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Seismic performance of irregular bridges - comparison of different nonlinear static procedures

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Seismic performance of irregular bridges – comparison of different nonlinear static procedures

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The adequate seismic performance of transportation infrastructures is important for the functioning of the economy and society. This paper focuses on the seismic assessment and analysis of one of the most important components of these infrastructures, the bridges. In this field, nonlinear static procedures (NSPs) have gained significant attention, resulting in different proposals to improve the accuracy of the procedures while keeping their simplicity. The main goal of this study is focused on the evaluation of the applicability of NSPs for irregular reinforced concrete viaducts. A comparative approach is pursued by resorting to (1) the analyses of the performance of three well-known NSPs (N2 method, modal pushover analysis and adaptive capacity spectrum method) and (2) the extension of the scope of previous studies in this field to a more recent method, the extended N2. As such, a set of bridges with different levels of irregularity, configurations and lengths is investigated. The accuracy of different NSPs is evaluated by comparing the results of NSPs with the ones obtained by means of nonlinear dynamic analyses. The comparison of results confirms the acceptable performance of the multi-modal NSPs and highlights the effectiveness of extended N2 method with respect to its simplicity.

Keywords: irregular bridges; seismic performance; nonlinear static procedures; nonlinear dynamic analyses; extended N2 method

1. Introduction

Bridges are considered as one of the key components of transportation infrastructures. Given their importance for development of economy and society, attentions should be devoted in design and construction of these structures in order to assure their sustainability throughout their lifetime, including during and after extreme events. Because many bridges have been designed according to old codes, which did not address today's performance requirements, it is relevant to adequately assess their potential performance under extreme events, namely earthquake actions. For this purpose, some new methods, based on nonlinear static procedures (NSPs), have been developed. These methods were introduced as powerful tools for seismic design and assessment of structures, as they are usually able to conservatively predict the seismic response of the structures while keeping the simplicity of application (Bhatt & Bento, 2012). The efforts on developing NSPs have led to the introduction of different methodologies and approaches, which are accepted in several guideline documents or design codes (American Society of Civil Engineers [ASCE], 2007; Applied Technology Council, 2005; EC8-1, 2004; Federal Emergency Management Agency [FEMA] 273, 1997; FEMA 356, 2000).

However, these guidelines are more focused on the seismic assessment of buildings rather than bridges. Because bridges have different typology of structures compared with building structures, the findings drawn based on studies of the latter should not be applied or extrapolated for the former without essential attention and careful understanding of the method, which may result in underestimated or even sometimes erroneous results as was shown by Fischinger, Beg, Isakovic, Tomazevic, and Zarnic (2004). In this study, the doubtful validity of application of standard NSPs for the case of bridge structures is addressed.

Recently, considerable attempts have been made to verify the applicability of different NSPs for design and assessment of bridge structures in transversal direction (Aydınoğlu, 2004; Casarotti, Monteiro, & Pinho, 2009; Isakovic & Fischinger, 2006; Isakovic, Pompeyo, Lazaro, & Fischinger, 2008; Kappos, Saidi, Aydınoğlu, & Isakovic, 2012; Monteiro, 2011; Paraskeva, Kappos, & Sextos, 2006; Pinho, Monteiro, Casarotti, & Delgado, 2009; Shakeria, Tarbalib, & Mohebbia, 2013). In general, these studies verify the acceptable performance of the various NSPs for the case of regular and short bridges. In addition, in these studies, the variability of the results depending on the pushover analysis type or selection of analysis parameters such as monitoring point, load distribution and spectral reduction factors, especially for long and irregular bridges, is highlighted.

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Although these studies show reliable findings about the applicability of NSPs for the seismic assessment of viaducts, a need for further research for other bridge configurations, mainly for irregular bridges, is pointed out. In addition, in most of the mentioned studies the definition of irregularity and its different possible types was not extensively studied. The exceptions are the works of Casarotti et al. (2009) and Pinho et al. (2009), which have addressed the evaluation of different NSPs for more than one irregular configuration, both short and long. Nevertheless, in these two works (Casarotti et al., 2009; Pinho et al. 2009) the performance of the extended N2 method has not been addressed.

In this study, the performance of the application of different NSPs to the seismic assessment of irregular reinforced concrete viaducts is analysed. The study comprises the evaluation of the performance of three well-known NSPs (N2 method, modal pushover analysis (MPA) and adaptive capacity spectrum method (ACSM)), and the extension of studies in this subject to a more recently proposed method, the extended N2. As such, a set of irregular double-column viaducts with different levels of irregularity, configurations and lengths, typical in modern motorway construction in Europe, is evaluated. Several parameters are used for definition of irregularity of bridges and proper indices are associated with each configuration. The bridges are defined as short, medium and long viaducts with four different predefined levels of irregularity. The analyses are carried out for different seismic intensities in order to gauge the applicability of the procedures for different pier ductility demands. Nonlinear dynamic analyses (NDAs), for three sets of seven ground motions matched with design spectrum, are carried out in order to be used as the most precise available analysis tool. Finally, a comparison between different NSPs and NDAs is presented. The results support the validity of the application of multi-mode procedures whilst the accuracy and simple application of extended N2 method especially for long, irregular bridges is highlighted.

2. Considered NSPs

Four well-known NSPs, N2 method, extended N2 method, MPA and ACSM have been selected for this study. Each method is representative of a different pushover analysis method (e.g. single mode, non-adaptive multi-mode and adaptive multi-mode), performance point estimation procedure or the alternative definitions of reference node.

2.1. *N2 method*

The original single-mode N2 method (Fajfar, 2000) is based on the assumption that the response of the structure is governed by one mode which does not change for different seismic intensities. In this method, the seismic displacement demand is calculated by the use of the displacement ductility demand and the corresponding inelastic spectrum, associated with the 'equal displacement rule' in medium and long period ranges. In N2 method, the effect of higher modes is in general neglected as, in most cases, the configuration of the first mode is used to choose the lateral load distribution.

There are two difficulties associated with this method regarding its application for the seismic analysis of bridges in transversal direction: (1) selection of the reference node to be monitored in pushover analysis as the characteristic node of the structure and (2) selection of the load pattern. Centre of mass of the superstructure or location of the maximum deck displacement (DD) can be used as the reference-monitoring node. However, the latter has been evaluated to be more effective according to several authors (Casarotti & Pinho, 2007; Kohrangi, Bento, & Lopes, 2012; Monteiro, 2011) and is used in this study. Uniform, modal load patterns as well as the envelope of these two load distributions can be applied for the pushover analyses. The effectiveness of each load pattern is examined in this work.

