard' or a more refined (modal) pushover analyses, or whether a complete non-linear dynamic analysis in the time domain is required.

The present study describes the general framework of modal pushover analysis and its application to a bridge of complex configuration, in order to highlight potential differences between the three possible types of non-linear analysis, hence shed some light on the feasibility of MPA for bridges.

Overview of the adopted methology

Modal Pushover Analysis is considered as an extension of the 'standard' (single-mode) 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 mode shape amplitudes. Modal pushover curves are then plotted and can be converted

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to SDOF capacity diagrams using modal conversion parameters based on the same shapes. Seismic demands, i.e., peak response quantities are separately estimated for each individual mode and finally combined with an appropriate modal combination rule. In particular, the basic steps [1] of the method are summarized below:

1. Compute the natural periods, T_n, and modes $\boldsymbol{\varphi_n}$, for linearly- elastic vibration of the structure.
2. Construct the (base-shear) – (displacement of the joint of control), (V_{bn}- u_{rn}), pushover curve for the *n*th-mode force distribution $\mathbf{s_n^*=m\varphi_n}$. Gravity loads are applied before the first-mode pushover analysis. P-Δ effects are also included.
3. Idealize the pushover curve as a bilinear curve
4. Convert the idealized pushover curve to the force – deformation (F_{sn}/L_n- D_n) relationship of the *n*th-mode inelastic SDF system.
5. Compute the peak deformation, D_n, of the *n*th-mode inelastic SDF system. The initial vibration period of the system is $T_n=2\pi(L_nD_{ny}/F_{ny})^{1/2}$. For a SDF system with known T_n, ζ_n, and force- deformation relationship, D_n for given ground motion can be computed by non-linear Response History Analysis.
6. Calculate the peak displacement of the joint of control, u_{rn}, associated with the *n*th-mode inelastic SDF system from

   $$u_{rn}=\Gamma_n\varphi_{rn}D_n \tag{1}$$

   In practical application, for the evaluation of the target displacement, the bridge is analysed in the transverse direction, using the elastic response spectra analysis. Then the target displacement can be evaluated from the elastic displacement of the deck mass centre (or the top of the middle pier) in the direction under consideration.
7. From the pushover results at the displacement u_{rn}, extract values of desired response r_n: displacements of the upper joint of the piers, plastic hinge rotations, etc.
8. Repeat steps 3 to 7 for the second mode in the transverse direction.
9. Determine the total response (demand) r_{MPA} by combining the peak ‘modal’ responses using an appropriate modal combination rule, e.g. the SRSS combination rule:

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

Description of structure studied

The selected bridge of complex configuration is the Krystallopigi bridge, a twelve span structure of 638m total length (Fig. 1) that crosses a valley, as a part of the 680 km Egnatia motorway in northern Greece. The curvature radius is equal to 488m, while its deck width is 13m. The slope of the deck and the pier height vary along the length. The deck is a prestressed at its top flange concrete box girder section; concrete grade is B45 (characteristic cylinder strength f_{ck}=35 MPa) and prestressing steel grade 1570/1770 (f_y =1570 MPa). Piers are in reinforced concrete, concrete grade is B35 (f_{ck}=27.5MPa), steel grade Bst500s (f_y=500 MPa). For abutments and foundations B25 (f_{ck}=20 MPa) and Bst500s are used. The structure is supported on piers (M1-M11 in Fig. 1) of height that varies between 11 and 27m. For the end piers M1, M2, M3, M9, M10, M11, a bearing type pier-to-deck connection is adopted (see Fig. 2), while the interior piers are monolithically connected to the deck. It is noted that for practical reasons (i.e. anchorage of the prestressing cables) the initial 0.50×0.20m pier section is flared to 0.70×0.20m at the pier top. Piers are supported on pile groups of length and configuration that differ between supports due to the change of soil profile along the bridge axis.

