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SEISMIC RESPONSE OF RC BRIDGES USING GENERALISED FORCE VECTORS

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Abstract. *Nonlinear Static Procedures (NSP) are gaining wider use in performance based earthquake engineering practice due to their simplicity compared to rigorous Nonlinear Time History Analysis (NTHA), and being implemented in the new generation seismic design codes. In this study, a recently proposed NSP for the seismic performance assessment of building structures using the concept of Generalized Force Vectors (GFV), is further validated through application to reinforced concrete Bridges. This method, named Generalized Pushover Analysis (GPA), maximizes the response of a chosen parameter during the seismic response, through the use of the GFV, which are a combination of modal forces representing the instantaneous force acting on the system when such parameter reaches its maximum. In this study, the original GPA implemented in buildings is preliminary adapted to a straight bridge structure and four versions of the algorithm are tested and validated. The accuracy of the GPA algorithm is assessed by comparing the nonlinear static results with the "exact" prediction from NTHA. Several levels of seismic hazard, represented by different return periods of the target spectra, are considered to test the accuracy of the method for low and high seismic demands. Furthermore, the GPA results are also compared with a commonly employed NSP, the Capacity Spectrum Method (CSM). The results obtained for the case study suggest that GPA algorithm for bridges is suitable as a NSP approach for the seismic assessment of bridge structures, demonstrating a good fit with NTHA results and superiority with respect to the predictions of the selected traditional NSP.*

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1 INTRODUCTION

Nonlinear Static Procedures (NSP) are relatively simple approaches for the seismic assessment and performance evaluation of structures when compared with the more complex Nonlinear Time History Analysis (NTHA). Whereas NTHA is unanimously considered as the most accurate procedure, and is widely implemented in academia and the research community, it still experiences implementation difficulties in design office environment and daily life structural calculations. This is due mostly to its complexity in terms of modelling, time demand for analysis, post processing of huge output obtained and minimum number of records to use. Furthermore, the modelling requirements of NTHA also imply a set of assumptions that shall be properly substantiated. On the other hand, there is clearly a loss of accuracy in the prediction of the structural response when a NSP is implemented, given that such approaches contain prior assumptions that are not necessarily realized during the true dynamic response. Nevertheless, NSPs are typically advantageous as they do not require complex modelling tools although they do require the modelling of material behaviour in the inelastic range. The time required for analysis is significantly shorter, the analysis can be conducted with a standard nonlinear static analysis software, and the results can be interpreted in a more general way, as with the use of design spectra instead of the acceleration records of particular ground motion components. Although the highlighted aspects regarding a successful NSP analysis must also be scientifically sound, the required low number of assumptions make it inherently more practical and more appealing for common practitioners. The differences among both methods depend highly on the characteristics of the structure, and the eventual loss of accuracy may pay off when considering the easier implementation of the NSP.

There are several NSPs currently available for the seismic assessment of structures. Some of the most commonly employed procedures include the N2 Method [1]; Capacity Spectrum Method (CSM) [2,3], Modal Pushover Analysis (MPA) and its variant, the Modified Modal Pushover Analysis (MMPA) [4,5]; Incremental Response Spectrum Analysis (IRSA) [6]; Adaptive Capacity Spectrum Method (ACSM) [7-9]; Modified Adaptive Modal Combination Method (M-AMC) [10]; and Multi-Mode Pushover Analysis using Generalized Force Vectors, or Generalized Pushover Analysis (GPA) [11]. All these procedures can be distinguished based on: (i) the use of single-mode based conventional pushover analysis, such as N2 and CSM; (ii) the use of multi-mode based conventional pushover analysis, such as MPA, MMPA, IRSA and GPA; and (iii) the use of multi-mode adaptive pushover analysis, such as ACSM and M-AMC.

The GPA procedure, which is the focus of this paper, uses Generalized Force Vectors (GFV), a combination of modal forces representing the instantaneous force acting on the system when a given response parameter reaches its maximum value during seismic excitation. The premise of GPA procedure is that when a GFV is applied on an elastic system, seismic response is the same as that computed from THA when the chosen response parameter reaches its maximum; and in the same way, when a GFV is applied on an inelastic system, the seismic response can be approximated to that computed with NTHA when the chosen response parameter reaches its maximum. Since a GFV maximizes the response of a given parameter, n different GFVs are required to estimate the maximum response of n parameters. As such, n conventional pushover analyses, applying incrementally each of the GFVs, are required to estimate the response of the structure. Interstorey drift is considered as the basic response parameter for building frames in GPA. A modal combination rule is not required for obtaining the response parameters in GPA, which is a significant advantage over the other multi-mode NSPs. Furthermore, GPA can be implemented with a standard nonlinear static analysis software. This is not possible for adaptive NSPs.

The major distinction between the aforementioned methods lies on the complexity of the pushover algorithm employed. It goes from conventional to adaptive – the former is currently more implemented in commercial software whereas adaptive pushover algorithms are still not common in widely used structural analysis software, which renders them less attractive to practitioners [12]. Correspondingly, the NSPs that incorporate higher mode effects provide a response that is more accurate, especially in structures where higher mode contributions are expected to be important [13-15]. Single mode based NSPs on the other hand typically present shortcomings for structures where higher mode effects are important, but they do tend to provide acceptable or good estimates for regular structures and may represent a good compromise for large scale assessment studies [16-17].

The main goal of the study presented herein is to preliminary assess the performance of the GPA method, a recently proposed nonlinear static procedure, for the seismic response estimation of a bridge structure. The results from the GPA are compared with results from NTHA and with a widely-implemented counterpart, the CSM, in order to assess the potential impact of GPA in current practice.