2.2. *Modal pushover analysis*

MPA (Chopra & Goel, 2002), on the other hand, is a non-adaptive multi-mode pushover method, which takes into account the effect of higher modes, although it neglects the changes in the mode shapes for higher seismic intensities. In this procedure, pushover curves are obtained by pushing the structure by invariant force distributions corresponding to various modes of vibration. Consequently, the performance point for each mode is estimated (e.g. using N2 method) and the total demand is obtained by combining the modal demands using Square Root of Sum of Squares (SRSS) or Complete Quadratic Combination (CQC) rules.

2.3. *Adaptive capacity spectrum method*

ACSM (Casarotti & Pinho, 2007) is based on displacement-based adaptive pushover analysis, the so-called DAP, Antoniou and Pinho (2004) accounting for stiffness degradation, period elongation and progressive structural damage as well as higher mode effects and the changes in modes in the nonlinear ranges. In each step, the single degree of freedom system adaptive capacity curve is derived according to the displacement profile of the multiple degree of freedom system. As such, in this method, the need for defining a reference node is avoided. In the next step, an over-damped elastic response spectrum is employed and scaled using equivalent viscous damping and spectrum scaling equations. The performance point is then introduced by intersecting the global capacity curve with the reduced spectrum. The ACSM uses a damping-based reduction factor proposed by Lin and Chang (2003), where the damping is computed using the formula proposed by Gulkan and Sozen (1974), based on the Takeda model without hardening; see Casarotti et al. (2009), Miranda (2000) and Miranda and Ruiz-García (2002).

2.4. Extended N2 method

Extended N2 method is a simple modification procedure in order to improve the results of N2 method where the torsional or higher modes of vibration have a significant effect on the results. The method is based on extensive studies developed by Fajfar, Marusic, and Perus (2005) and Kreslin and Fajfar (2011) and has been tested for buildings (Kreslin & Fajfar, 2011) and bridges (Isakovic & Fischinger, 2011). A summary of the method as applied in this study for the case of bridges is as follows:

(1) Based on N2 method, the performance point displacement corresponding to the first transversal mode of vibration is calculated.
(2) A linear modal response spectrum analysis (RSA) for the transverse direction is performed combining the displacement results according to SRSS rule.
(3) Correction factors (Cr) are computed as the ratio between the normalised DDs obtained by RSA and N2 method (both based on the same elastic response spectrum) considering only the most relevant mode in the transversal direction, usually the first mode in this direction. The normalised DDs are calculated by normalising the displacement value at a specific location with respect to that of maximum DD. The correction factors less than 1.0 should be considered 1.0 to avoid the reduction of the results obtained by means of pushover analysis.
(4) The quantity under study for a certain location is then multiplied by the correction factor estimated for that location.

3. Case study bridges, seismic action and structural analyses

3.1. Definition of structural systems

Double-column bents, typical structural solution in motorway bridges in Europe, with different degrees of irregularity are selected. However, note that the difference between single- and double-column bents (with no cap beam) is not relevant for the purpose of this study, as both systems behave as cantilevers. In this way, what influences the results is mainly the lateral stiffness per alignment, regardless of the correspondent number of columns. Irregularity in the transversal direction of bridges is induced by different parameters. Relative stiffness of deck to piers, location of stiff piers along the bridge and the seismic intensity level can affect the irregularity of bridge structures. The different stiffness of the columns could be induced by changing several characteristics: height, diameter and reinforcement. Even though in common practice, the three characteristics could change, for the sake of simplicity, it was decided to vary only the height of the columns.

It is common that in irregular bridges, higher modes have significant effect on the response of the structure; therefore, modal mass participation of higher modes can be considered as a proper parameter for evaluation of irregularity. Alternatively, a regularity parameter (RP), introduced by Calvi, Pavese, and Pinto (1993), is applied here as a tool for definition of irregularity. RP value can be defined as follows:

$$\mathrm{RP}=\sqrt{\frac{\sum_{j=1}^{n}\left((\phi_j^{\mathrm{T}}/\sqrt{\phi_j^{\mathrm{T}}[M]\phi_j})\cdot[M]\cdot(\psi_j/\sqrt{\psi_j^{\mathrm{T}}[M]\psi_j})\right)^2}{n}}. \tag{1}$$

The eigenvectors of the superstructure considering only degrees of freedom associated with horizontal displacements of the deck, with and without the piers, are defined, respectively, by ψ_j and ϕ_j and the mass matrix is defined by $[M]$, while n is the number of eigenvalues taken into account in the study. In these calculations, all of the significant bridge modes (such that the cumulative mass participation factor exceeds 90% of the total mass) are considered. The values of RP can theoretically range from 0 to 1. For regular bridges, due to the similarity of the mode shapes of the bridge and deck, RP tends to be closer to 1.0. For irregular bridges, the modal shapes of the bridge and deck are not analogous and subsequently, RP will get values less than 1.0, depending on the level of irregularity.

It is observed that the lower is the relative stiffness of deck to piers, the higher is the irregularity of the structure. As such, relative stiffness index (RSI), Priestley, Seible, and Calvi (1996) is simply defined as the ratio of the lateral stiffness of the deck (K_s) to the total lateral stiffness of the piers ($\sum K_p$). For bridges with fixed abutments, this value can be approximately defined according to the following expression:

$$\mathrm{RSI}=\frac{K_s}{\sum K_p}=\frac{384\,E_s\,I_s}{5\,L_s^3}\cdot\sum\frac{H_p^3}{C_p E_p I_p} \tag{2}$$

in which, E_s, I_s, L_s are Young's modulus, moment of inertia of the deck cross-section around the vertical axis and total length of the bridge deck, respectively. E_p, H_p and I_p are the modulus of elasticity, moment of inertia and pier height, respectively. C_p is a value that is defined according to the fixity of the piers at the two ends and ranges from 3 for piers built-in at one end and 12 for piers built-in at both

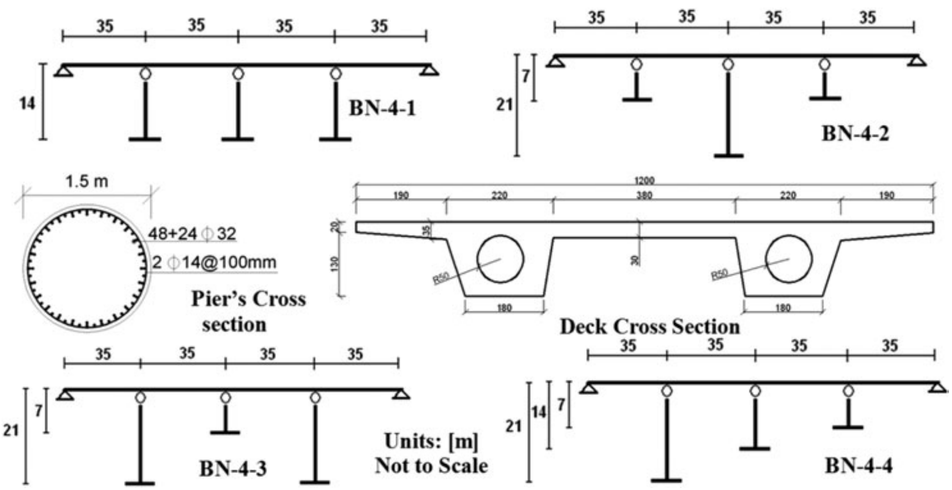

Figure 1. Selected typical four-span bridge configurations, with pier and deck cross-section details.

ends. In such calculation, it is suggested to consider uncracked section for the superstructure and cracked section for the piers. Therefore, a moment of inertia of 0.5 I_p is used to account for the cracking of the piers.