Finite element analysis was used for the assessment of the linear and non-linear response of the bridge, involving the discretisation of the structure in 220 non-prismatic 3D beam elements (see Fig. 3). 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, are unrestrained. On the contrary, the existence of shear keys results in the prevention of transverse displacements and the movement and rotation along and about the vertical axis. The structure was assessed using response spectrum, 'standard' and modal pushover, and non-linear time history, analyses with the aid of the F.E. program SAP2000 [5]. The inelastic behaviour of the critical cross-sections of the piers was evaluated using the program RCCOLA-90 [6]. For the pushover analyses, the inelastic behaviour was simulated through software built-in plastic hinges (that are available in SAP2000 for pushover analysis only), whereas for the case of time history analysis, a compatible lumped plasticity model (Non-linear links at member ends) was employed. The Greek Code (EAK2000) [7] design spectrum was used as the target spectrum for both pushover analysis and the generation of the (scaled to the design PGA of 0.24g) artificial records required for the analysis in the time domain.

Artificial records were simulated using the code ASING [8]; the very good match between the target spectrum and the mean of the response spectra of the artificially generated motions is clear in Figure 4. Excitation is considered in the transverse direction only, where the corresponding modal vibrations are expected to be relatively more complicated. Due to the different scope of this work, soil-structure interaction and spatial variability of ground motion effects that have been accounted for in previous assessments of the Krystallopigi bridge ([9], [10]) are neglected.

Inelastic static analysis

In the case of buildings, the pushover curve is a plot of the base shear versus the top displacement. This displacement is used in current procedures to establish the seismic demand over the height of the structure (at the estimated peak displacement, or performance point). In the case of Krystallopigi bridge, the displacement control point was selected as the upper joint of the central pier M6 of the bridge (Fig. 1), which practically coincides with the centre of mass of the structure. Selection of the control point for monitoring displacements in complex structural systems, such as bridges in their transverse direction, is an important issue to be further investigated, particularly when piers are of unequal height.

Within the context of the MPA approach, the required dynamic characteristics of the structure were determined using standard eigenvalue analysis. Fig. 5 shows the first three mode shapes for the bridge, while Table 1 lists the periods, participation factors and mass ratios for each significant transverse mode of the structure (mode 1 is a longitudinal one, with T=1.46sec). It is seen that participation of higher transverse modes is not significant, a fact that should be primarily attributed to the curvature of the bridge in plan. The standard pushover curve was then calculated by applying the modal load pattern of the 1st mode in the transverse direction (2nd global mode) of the bridge, and was idealized by a bilinear curve, (shown in Fig. 6), referring to the central pier M6; similar curves are shown in Fig. 6 for the other two modes in the transverse direction, calculated by applying the corresponding modal load patterns. The target displacement was evaluated from the elastic displacement of the upper joint of pier M6, in the direction under consideration. In the spectrum considered, soil conditions correspond to category 'B' of the Greek seismic code (EAK2000), which can be deemed equivalent to subsoil class 'B' of the ENV version of Eurocode 8 [11]. For Zone III of the Greek code used for the design of the bridge, a peak ground acceleration of 0.24g is specified, while a behaviour factor of 3.0 was adopted. The target displacement for the design seismic demand was therefore calculated according to the 1st mode in transverse direction as 129mm (0.5% drift in the central pier) while for the 2nd mode in the same direction, δ_{target}=34mm (0.13% drift). The pier top displacements for all piers were evaluated according to the target displacements of each mode, as discussed previously.

The peak 'modal' responses r_{no}, each determined by a pushover analysis, were then combined using an appropriate modal combination rule, to obtain an estimate of the peak value, r_o, of the total response. It has to be noted that the 3rd transverse mode of vibration is such that failure occurs before pier M6 enters the inelastic range. Fig. 7 presents estimates of the combined displacement response, in the transverse direction, obtained by combining the first three transverse modes using the SRSS rule.