2 GENERALIZED FORCE VECTORS FOR BRIDGE SEISMIC ANALYSIS

The concept of Generalized Force Vectors has been originally conceived and validated for buildings. Bridge structures, on the other hand, are different, particularly for what concerns seismic behaviour. In building structures, the seismic energy dissipation through hysteresis takes place mostly in beams and in the base of columns or shear walls, whereas in bridges it is essentially concentrated at the piers, whilst the superstructure is usually protected. Buildings are typically modelled as "cantilever" MDOF systems, where the masses are lumped at the centre of mass of each floor and the lateral stiffness is computed according to the contributions of vertical, horizontal and/or tilted elements. For straight in-plan bridges, seismic response can be analysed in the two main directions. In the longitudinal direction, bridges usually can be represented as SDOF systems dominated by displacement capacity of the shortest pier [18]. Concerning the transverse direction, the mass of the deck is usually lumped at the pier's top and the higher modes of vibration gain particular importance. For this reason, it is for this direction that a multi-mode procedure is worth to be investigated and the subsequent part of this study will be directed to the implementation of GPA procedure in bridges in the transverse direction. Figure 1 shows the idealized modal response in bridges.

Specific support conditions in bridges (related to bearings, mechanical behaviour of abutments or soil-structure interaction) have a major impact on the seismic response of the structure thus the GPA procedure proposed in this study is tested over models with the following characteristics: support conditions at piers and abutments are fixed, with exception, for the latter, for some specific rotational degrees of freedom (the abutment is modelled as a node with the corresponding degrees of freedom, fixed or released). Furthermore, the connectivity between the deck and the superstructure is modelled by enabling transfer of all forces, or by releasing the degree of freedom for which there is no load transfer (relative displacement is not permitted between deck and piers). In addition, depending on the degree of refinement sought in the analysis, or if performance assessment requirements demand to, the connectivity between the deck and piers can be modelled with link elements, reproducing the behaviour of bearings, shear keys, or any other device that connects the deck with the substructure; similar type of links are also employed for the abutment behaviour. In this study, link elements were implemented for modelling the abutment behaviour and the connection between the deck and the piers.

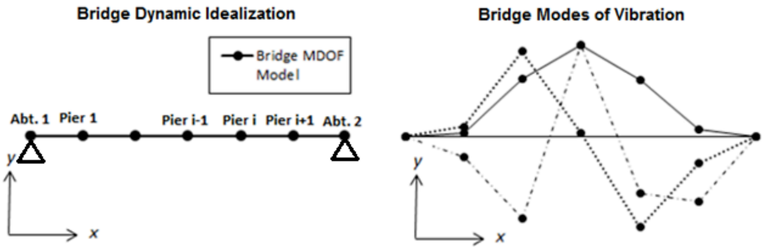

Figure 1: "Dynamic" Behaviour of building and bridges.

In order to adapt the GPA algorithm to bridges, let a multi-span bridge in Figure 2 be considered. The effective force vector acting on the structure when an arbitrarily chosen response parameter reaches its maximum value is given by Equation (1).

$$\{f(t_{\max})\} = \sum_n \{f_n(t_{\max})\} \tag{1}$$

The n^{th}-mode effective vector in Equation (1) at $t_{\max}$ is given in Equation (2), in which $A_n(t_{\max})$ is defined in Equation (3).

$$\{f_n(t_{\max})\} = \Gamma_n [M]\{\phi_n\} A_n(t_{\max}) \tag{2}$$

$$A_n(t_{\max}) = \omega_n^{\,2} D_n(t_{\max}) \tag{3}$$

In Equation (2), $\Gamma_n = L_n / M_n$ is the modal participation factor, with $L_n = \{\phi_n\}^T [M]\{1\}$ and $M_n = \{\phi_n\}^T [M]\{\phi_n\}$. $\{\phi_n\}$ is the n^{th}-mode eigenvector or n^{th} modal shape, $[M]$ is the diagonal mass matrix, and $\{1\}$ is the influence vector. Depending on the parameter that is chosen to be maximized, the GPA approach will be applied to bridges in two different versions.

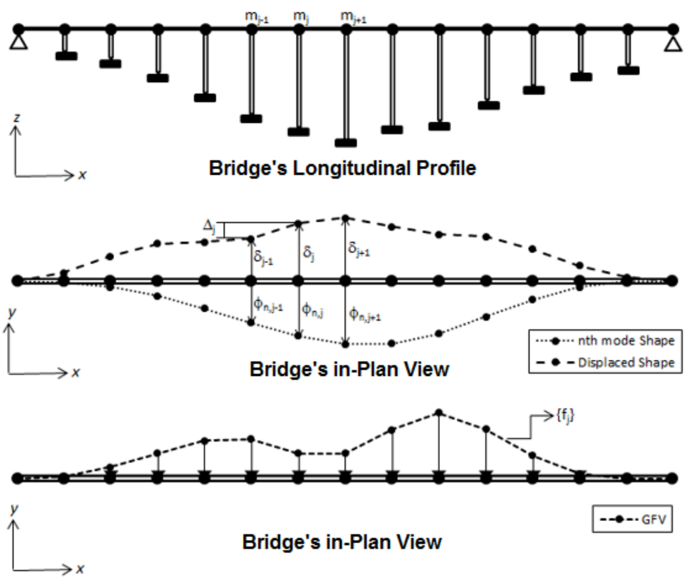

Figure 2: Bridge longitudinal and in-plan view and GFV.