It is worth noting that both procedures presented previously are only able to properly show the irregularity of bridge structure in the elastic range, whereas for high-intensity seismic actions, the bridge regularity changes due to development of plastic hinges. In addition, it should be noted that, the RP and RSI values presented here are valid in the case that transverse displacement at the abutments is restrained. In this work, based on the aforementioned parameters, four types of viaducts with different regularity levels, from regular to very irregular, and three different lengths of 140, 350 and 560 m (4, 10 and 16 spans of 35 m, respectively) with fixed abutments have been selected. The examples chosen do not represent real cases but the dimensions are similar to the ones of real structures. The representation of each group, the geometry of the deck and the typical pier cross section are shown in Figure 1. All the bridges have been designed based on RSA according to EC8 for a design peak ground acceleration (PGA) of 0.38g.

Each bridge is designated by a bridge number, which includes two digits. The first digit shows the number of bridge spans and the second shows the irregularity level, in which 1 denotes the most regular bridge and 4 defines the most irregular one according to RP value; consequently, a total of 12 bridge configurations have been considered. A summary of all considered bridges, modal properties, RP and RSI are listed in Table 1. As can be seen in the table, according to this study, the most irregular bridges

Table 1. Selected bridge configurations and modal properties.

					Modal mass participation (%)		
Bridge number	Designation[a]	RSI	RP	Period (s)[b]	1st mode	2nd mode	3rd mode
BN-16-1	222,222,222,222,222	0.008	1.000	1.41	82	9	3
BN-16-2	111,222,333,222,111	0.002	0.952	1.72	46	19	25
BN-16-3	333,222,111,222,333	0.004	0.884	1.54	70	6	20
BN-16-4	333,332,222,211,111	0.003	0.429	1.98	43	11	27
BN-10-1	222,222,222	0.05	1.000	1.13	82	9	3
BN-10-2	112,232,211	0.01	0.926	1.07	54	31	9
BN-10-3	332,212,233	0.04	0.892	1.02	75	13	7
BN-10-4	333,222,111	0.02	0.603	1.37	55	19	11
BN-4-1	222	2.46	1.000	0.82	85	8	3
BN-4-2	131	0.45	0.994	0.52	82	11	2
BN-4-3	313	0.86	0.993	0.52	87	6	3
BN-4-4	321	0.79	0.992	0.65	82	9	3

[a] Each number shows the pier height from left to right side of the model (numbers 1, 2 and 3 stand for 7, 14 and 21 m high piers, respectively).
[b] Period of the first transversal mode.

are the ones with stiff short columns in one or two sides of the viaduct. For longer viaducts with small RSI value, more irregular behaviour is expected and relatively smaller RP values are derived.

However, for short viaducts, specifically four spans, RP value is relatively high even for very irregular column distributions that can be attributed to the high stiffness of the deck compared with piers. In only two cases of BN-10-2 and BN-16-2, there is inconsistency between the two methods, i.e. the RP value is high but the modal mass participations of the first modes are small. The high RSI value for BN-4-1 is due to the fact that this is the case study that combines short-length viaduct with no short 7 m columns. As the purpose of this study is to perform a comparison of methods of seismic analysis, the mechanical properties of the materials were considered identical to be able to perform a fair comparison.

3.2. *Modelling assumptions*

Finite element analyses were carried out using SeismoStruct (2011), which accounts for both material inelasticity and geometric nonlinearity. The piers are modelled through a 3D inelastic beam-column element. The members' inelasticity was modelled through the use of force-based beam-column fibre elements along the full height of the column. Each fibre was characterised by the respective material relationship. The constitutive laws of the reinforcing steel and of the concrete are considered with strength of 500 and 33 MPa, respectively. Applied models are described in related papers (Mander, Priestley, & Park, 1988; Menegotto & Pinto, 1973).

The deck is a 3D elastic beam-column element, fully characterised by the sectional property values, based on Young's and shear modulus of 25 and 10 GPa, moments of inertia of 2.15 and 67.2 m^4 and a torsional constant of 1.46 m^4. The most important source of energy dissipation is hysteretic damping (ASCE, 2007; Casarotti & Pinho, 2007; FEMA, 2000), which is implicitly included in the nonlinear fibre model formulation of the inelastic frame elements. A 1% Rayleigh damping was assigned to the deck, proportional to the first two transversal modes of the structure.

Because the total failure or collapse of the structure is not explicitly defined in the analysis programme, it is necessary to define the collapse point in order to stop the analysis at collapse occurrence. However, although estimation of the bridge collapse point is very important, due to the complexity of its determination, it has received little attention in previous works.

The collapse of a structure might occur when the structural gravity-load capacity reduces below the existing loads as a result of shear failure or disintegration of a column plastic hinge (Priestley et al., 1996). In this study, collapse (global or local) was considered to take place when one of the following four limit conditions, that may jeopardise the load-bearing capacity for vertical loads, is exceeded: (1) stability failure (i.e. when P-Δ moments exceed the residual capacity of the bridge columns); (2) the ultimate core concrete strain (0.012) calculated according to EC8-2 (2004); (3) the ultimate steel strain specified in EC2 for ductile steel type C (0.075) and (4) the maximum shear capacity of the columns. The shear capacity of piers according to EC8-3 (2005), UCSD shear model (Kowalsky & Priestley, 2000) and ACI-318-05 (2005) is estimated and is listed in Table 2.

Table 2. Column shear (CS) capacity (kN) in plastic hinge region.

EC8	UCSD	ACI
3968	2900	3599

It should be noted that, the UCSD shear capacity corresponds to a curvature ductility of more than 12 and the EC8 shear capacity is calculated based on chord rotation ductility in the collapse prevention limit state, as defined in the code. It was verified in this study that shear failure would not take place because, in all cases, the maximum shear demand based on the capacity design and maximum flexural capacity is below the lowest shear capacity value (i.e. the one from UCSD).