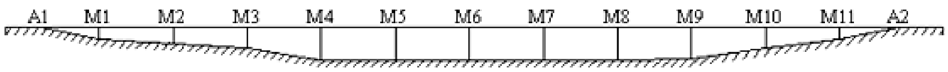

Figure 1: Overview of the Krystallopigi Bridge

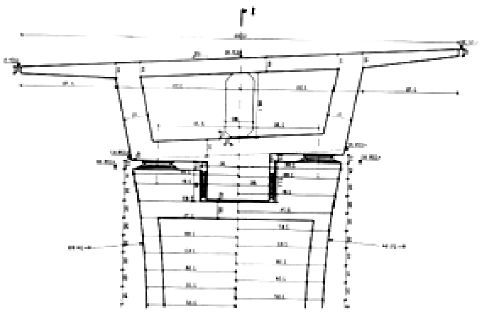

Figure 2: Non-monolithic pier-to-deck connection (end piers)

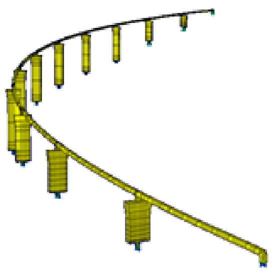

Figure 3: Layout of the 3D Finite element model

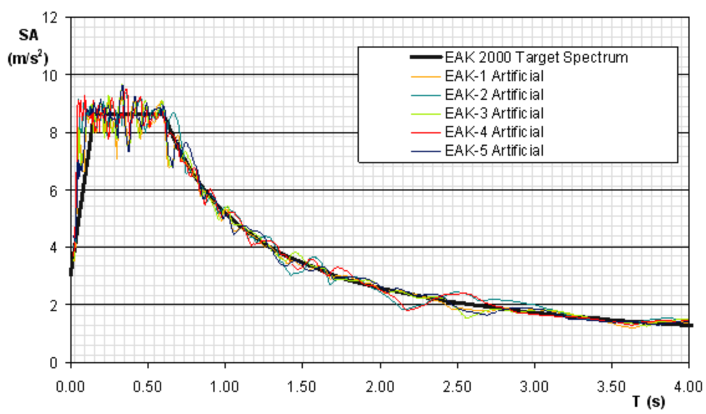

Figure 4: Comparison between target and mean of the artificially generated response spectra

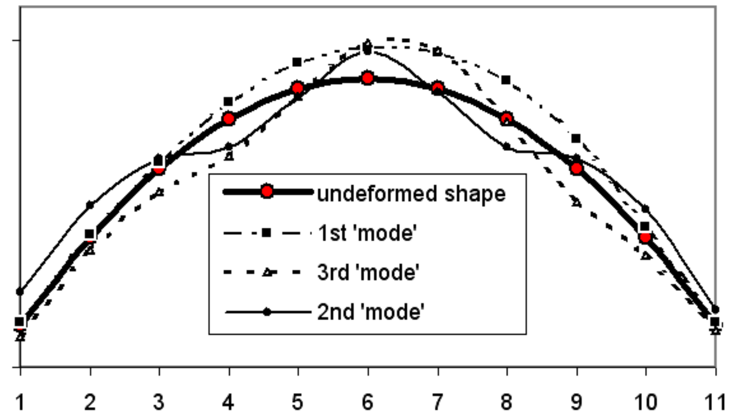

Figure 5: Deformed shape of the bridge for the 1st, 2nd and 3rd mode (in transverse direction)

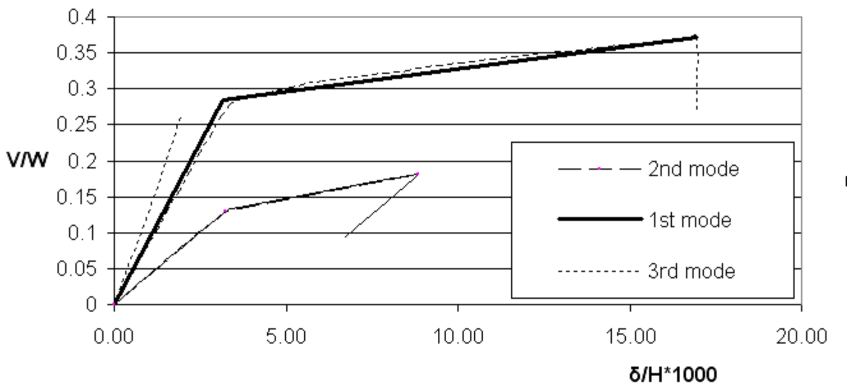

Figure 6: Pushover curves for the first three modes in the transverse direction of the bridge

From Fig, 7 it is observed that, due to the relatively minor participation of the higher transverse modes, in the particular case studied, the ‘standard’ and the modal pushover analyses yield similar results. This observation cannot be generalized and further case studies are currently being considered in this respect.