2.1 Total Displacement Based Approach

The response parameter to be maximized in this approach is the top displacement of the $j^{'th}$ pier or the lateral displacement of the $j^{'th}$ deck node. Maximizing the top displacement of the $j^{'th}$ pier or the lateral deck displacement at a certain node maximizes at the same time the drift ratio of that specific pier, as well as the lateral displacement of the bearing connecting the deck with the pier. As such, this approach makes use of *total displacement* (TD) *based generalized force vectors*. The maximum value of the response parameter is obtained at the time step t_{max} – Equation (4) – and the associated modal expansion is given by Equation (5). Normalizing Equation (5) with respect to $\delta_j(t_{\max})$ yields Equation (6).

$$\delta_{j,\max} = \delta_j(t_{\max}) \tag{4}$$

$$\delta_j(t_{\max}) = \sum_n \Gamma_n D_n(t_{\max})\phi_{n,j} \tag{5}$$

$$1 = \sum_n \Gamma_n \frac{D_n(t_{\max})}{\delta_j(t_{\max})}\phi_{n,j} \tag{6}$$

The maximum value of the lateral displacement of the $j^{'th}$ pier or deck node can be estimated (approximately) through RSA using modal combination (SRSS), according to Equation (7), which transforms into Equation (8) when normalized with respect to $(\delta_{j,\max})^2$.

$$(\delta_{j,\max})^2 = \sum_n \left[\Gamma_n D_n \phi_{n,j}\right]^2 \tag{7}$$

$$1 = \sum_n \left[\Gamma_n \frac{D_n}{\delta_{j,\max}}\phi_{n,j}\right]^2 \tag{8}$$

In Equations (7) and (8), D_n is the spectral displacement corresponding to the $n^{'th}$ mode at the corresponding natural period – T_n. The right-hand sides of Equations (6) and (8) are the normalized contributions of the $n^{'th}$ mode to the displacement of the $j^{'th}$ pier or the $j^{'th}$ deck node, computed through elastic response history or RSA. Equating both terms and making use of Equation (4), we obtain Equation (9). Replacing $\delta_{j,n}$ from Equation (7) into Equation (9), $D_n(t_{\max})$ can be rewritten as in Equation (10).

$$D_n(t_{\max}) = \frac{D_n}{\delta_{j,\max}}\left[\Gamma_n D_n \phi_{n,j}\right] \tag{9}$$

$$D_n(t_{\max}) = \frac{\delta_{j,n}}{\delta_{j,\max}} D_n \tag{10}$$

Finally, the GFV can be obtained by replacing Equation (10) into Equation (3), and successively in Equations (2) and (1), leading to Equation (11). Given that the terms in Equation (11) are related to the maximum displacement of the $j^{'th}$ pier or the maximum displacement of the $j^{'th}$ deck node, it is associated with the subscript of the $j^{'th}$ pier or deck node.

$$\{f_j\} = \sum_n \Gamma_n [M]\{\phi_n\}\frac{\delta_{j,n}}{\delta_{j,\max}} A_n \tag{11}$$

The target displacement at the $j^{'th}$ pier or the $j^{'th}$ deck node, following the "Total-Displacement Based Approach", is thus given by Equation (12), putting together Equations (5) and (10).

$$\{\delta_{j,t}\} = \sum_n \Gamma_n \frac{\delta_{j,n}}{\delta_{j,\max}} D_n \phi_{n,j} \tag{12}$$

2.2 Relative-Displacement Based Approach

In the alternative approach, employing relative displacement, analogous to inter-storey drift in buildings, may improve the NSP predictions with respect to NTHA results [11] in bridges. One reason is related to the fact that higher order seismic response parameters, such as plastic rotations and curvatures, are closely related to the inter-storey drift and not to floor displacements. In bridge structures, there is no inter-storey drift but a similar parameter has been considered, corresponding to the relative displacement between succeeding piers, as illustrated in Figure 2 and denoted by Δ_j. This approach will thus be referred to as *relative-displacement* (RD) *based generalized force vectors*. For this alternative, the formulation presented in [11] for GFV is used, although the physical interpretation is different. For convenience and completeness, the formulation is summarized in the following.

The GFV that acts on the structure when the relative displacement of the $j^{'th}$ pier with respect to the $(j\text{-}1)^{'th}$ pier, or the $j^{'th}$ deck node with respect to the $(j\text{-}1)^{'th}$ deck node reaches its maximum value is given by Equation (13).

$$\{f_j\} = \sum_n \Gamma_n [M]\{\phi_n\} \frac{\Delta_{j,n}}{\Delta_{j,\max}} A_n \tag{13}$$

The modal combination of the $n^{'th}$ mode for relative displacement is obtained according to Equation (14) whilst the maximum relative displacement of the $j^{'th}$ pier with respect to the $(j\text{-}1)^{'th}$ pier, or the $j^{'th}$ deck node with respect to $(j\text{-}1)^{'th}$ deck node is given by Equation (15).

$$\Delta_{j,n} = \sum_n \Gamma_n D_n (\phi_{n,j} - \phi_{n,j-1}) \tag{14}$$

$$(\Delta_{j,\max})^2 = \sum_n \left[\Gamma_n D_n (\phi_{n,j} - \phi_{n,j-1})\right]^2 \tag{15}$$

Finally, the target displacements for the "relative-displacement based approach" is simply defined according to Equation (16).

$$\Delta_{j,\max} = \sum_n \Gamma_n \frac{\delta_{j,n}}{\delta_{j,\max}} D_n (\phi_{n,j} - \phi_{n,j-1}) \tag{16}$$

3 GENERALIZED PUSHOVER ANALYSIS ALGORITHM

The GPA procedure applied to bridges can be summarized in the following steps:

1. **Data:** A MDOF model is constructed, with the usual data required for elastic analysis.
2. **Seismic Demand:** The seismic demand can be represented by either a design spectrum or the response spectrum of a specific record.
3. **Eigenvalue Analysis:** Natural periods, modal vectors and modal participation factors are determined for the linear elastic structure.
4. **Response Spectrum Analysis:** Modal spectral amplitudes are obtained from the corresponding design or response spectrum (in this study 5% damping has been assumed) for

each mode considered. Maximum displacement demand for the *total displacement* approach or for the *relative displacement* approach, are computed from RSA.