3.3. *Seismic action and ground motion selection*

Three sets of ground motion records were used. Each set contains seven records, which were selected from European Strong-motion Database (Ambraseys, Smith, Sigbjornsson, Suhadolc, & Margaris, 2002) using program REXEL3.3 (Iervolino, Galasso, & Cosenza, 2010). All records were matched with design spectrum for a PGA of 0.38g, Type 1, Soil C with $T_C = 0.6$s. The T_C value was chosen according to EC8/Portuguese National Annex for far-field events, reach in low frequencies, as they are usually more demanding for the seismic design/assessment of bridges, than near field events richer in high frequencies.

SeismoMatch (2010) was used to match the records with EC8 elastic spectrum for the period range of 0.1–2.0 s, which includes the fundamental periods of all the analysed bridges. SeismoMatch is an application capable of adjusting earthquake accelerograms to match a specific target response spectrum. The method used for spectral matching adjusts the time history in the time domain by adding wavelets to the acceleration time-series (Hancock et al., 2006). Even though these records are not fully natural, they maintain relevant characteristics of real accelerograms such as the sequence of peaks. In this manner, it is possible to overcome the shortage of real accelerograms of high magnitude events. Figure 2 shows the design, matched and mean spectrum, for all three sets of records.

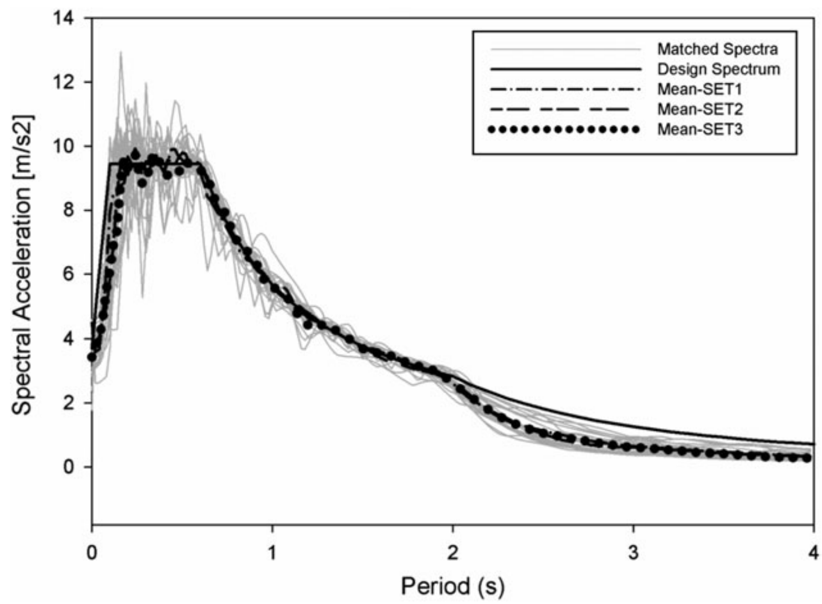

Figure 2. EC8 design spectrum, response spectrum of 21 matched records and the mean spectrum of each set.

In order to assess the sensitivity of the NSPs to different seismic intensities, all ground motions were scaled for different intensity levels. Therefore, the accelerograms were scaled to five intensities of 0.5, 1.0, 1.5, 2.0 and 3.0 of the design spectrum named as I1, I2, I3, I4 and I5, respectively. In order to reduce the time of analysis, an interval between the build-up of 5% and 95% of the total Arias intensity (Bommer & Martínez-Pereira, 1999) was considered. The corresponding target design spectrum of each intensity level was used to compute the performance point in the NSPs. It should be noted that, although ACSM foresees the employment of each of the real response spectra corresponding to each record, rather than a design spectrum, in order to have a consistent demand for all methodologies, the same design spectrum was used for ACSM as well.

4. Parametric study: post-processing results

The results of the mean of NDAs, for all three record sets, have been compared with the results obtained from NSPs. Comparisons are focused on two parameters: DD and column shear (CS). For each intensity level scenario, a bridge index (BI) value, similar to the one introduced by Pinho, Casarotti, and Antoniou (2007), for the two comparison values (DD and CS), are derived according to the following expression:

$$\mathrm{BI} = \mathrm{mean}_{i=1:m}(\bar{\Delta}_i) \tag{3}$$

in which, i is the node location number in the structure, m is the maximum number of considered locations and $\bar{\Delta}_i$ is shown in Figure 3 and defined as follows:

$$\bar{\Delta}_i = \frac{\Delta_{i,\mathrm{NSP}}}{\hat{\Delta}_{i,\mathrm{NDA}}}, \tag{4}$$

where $\Delta_{i,\mathrm{NSP}}$ is the comparison value (DD and CS) derived from NSPs corresponding to node location i.

In addition, $\hat{\Delta}_{i,\mathrm{NDA}}$ is the mean of comparison values (DD and CS) derived from NDAs according to the

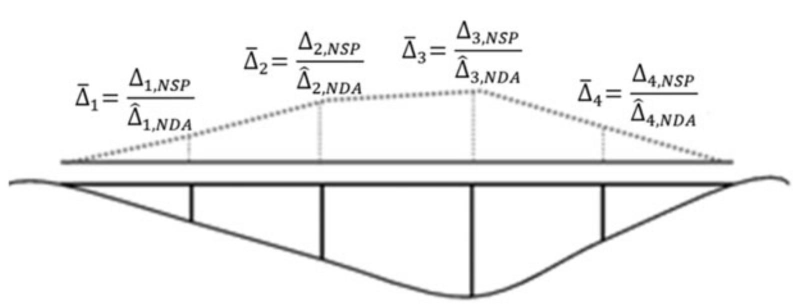

Figure 3. Normalised transverse deformed pattern, adapted from Monteiro (2011).