Non-linear Time history analysis

Having performed the aforementioned non-linear pushover analysis, both the standard and the MPA, it was deemed appropriate to compare inelastic pushover with Non-Linear Time History Analyses (NL-THA), the latter presumed to be the most rigorous procedure to compute seismic demand. Along these lines, a set of NL-THA was performed using 5 artificial records generated to be compatible with the EAK2000 elastic spectrum (see Fig. 4). The pier displacements determined by the MPA procedure and the non-linear time history analysis compare as shown in Fig. 8. It has to be noted that pier top displacements were expressed both as (i) the envelope of the maximum pier top displacements that the structure exhibited during the 5 time history analyses (denoted as NL-THA envelope) and (ii) the displacements that occurred simultaneously with the peak value of top displacement of pier M6 (NL-THA-maxM6) and of pier M8 (NL-THA-maxM8).

From Fig. 8 it is observed that both NL-THA and MPA provide similar maximum displacement of the bridge in the transverse direction, 170 and 181 mm respectively. NL-THA on the other hand, yields higher estimates for the displacements of piers M3, M4, M5 and M6 with respect to the MPA procedure. This should be primarily attributed to the fact that, contrary to what happens in MPA, during NL-THA additional plastic hinges develop at the base of piers M4 and M5 (Fig. 9), leading to relatively higher pier top displacements. The formation of a larger number of plastic hinges in regions of the structure wherein 2nd (or higher) modes play a significant role is a characteristic already observed in time history analysis of buildings (more hinges form in the upper part, compared to the case of pushover analysis). In any case, the difference in terms of maximum pier displacements between the two methods is not very substantial, especially if displacements from MPA (wherein pier M6 is used as the control point) are compared with the displacement pattern that is simultaneous with the maximum displacement at pier M6 (i.e. case NL-THA maxM6). This is a further indication of the contribution of (inelastic) higher modes in NL-THA.

Another interesting point is the fact that both the linear and non-linear, static and dynamic response of the structure is not symmetric, despite the (almost) symmetric shape of the 1st and 2nd mode of vibration. This is the result of the structural irregularity in terms of curvature, bearing location, and pier height, as well as due to the non-symmetric shape of the 3rd mode that is illustrated in Figure 5. The first mode (Fig. 5) is essentially a deck mode, whereas the response of the bridge in both pushover and THA is clearly influenced by the piers too (which are not symmetric and tend to yield following an unsymmetric pattern).

Based on the above observations, it can be stated that the Modal Pushover Method (as well as the ‘standard’ pushover for the particular bridge) provides a good estimate of the non-linear pier top displacements in the transverse direction. Nevertheless, plastic hinge distribution and the corresponding energy dissipation mechanisms that are predicted through the MPA do not exactly match those derived by the NL-THA. It is therefore necessary to investigate the effectiveness of both methods further through the evaluation of their application on bridge structures with different configuration, degree of irregularity and dynamic characteristics.

Table 1: Dynamic characteristics of the bridge for the transverse direction

Mode	Period (s)	Participation Factor (%)	Mass Ratio (%)
2nd	0.91	64.8	64.9
3rd	0.79	2.5	67.4
4th	0.67	6.1	73.5
8th	0.47	8.2	84.2

Conclusions

The work presented herein investigates the feasibility and accuracy of the Modal Pushover Analysis procedure, applied to a bridge of complex configuration. By analysing the structure using inelastic ‘standard’ and modal pushover analysis, as well as non-linear time history analysis, it is concluded that:

- For the particular structure studied, all three methods yield similar maximum pier top inelastic displacements although their pattern is different in some regions.

- Also different is the sequence of plastic hinge formation along the bridge, the general trend being that more hinges form in THA than in pushover analysis, a trend also observed in buildings.
- Further investigation is required, especially for cases that the higher modes play a more significant role on the transverse response of bridge structures.