5. **Generalized Force Vectors:** The generalized force vectors that produce the maximum total displacement at the $j^{'th}$ pier, or the maximum relative displacement between the $j^{'th}$ and $(j\text{-}1)^{'th}$ pier, are computed; and depending on the configuration of the model, the GFVs that produce the maximum total displacement at the $j^{'th}$ deck node, or the maximum relative displacement between the $j^{'th}$ and $(j\text{-}1)^{'th}$ deck node, are computed from Equations (11) and (13), respectively.
6. **Target Displacement Demand:** Target total displacement demand for the $j^{'th}$ pier or the $j^{'th}$ deck node is computed from Equation (12) whereas the target relative displacement of the $j^{'th}$ pier with respect to the $(j\text{-}1)^{'th}$ pier, or the $j^{'th}$ deck node with respect to the $(j\text{-}1)^{'th}$ deck node is computed from Equation (16). If improved accuracy is sought, the spectral contribution corresponding to the first mode can be replaced by its inelastic counterpart. The latter is estimated by conducting a 1st-mode based pushover analysis, computing the capacity curve, which is then transformed into an equivalent SDOF system, and running NTHA over such system.
7. **Generalized Pushover Analysis:** A total of *n* (where *n* is the number of piers, or lumped mass locations), or *n+1* (if the first mode based pushover analysis is performed for determining inelastic "first mode" contributions) analyses are conducted (either total-displacement based or relative-displacement based). In the $j^{'th}$ GPA the structure is pushed laterally with the associated monotonically increasing lateral load vector and at each load increment the total displacement (or relative displacement) is controlled and compared with the target displacement, until it is reached.
8. **Total Seismic Demand:** Once the *n* number GPA analyses are performed, the envelope of maximum response parameters among the *n* analyses is considered as the maximum seismic demand in the structure.
9. **Performance Evaluation of the System:** Once the total seismic demand on the structure is determined, seismic performance evaluation can be carried out.

4 CASE STUDY

In order to investigate the applicability of the GPA procedure, an existing viaduct, located in Cingoli, Italy, was selected as a case study. This bridge features a number of characteristics that favour a comprehensive validation of the method: it is a multi-span bridge with mainly uniform deck geometry; it is a relatively long bridge; and it has a substructure with regular prismatic geometry. Figure 3 shows the longitudinal profile and the in-plan view of the bridge. For the sake of simplicity, the in-plan curvature will be ignored in the modelling. The Cingoli bridge is a multi-span viaduct with fourteen spans, each one approximately 31.5m of length, and a total length of 441m. The thirteen piers of the viaduct vary in height between 7.2m for the shortest, to 31.5m for the tallest. Piers are either composed by a constant circular cross section of 2.6m diameter along the entire height, or by two sections: the lower part (connected to the foundation) of 4.0m diameter for a certain height and the upper part with 2.6m diameter, connected to the cap beam and then to the deck.

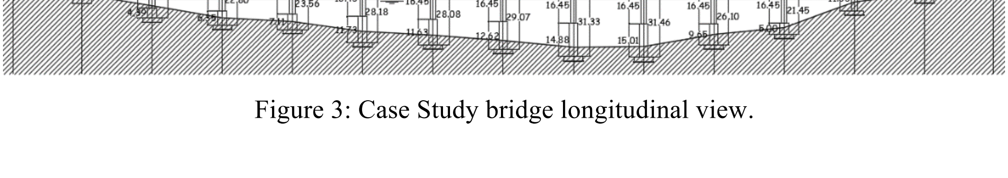

Figure 3: Case Study bridge longitudinal view.

4.1 Modelling

The deck is modelled as a linear elastic element whereas the piers are modelled using force-based fibre elements. For the detailing of the model of the bridge, the mechanical behaviour of the abutments and bearings was required. The mechanical behaviour of the abutments was characterized according to the provisions outlined in Caltrans Seismic Design Criteria [19] and in [20]. Trying to reproduce as best as possible the devices of the existing viaduct, built in 1980, the hysteretic behaviour of the bearings was assumed with a tri-linear relationship, intending to simulate the recommendations of the NCEER report “Response of Steel Bearings to Reverse Cyclic Loading” [21]. Both linear and nonlinear static and dynamic analyses were carried out with the fibre-based software *SeismoStruct* [22]. The Mander confined model for the concrete response and the Menegotto-Pinto model for the reinforcement steel response have been employed. Abutments and bearing devices were modelled with link elements to reproduce their cyclic response whereas the pier support conditions were modelled as fully fixed.

4.2 Eigenvalue Analysis

Eigenvalue analysis was performed for the model of the bridge. It was observed that the first and third modes are the ones contributing the most to the dynamic behaviour of the bridge. Moreover, the modal mass participation from mode 6 onwards is residual, hence their contribution could be ignored for the application of the GPA procedure. However, for this study, thirteen modes of vibration and their respective contributions are considered for the implementation of the method. The first 4 modes of vibration are illustrated in Figure 5.