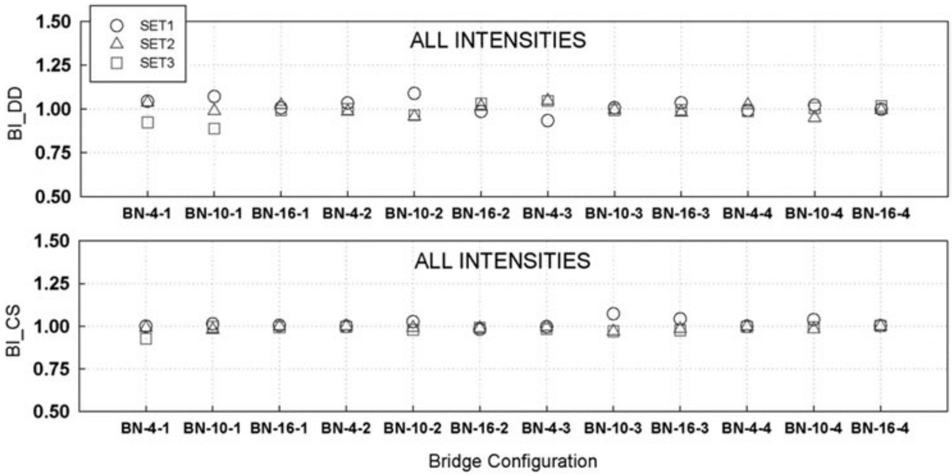

Figure 4. BI values for comparison of NDA: (a) DD and (b) CS.

following expression:

$$\hat{\Delta}_{i,\mathrm{NDA}} = \mathrm{mean}_{j=1:\mathrm{nr}}(\Delta_{i,j,\mathrm{NDA}}) \tag{5}$$

in which j is the record number (from 1 to the number of records nr), $\Delta_{i,j,\mathrm{NDA}}$ is the comparison value in node location i derived from jth record. Note that BI is a mean value for all nodes of the deck and it is a useful tool to synthesise the global behaviour of the viaducts. In this comparison, the dispersion of the NDA results was not considered and only the mean of NDA was compared with the NSP results.

5. Nonlinear dynamic versus nonlinear static analyses

NDAs for different intensity levels and Nonlinear Static Analyses for different loading types are carried out for all of the 12 bridge configurations. For short bridges (four spans), the analyses for four intensity levels of I1, I2, I3 and I4 are performed. For longer bridges (10 and 16 spans), in order to obtain higher ductility levels, Intensity I5 is also examined. Note that such extreme ground motion was used only for the determination of trends in the application of the NSPs. In Figure 4, the BI derived for all of the test cases for individual sets are presented. In this figure, BI represents the mean of the results for each record set over the mean of all three sets, as follows:

$$\mathrm{BI} = \mathrm{mean}_{i=1:m}\left(\frac{\Delta_{i,\mathrm{NDA,mean\ one\ set}}}{\hat{\Delta}_{i,\mathrm{NDA,mean\ three\ sets}}}\right). \tag{6}$$

The results of this BI are similar for all intensities. As can be seen, the mean results of each individual set compared with the mean of all three sets are close to one for all of the cases and for all intensity levels. The maximum and minimum observed dispersion of the BI factor from the mean of dynamic analyses for all configurations are $+11\%$ and -9%. This justifies the fact that the use of one set of seven matched records can be sufficient for the seismic assessment or design of similar structures in order to reduce the analysis time, while getting relatively acceptable results using less number of records.

The displacement ductility for all bridge configurations and intensity levels is shown in Table 3, Table 4 and Table 5 for 4-, 10- and 16-span bridges, respectively. The values are presented in local and global coordinates. The local ductility is the maximum for all columns of the mean of the displacement ductility in each column derived from NDAs. The global displacement ductility is the mean of maximum displacement of the monitoring node from NDAs divided by the yield displacement of the bridge derived from the bilinear modal pushover curve. It should be noted that this point corresponds to a displacement higher than the displacements at the first column hinging

Table 3. Global and local (columns) displacement ductility for four-span bridges.

	BN-4-1				BN-4-2			
Intensity	I1	I2	I3	I4	I1	I2	I3	I4
Global	[a]	[a]	[a]	[a]	0.8	1.6	2.3	2.9
Local	0.7	1.2	1.7	2	0.4	1.8	2.5	3.8
	BN-4-3				**BN-4-4**			
Intensity	I1	I2	I3	I4	I1	I2	I3	I4
Global	1.0	2.0	3.1	4.2	0.8	1.7	2.4	3.6
Local	1.4	2.3	3.4	5.9	1.1	1.9	3.3	4.3

[a] The response is governed by the superstructure and the capacity curve is linear.

Table 4. Global and local (columns) displacement ductility for 10-span bridges.

	BN-10-1					BN-10-2				
Intensity	I1	I2	I3	I4	I5	I1	I2	I3	I4	I5
Global	1.0	2.0	2.4	2.9	3.9	0.9	1.7	2.0	2.5	3.6
Local	0.9	1.8	2.2	2.7	3.6	0.8	2.3	3.0	3.8	6.1
	BN-10-3					BN-10-4				
Intensity	I1	I2	I3	I4	I5	I1	I2	I3	I4	I5
Global	0.9	1.5	1.8	2.4	3.2	0.9	1.9	2.6	3.2	4.0
Local	2.2	3.8	7.3	10.4	14.3	1.4	2.0	3.0	3.7	6.0

Table 5. Global and local (columns) displacement ductility for 16-span bridges.

	BN-16-1					BN-16-2				
Intensity	I1	I2	I3	I4	I5	I1	I2	I3	I4	I5
Global	0.9	1.6	2.1	2.5	3.2	0.9	1.6	2.0	2.2	2.7
Local	0.9	1.6	2.1	2.6	3.2	1.2	2.0	2.7	3.1	3.7
	BN-16-3					BN-16-4				
Intensity	I1	I2	I3	I4	I5	I1	I2	I3	I4	I5
Global	0.7	1.1	1.7	2.1	2.5	1.3	2.1	2.5	2.9	3.5
Local	1.5	1.8	2.5	3.4	8.2	1.0	1.9	2.7	3.7	8.3

because it is derived from the bi-linearisation of the capacity curve; thus, the local ductility may not be higher than the global ductility.

The results show that, for almost all cases, the bridges remain elastic for intensity I1. For the most regular bridge configurations, the local and global displacement ductility values are close, whereas by increasing the irregularity in the bridge configurations, the local ductility becomes higher than the corresponding global value. Therefore, some columns may enter the nonlinear range (i.e. local ductility above 1) even though the global ductility displacement is not higher than 1. This fact is due to the irregular distribution of the ductility along the bridge, which leads to concentration of ductility requirements in shorter columns.

It should be noted that for bridge BN-4-1, the derived pushover curve is almost linear with a slight change in the slope after yielding. This can be attributed to the high stiffness of the superstructure compared with the stiffness of piers (RSI = 2.46, see Table 1). In addition, because of the restrained condition in the abutments, the stiffness of the superstructure, rather than piers, governs the capacity curve. Consequently a high portion of the base shear, especially after yielding of the columns, is transferred to the abutments and an almost linear capacity curve is obtained.

6. Evaluation of different NSPs

In this section the results obtained from the NSPs are compared with NDAs. In particular, the results herein presented include the CS and the displacement of the superstructure expressed by the BI and displacement profiles.