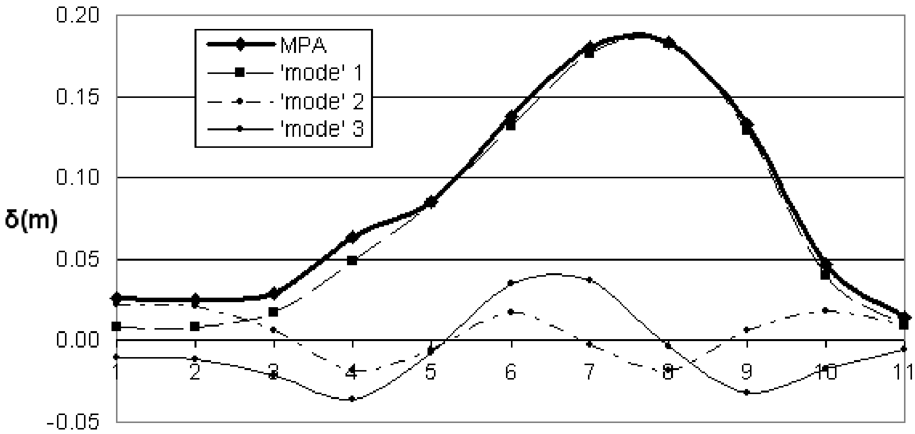

Figure 7: Pier top displacements according to the MPA procedure.

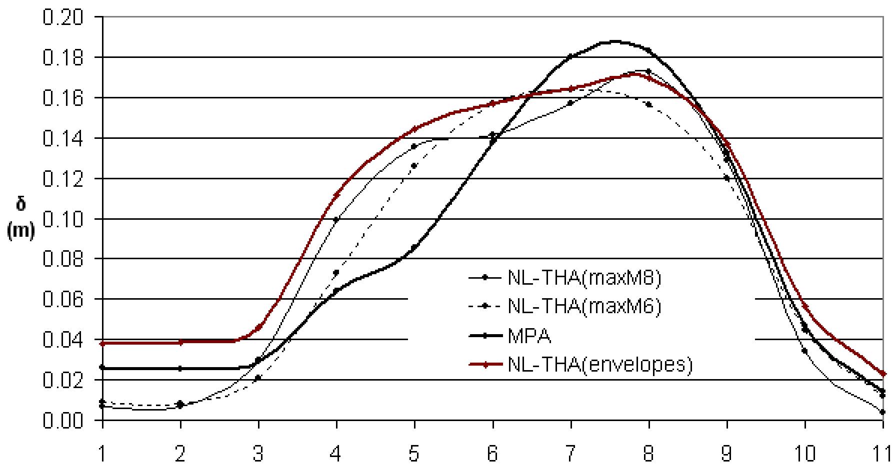

Figure 8: Pier-top displacements by MPA procedure and non-linear time history analysis

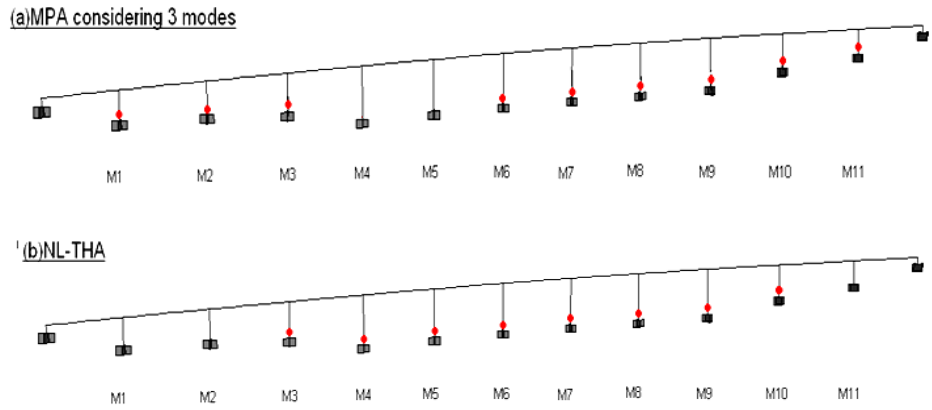

Figure 9: Location of plastic hinges determined by (a) MPA considering three 'modes' and (b) non-linear time history analysis

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