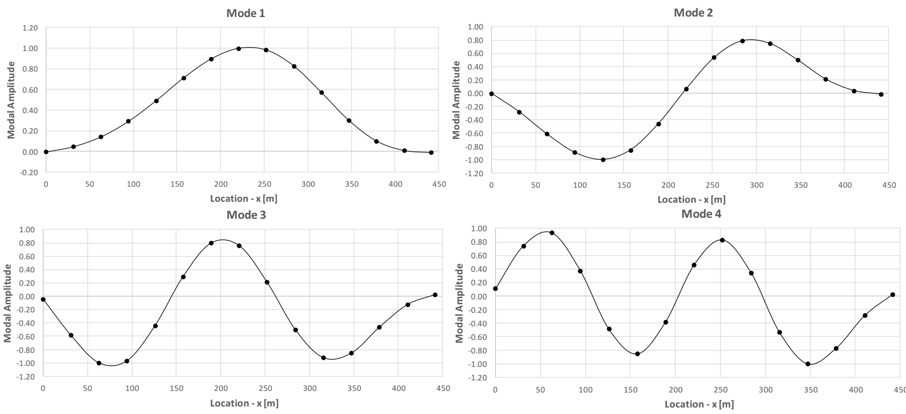

Figure 4: Modal shapes.

4.3 Selection of ground motion records

The selection of records for NTHA and the computation of corresponding response spectra was carried out using a state-of-the-art target spectrum approach; the *Conditional Mean Spectrum* (CMS) method [23]. CMS provides the expected response spectrum for a certain site, conditioned to the value of a target spectral acceleration according to the seismic hazard level or specified return period for the ground motions. The main purpose of using a target spectrum for selecting records was to reduce the record to record dispersion in the NTHA results [24], thus obtaining more reliable estimates for structural demand. CMS assures the achievement of a response spectrum that is compatible with the zone and the seismic hazard level, conditioned to a specific target period that is more relevant to the behaviour of the structure. The computation of the CMS requires parameters from probabilistic seismic hazard analysis (PSHA). In this study, the software *OpenSignal* [25] was used. *OpenSignal* is directly connected to the database of the Italian Institute for Geophysics and Volcanology (INGV), performs the PSHA and produces the CMS for the specified seismic hazard level, based on the geographical coordinates of the site of the structure and the target period. Three hazard levels were considered for this study, represented by the return periods of 475, 2475 and, if no significant yielding occurs in the structure, 9210 years. The target period for the computation of CMS was taken as the fundamental period of vibration. Different spectra were computed, real records were selected from the PEER and ESMD databases, and scaled to match the CMS at the target period. Ten records featuring the lowest value of SSE, illustrated in Figure 5 for two of the return periods, were selected.

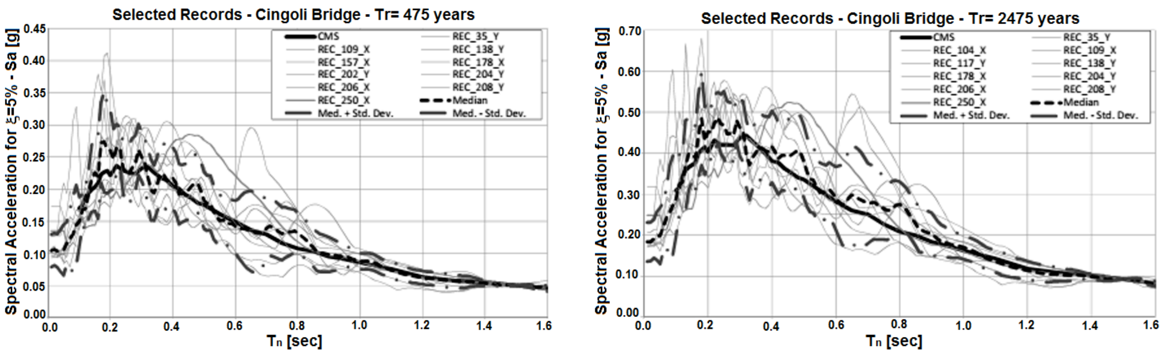

Figure 5: CMSs and acceleration response spectra from selected records.

4.4 First Mode Pushover Analysis

First mode pushover analysis was carried out and the capacity curve using the centre of mass of the deck as the reference node, as suggested in [26], was computed. The capacity curve was then transformed to an equivalent SDOF system. By running NTHA with the selected records over this equivalent SDOF system, the inelastic first mode spectral displacements (D_1*) were computed to define the capacity curve and the equivalent SDOF system, characterized by an approximate bilinear curve.

5 RESULTS

Applying the procedure outlined previously, all pushover analyses (required for GPA and CSM procedures) and all NTHA were performed, for all selected records. The target displacements can be computed in different ways, either directly from the response spectrum analysis (conventional – C), or by utilizing the inelastic estimation for the first mode and the response spectrum demand for higher modes (improved – D_1*). As such, a set of four different target displacements can be computed, by putting together the total or relative displacement (TD or RD) approaches with the conventional or improved target displacements: (i) TD-C; (ii)TD-D_1*; (iii) RD-C; and (iv) RD-D_1*. With respect to the NTHA, in agreement with the computation of the CMS and response spectra from the selected records, viscous damping ratio was assumed as 5% (ξ=0.05). A tangent stiffness proportional damping model has been adopted, which has been found superior to Rayleigh damping and initial stiffness proportional damping for SDOF systems [27]. In this section, the results are presented, for each ground motion record and each return period, in terms of the lateral displacement profiles or rotations at the base of the piers, estimated with: (i) the four GPA versions; (ii) the alternative nonlinear static procedure (CSM); and (iii) nonlinear time history analysis. For brevity, detailed results are presented in terms of deck lateral displacement and rotation at the base of the piers.