6.1. NSPs for short viaducts

The results obtained for short bridge configurations are presented in this section. In the following, the main issues observed in the analyses of the four-span bridges are discussed.

6.1.1. Load patterns for N2 method

A modal proportional load pattern, uniform load pattern, the envelope of the modal and uniform load patterns, and DAP procedure are applied for pushover analysis of the viaducts in the transversal direction. The pushover curves obtained for BN-4-2 and BN-4-4 are plotted in Figure 5(a),(b). The corresponding displacement profiles derived according to N2 method for intensity I4 and for different load patterns are also shown in Figure 5(c),(d).

The pushover curves correspond to the displacement of the reference node, which is the maximum displacement of

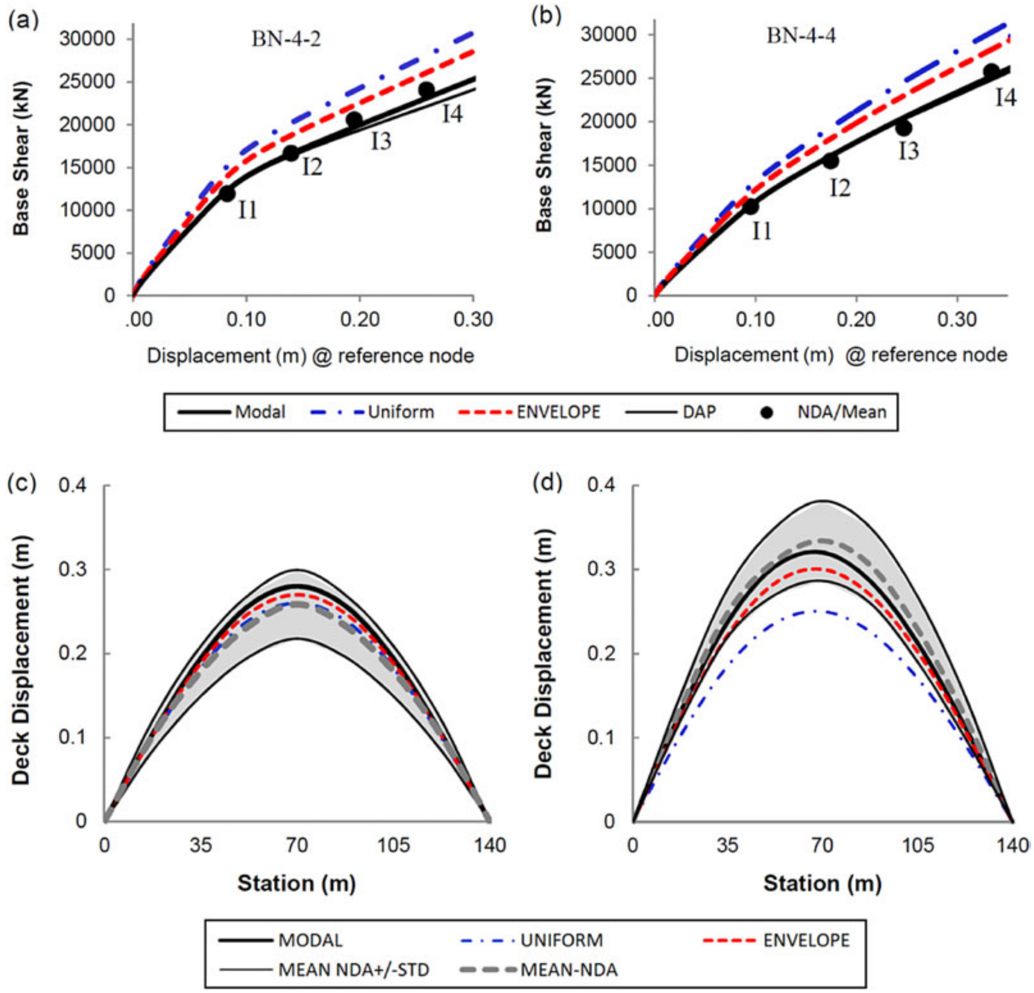

Figure 5. Comparison of pushover curves and displacement profiles according to different load patterns. Pushover curves: (a) BN-4-2, (b) BN-4-4; displacement profiles, N2 method, I4: (c) BN-4-2, (d) BN-4-4.

the superstructure and that for the case of short bridges, in general, is located near or at the centre of the deck (the exceptions are the cases with very short columns near the centre of the deck). As can be seen in the pushover curves, the DAP and modal load patterns match very well with the results of NDAs, especially in the linear range, and the difference in the nonlinear range is very small. Uniform and modal load patterns yield the lower and higher bounds for the base shear capacity, and the load pattern associated with the envelope of the uniform and modal load patterns yields results that are between the previous two. The mean of the NDAs are shown as points for four intensity levels, in the pushover curves. As can be seen, NDA results are closer to the modal and adaptive pushover curves.

The performance points according to N2 method are calculated for different load patterns. Figure 6 shows the calculated errors as difference of the results from the mean of NDA, in percentage. The dispersion representing the difference of the standard deviation (SD) of all NDA results from the mean of NDA values, in percentage, is also presented in this figure. As it can be noted, for all cases, the modal load pattern gives the highest prediction of the performance point displacement followed by the ones due to the envelope and uniform load patterns. In addition, due to the relatively high stiffness of the deck as compared with the stiffness of the piers, in these shorts viaducts, the displacement profiles for all different load patterns are similar. As such, the modal load pattern leads to results that are more conservative than the ones due to the other load patterns. Thus, the modal load pattern is further used in this study.

To apply the N2 method, one needs to define an idealised force-displacement relationship. Due to the relatively large slope of the pushover curves after the plastification of the columns, the use of the bilinear idealisation, as suggested by Isaković and Fischinger (2011), yields better results than the suggested elasto-perfectly plastic idealisation proposed by the original N2 method. Thus, the bilinear idealisation is adopted herein.

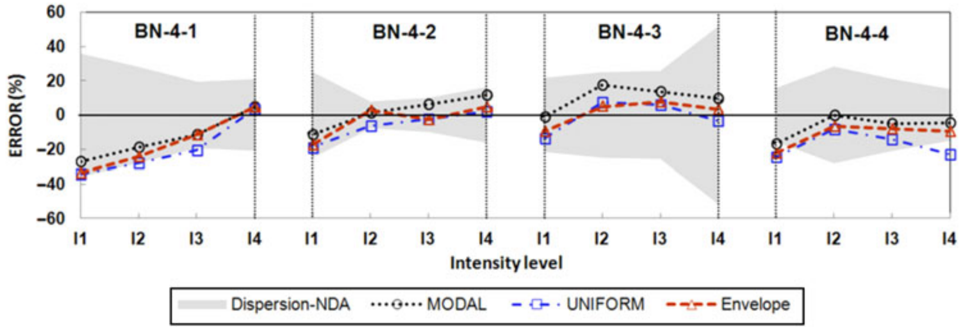

Figure 6. Accuracy of different loading patterns for prediction of target displacement of four-span bridges according to N2 method.