5.1 Deck lateral displacements

Figure 6 illustrates the median response profiles, in terms of deck lateral displacement profiles of the bridge, (across all the records) for the three tested return periods. The results reveal that there is generally a good agreement between the predictions from NTHA and GPA, especially when the improved target displacement versions (GPA-TD-D_1* and GPA-RD-D_1*) are employed. Indeed, for shorter return periods (475 years), the conventional target displacement GPA version (GPA-TD-C) is able to provide a good match with NTHA estimates, whereas for the 2475 and 9210 years return periods, the improved target displacement GPA approach (GPA-TD-D_1*) is the one performing better. The underperformance of the CSM is also evident.

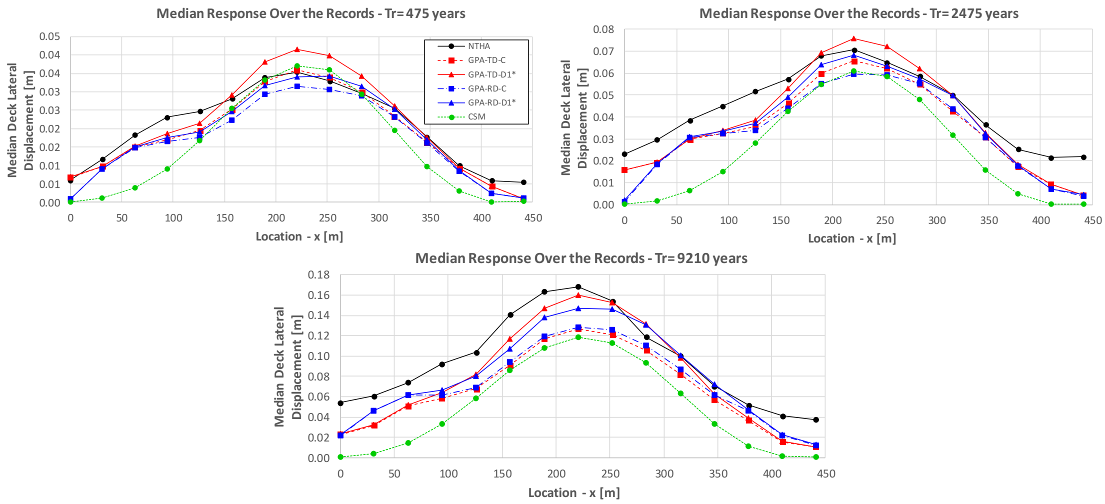

Figure 6: Median Response for the Deck lateral displacement prediction.

5.2 Pier rotation at the base

Figure 7 presents the median response profiles (across all the records), in terms of pier rotation at the base, for the three tested return periods.

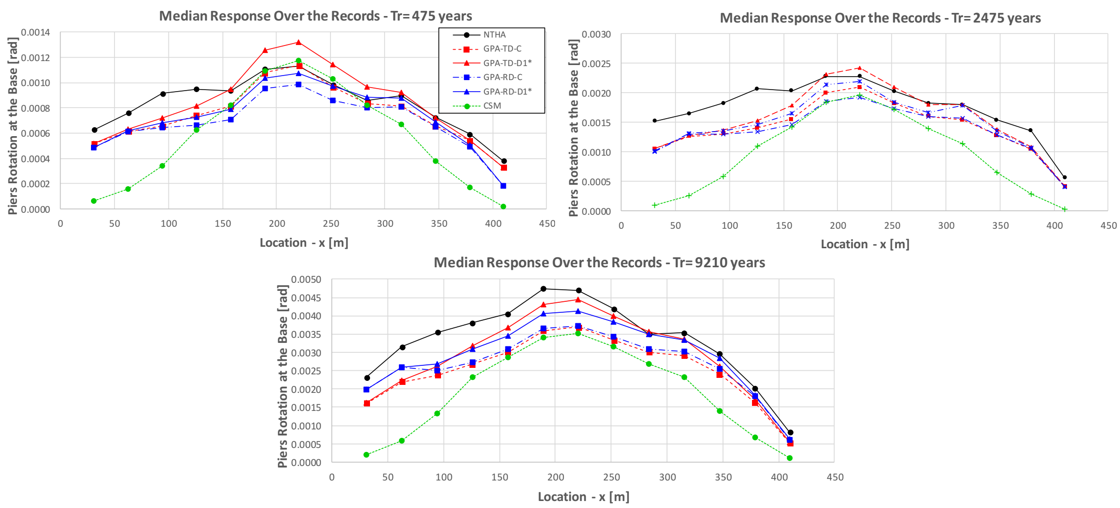

Figure 7: Median Response for the piers' base rotation prediction.

When the EDP under analysis is the rotation at the base of the piers, the best agreement between GPA and NTHA occurs when the former is carried out with the improved target displacement versions (GPA-TD-D_1* and GPA-RD-D_1*). Moreover, for the 475 years return period the GPA-TD-C version is capable of producing acceptable estimates while for the 2475 and 9210 years return periods the GPA-TD-D_1* version works better when compared to NTHA results, hence should be preferred. The relative performance of the different approaches in terms of the pier's rotation at the base is therefore very similar to the one observed for the lateral displacement prediction.

5.3 Overall comparison

In order to identify the best performing option among the different GPA versions and CSM, for all the considered records and hazard levels, the results were statistically treated. This was achieved by calculating the Normalized Root Mean Square Deviation (NRMSD), at a location j, according to Equation (17), where the RMSD at a location j is defined in Equation (18). In the equation, n represents the number of records.

$$NRMSD_j = \frac{RMSD_j}{\max(EDP_{NTHA,j})\Big|_{i=1}^{i=n} - \min(EDP_{NTHA,j})\Big|_{i=1}^{i=n}} \tag{17}$$

$$RMSD_j = \sqrt{\frac{\sum_{i=1}^{n}\left(EDP_{NTHA,i,j} - EDP_{NSP,i,j}\right)^2}{n}} \tag{18}$$

Taking the median and the standard deviation of the NRMSD over different locations and/or different hazard levels will account for the assessment of the relative accuracy of each method with respect to NTHA in a more comprehensive manner.

The NRMSD was computed at each lumped mass location for the records corresponding to each return period for each GPA version. The median and the standard deviation (STD) over the NRMSDs at different locations are computed and reported in Figures 8 and 9 as a characterization of the accuracy of each version of the GPA method (for each version, the median and the median+STD are represented). Finally, the global median NRMSD across all the locations and intensities, is also calculated and presented as a global indicator of the accuracy of each GPA version with respect to NTHA. In general, for low hazard levels corresponding to low return periods, the GPA alternatives based on the conventional target displacement (GPA-TD-C and GPA-RD-C) provide the lowest normalized deviations from NTHA, across all the locations. On the other hand, for the higher return periods the improved target displacement versions (GPA-TD-D_1* and GPA-RD-D_1*) showed a better performance in predicting the response of the structure, providing the lowest normalized deviations. Another immediate observation is the superiority on the predictions of the GPA algorithm when compared with the accuracy of CSM, regardless of the GPA version implemented in the analysis.

In terms of deck lateral displacement, Figure 8 shows that for higher return periods, hence higher nonlinearity levels, the accuracy of the improved target displacement GPA versions is higher (lower values of median NRMSD). On average, across all the hazard levels, the versions GPA-TD-D_1* and GPA-RD-D_1* are indeed the most accurate. Moreover, for what concerns the piers' base rotation, Figure 9 shows a similar trend, although the difference between the traditional and improved target displacement GPA versions is less evident for low returns periods. Globally, the indices presented in the bottom-right corner of both figures clearly denote the superiority of the improved (D_1*) with respect to the conventional (C) approach.

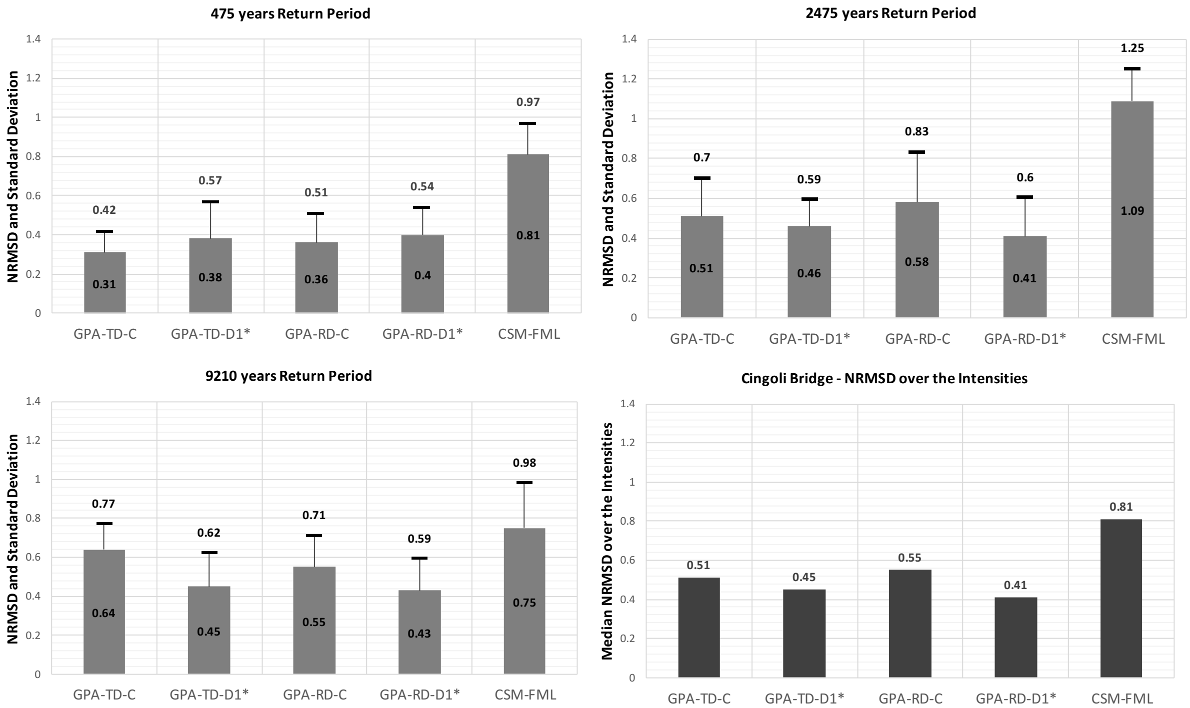

Figure 8: Median NRMSD and standard deviation over the locations for the deck's lateral displacement.