6.1.2. Comparison between NSPs

The values of the bridge index for deck displacement (BI_DD) are shown in Figure 7. In this figure, each point corresponds to the BI of the corresponding bridge according to the applied seismic intensity level. The values of the mean NDA for the DDs ± SD are also presented in this figure. As BI is a measure of the global behaviour but does not provide information regarding variations along the span, this is provided on the displacements profiles (in Figure 5 and in others that follow). In Figure 8, the displacement profiles of all configurations for intensity I4 are shown. The following conclusions can be drawn from the comparison of different NSPs.

Because the effective modal mass of the first transversal mode in all of the short configurations is higher than 80% (see Table 1), as expected, the higher modes do not have significant effect on the response of these structures. As such, single-mode procedure (N2 method) leads to close estimation of the displacement profile for the superstructure, particularly for the higher intensities. This is reflected on the fact that displacement profiles obtained with different NSPs are similar and close to the mean NDA profiles for all BN4 viaducts, even the more irregular ones.

MPA in most of the cases provides the highest estimation of the DD. The results provided by ACSM are very close to N2 method for BN-4-1 and BN-4-3; however, ACSM performs better for the more irregular cases of BN-4-2 and BN-4-4. The correction factors (Cr, see Section 2.4) calculated according to extended N2 method for almost all bridge configurations, due to the insignificance of higher modes are 1 or close to 1 (maximum of 1.06 for BN-4-3), as such, the results of extended N2 method do not change considerably compared with N2 method.

In addition, it should be noted that except for the case of BN-4-1 all the results for intensity I2, I3 and I4 are in the range of mean ± 1SD, which shows the good performance of NSPs for short bridges. Nevertheless, generally in all the methods, the results for the lowest intensity level (I1) are underestimated and the accuracy of the results increases by increasing the seismic intensity. In the case of the N2 method, because it is based only on the first mode, one of the reasons that may improve the

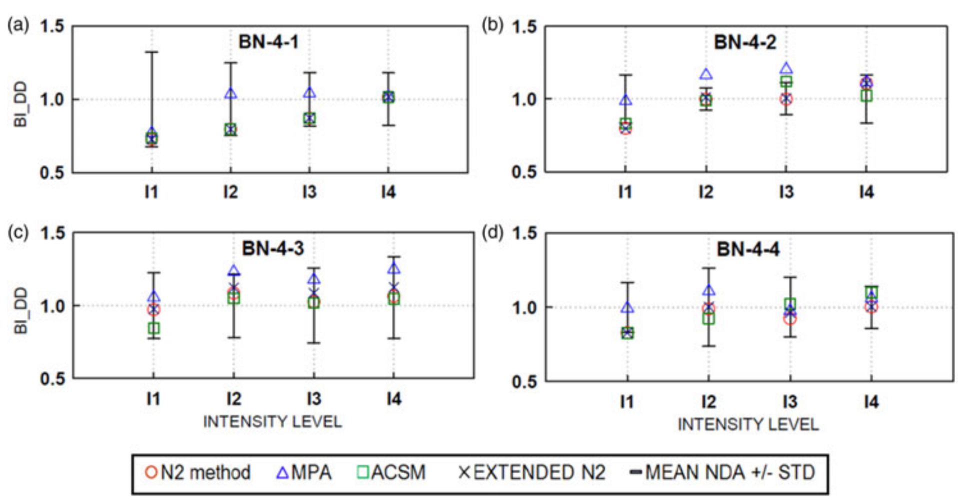

Figure 7. Comparison of BI_DD of short viaducts for different intensity levels according to N2, MPA, ACSM and extended N2: (a) BN-4-1, (b) BN-4-2, (c) BN-4-3 and (d) BN-4-4.

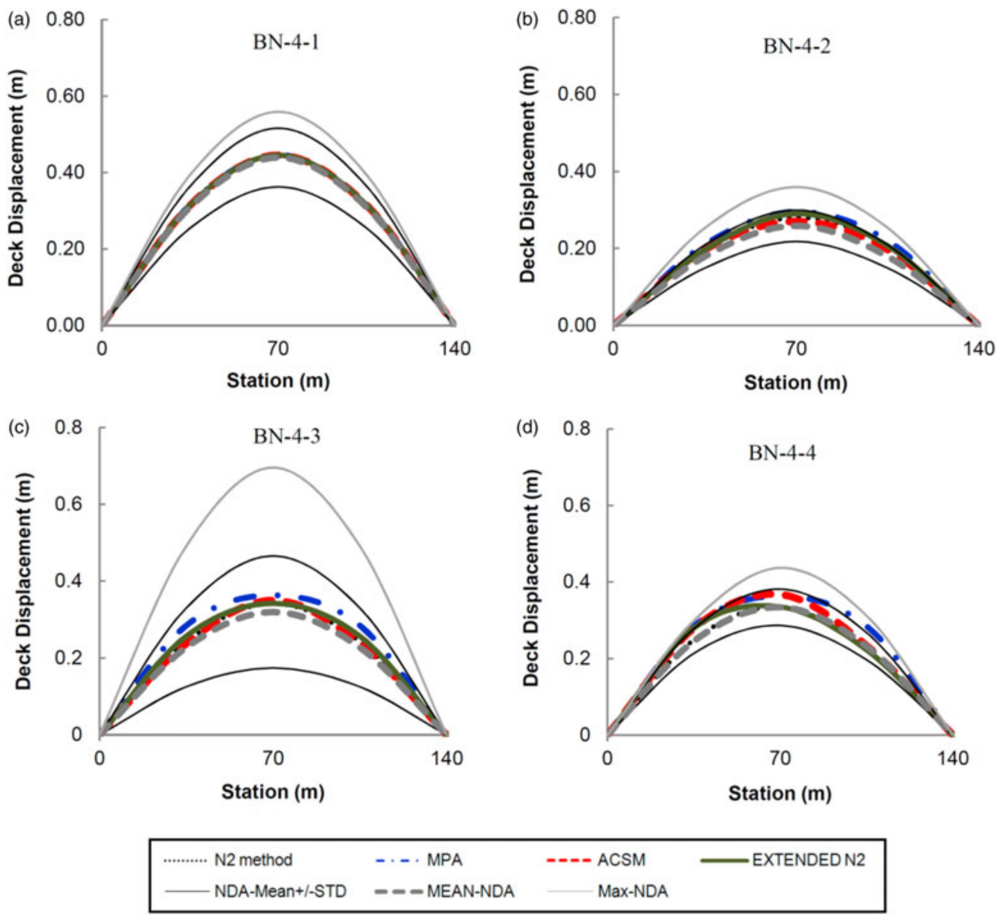

Figure 8. DD profiles for four-span bridges obtained according to different NSPs compared with mean of NDA for intensity I4: (a) BN-4-1, (b) BN-4-2, (c) BN-4-3 and (d) BN-4-4.

accuracy for high-intensity levels may be the increase in the modal mass participation of the first mode for large incursions in the nonlinear range. This is due to the fact that the main source of the importance of the higher modes is associated with the irregularities due to the different stiffness of the columns. As this decreases as the structure goes into the nonlinear range, so does the influence of the higher modes (Isakovic et al., 2008), increasing the relative contribution of the first mode to the dynamic behaviour.