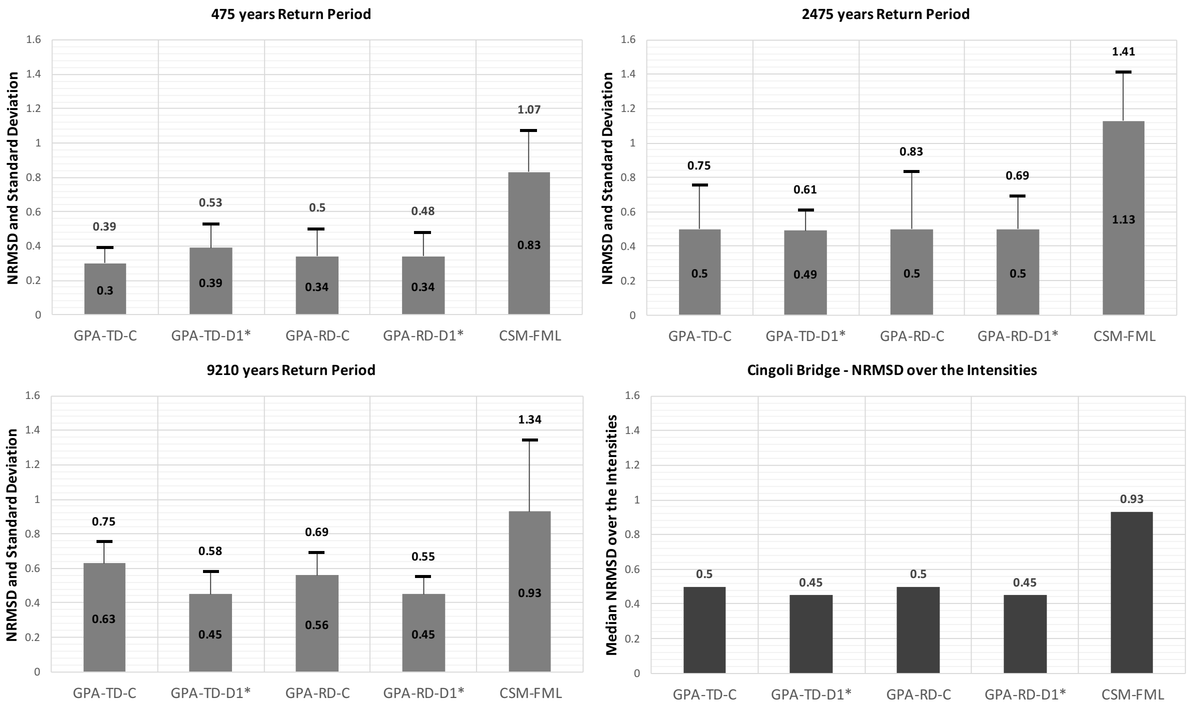

Figure 9: Median NRMSD and standard deviation over the locations for the piers' base rotation.

The dispersion on the results proved not to be highly relevant and almost the same amongst the different versions of the GPA method. Nevertheless, for low return periods the dispersion on the results is slightly lower for the conventional alternatives (GPA-TD-C and GPA-RD-C) which is understandable, given the nearly elastic behaviour of the structure. On the other hand, for higher return periods, it is the improved version of GPA that yields lower dispersion, which confirms the advantage of employing such version in the presence of nonlinear behaviour.

When the relative performance of the GPA alternatives is assessed in terms of Total or Relative Displacement approach, the differences are not as evident as when the performance is assessed taking into account the conventional or improved target displacement. Figures 8 and 9 show that for both EDPs investigated and low hazard levels the performance of the Total Displacement approach and the Relative Displacement approach are very similar. On the other hand, for higher hazard levels, the performance of the Relative Displacement approach with respect to the Total Displacement approach is just slightly superior for the deck lateral displacement prediction and again virtually the same for the pier base rotation. The difference in the dispersion on the results when comparing TD approach and RD approach proved again not to be significant although the relative displacement approach tended to present higher dispersion for some of the return periods.

6 CONCLUSIONS

A nonlinear static procedure for the seismic performance evaluation of bridges, using generalized force vectors, named Generalized Pushover Analysis (GPA), has been implemented in a case study composed of an existing reinforced concrete straight viaduct. The multi-mode pushover procedure, which incorporates the simplicity of a conventional (non-adaptive) pushover algorithm with the advantage of taking into account higher mode contributions on seismic response estimates, was investigated in the transverse direction only. Two different engineering demand parameters (lateral pier top or deck displacement profile and cross section rotation at the base of piers) were considered. The accuracy of the GPA algorithm was assessed by direct comparison with nonlinear time history analysis (NTHA) estimates. Furthermore, the GPA results were also compared with a single mode based widely employed nonlinear static procedure the Capacity Spectrum Method (CSM). Four versions of the GPA algorithm were implemented, differing essentially with respect to the way in which the target displacement is considered: through a total displacement (TD) or relative displacement (RD) approach, with conventional (C) or improved (D_1*) spectral demand, by using corresponding target conditioned spectra. Ten ground motion records were selected and matched to such spectra at the structure's fundamental period, using the conditional spectrum method procedure. Based on the findings discussed in this document, the following conclusions can be drawn.

The results have demonstrated that the GPA algorithm is able to successfully capture the seismic response of the case study viaduct for all hazard levels considered. In general GPA predictions matched in a very good way the seismic demand estimated using NTHA, especially for what concerns the lateral displacement demand at lumped mass locations The results have also highlighted that when seismic events of low intensity level (low return periods and higher probability of exceedance) are considered, the GPA employed with the conventional target displacement formulation (GPA-TD-C) is capable of producing good estimates, at times even slightly superior to the improved target displacement (GPA-TD-D_1*). On the other hand, the latter is definitively the best choice for seismic events with high return periods i.e. low probability of exceedance. The results also showed that the relative displacement (RD) approach proved to be slightly more accurate than the total displacement (TD) counterpart. Nevertheless, the relative superior performance of the relative displacement approach is not significant in improving the accuracy of the results with respect to NTHA. In addition, the total displacement approach proved to be easier in implementation for parametric studies, therefore this option is preferred over its counterpart.

With respect to the alternative nonlinear static procedure that was tested the results univocally showed that GPA predictions are superior to CSM predictions for all response parameters and for all hazard levels investigated. It is therefore believed that the GPA algorithm, in the suggested versions, is a suitable NSP for the seismic performance assessment of bridge structures especially when the higher mode contributions to seismic response are significant.

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