This can be illustrated by the values of the relative mass participation factor of the first mode of one of the most irregular structures BN-4-4: at the initial stages, with elastic stiffness, it is 82% and if the secant stiffness at intensity 4, well beyond the yield displacement is considered, a value of 88% is obtained (this fact is more illustrative for BN-16-4, in which the modal participation factor increases from 43% to 53%). Other authors have also pointed out that the influence of higher modes is less important in the inelastic range (Isakovic & Fischinger, 2006; Isakovic, Fischinger, & Kante, 2003).

However, the main reason for the underestimation of the results in the linear range for all NSPs is the fact that the damping coefficient of the elastic spectra used in NSPs was 5%, a value above the damping coefficient considered in the NDA analysis. The 5% viscous damping intends to account for the energy dissipation in the pre-yield range, whereas the lower value considered in the NDA is recommended in recent studies. According to these studies, no or limited viscous damping should be considered in NDA when the hysteretic damping, clearly the major source of energy dissipation when large inelastic incursion take place, is explicitly considered in the model (e.g. Finley & Charney, 2008; Moaveni, Barbosa, Panagiotou, Conte, & Restrepo, 2009).

When comparing the results of the NDA and NSP for high-intensity levels, because a higher portion of the damping comes from the hysteretic damping and the viscous damping does not have a significant role, this inconsistency between the methods is small. For lower intensity level, because of the lack of hysteretic damping, the absence of viscous damping in NDA can significantly

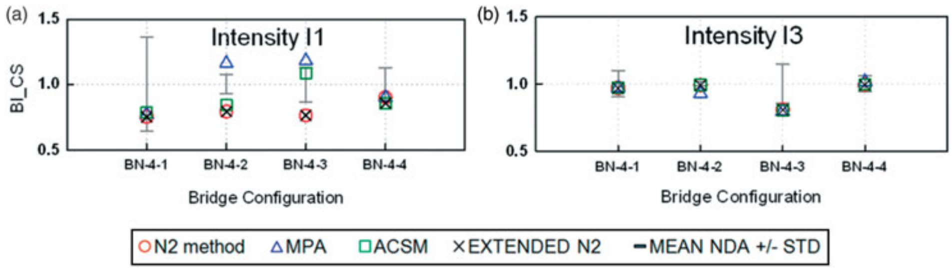

Figure 9. BI_CS of short viaducts for different bridge configurations according to N2, MPA, ACSM and extended N2: (a) Intensity I1 and (b) Intensity I3.

increase the response in relation to NSPs methods. Nevertheless, it should be noted that, the NSPs are mainly the methods that are introduced for the nonlinear response of the structures, and the engineers should be more careful if using these methods for the structural linear state.

BI values for column shear of four-span bridges are shown in Figure 9 for two intensity levels, I1 and I3. These intensities have been selected to represent the effect of both linear and nonlinear range on estimation of shear demand in the piers by NSPs. It should be noted that, because of the attainment of maximum flexural capacity, especially for higher intensities, the CSs could not exceed the shear corresponding to the moment capacity of the column; however, MPA and extended N2 method may increase the shear demands regardless of this fact. As such, the method introduced by Goel and Chopra (2005) is applied to avoid the shear demand overestimation. Moreover, for higher intensities, due to the mentioned demand saturation, the dispersion of the results from mean of NDA is very small.

Most of the procedures underestimate the CS for low intensity, whereas by increasing the intensity level, the bridge index for column shear (BI_CS) value becomes more accurate. However, in almost all of the examined cases, MPA provides the closest results to the mean of dynamic analysis for low-intensity level.

6.2. NSPs for medium and long viaducts

In this section, the results obtained for the analysis of medium and long viaducts are presented and discussed. Selection of monitoring node to develop the pushover curve of single-mode and non-adaptive multi-mode methods (e.g. N2 and MPA) in long viaducts is observed to be more complicated as compared with short viaducts, as the selection of different nodes can lead to different results. In the more regular configurations, the centre of mass and maximum displacement points are usually in the same location, and it is observed that, in these cases, the node with maximum displacement can lead to appropriate results. However, for irregular cases, the selection of the point with maximum displacement is not necessarily the best choice. In addition, for these cases the point of maximum displacement can change with respect to the intensity level. In this study, however, the predicted maximum displacement point by linear modal analysis was selected as the monitoring point, and this has been assumed as one of the shortcomings of the extended N2 method.

6.2.1. *Load patterns for N2 method*

Figure 10(a),(b) present the pushover curves of viaducts BN-16-2 and BN-16-4 (total base shear versus the displacement at the reference point), respectively. The pushover curves are obtained for different load patterns: modal, uniform, envelope and DAP procedure. The mean displacements resulting from NDAs are also plotted in the figure as dots for five different intensity levels. Modal load pattern and DAP assign the lower and upper bound of the base shear capacity, respectively. As can be seen, uniform load pattern provides the closest capacity curve to the results of NDA. The low capacity estimated by modal load pattern is due to the low displacement of the stiff sides of the bridges in the first mode of vibration (see Figure 11), which leads to small induced forces in the stiff parts of the viaduct and consequently poor estimation of the total base shear.

Superstructure displacement profiles for BN16-2 and BN-16-4 for different load distributions according to N2 method are shown in Figure 10(c),(d). The displacement profiles correspond to intensity I4. As can be seen, the performance point displacement obtained from modal load pattern is overestimated, however, as it is expected, the displacements at the stiffer parts of the bridge are underestimated. Uniform and envelope load patterns, on the other hand, lead to a lower estimation of performance point displacements although slightly improve the displacement profile in the stiff parts of the viaduct.

The accuracy of the estimation of the displacement corresponding to the performance point based on different load patterns compared with the mean of NDAs for 10- and 16-span viaducts for different intensity levels are shown in Figures 12 and 13, respectively. As can be seen,

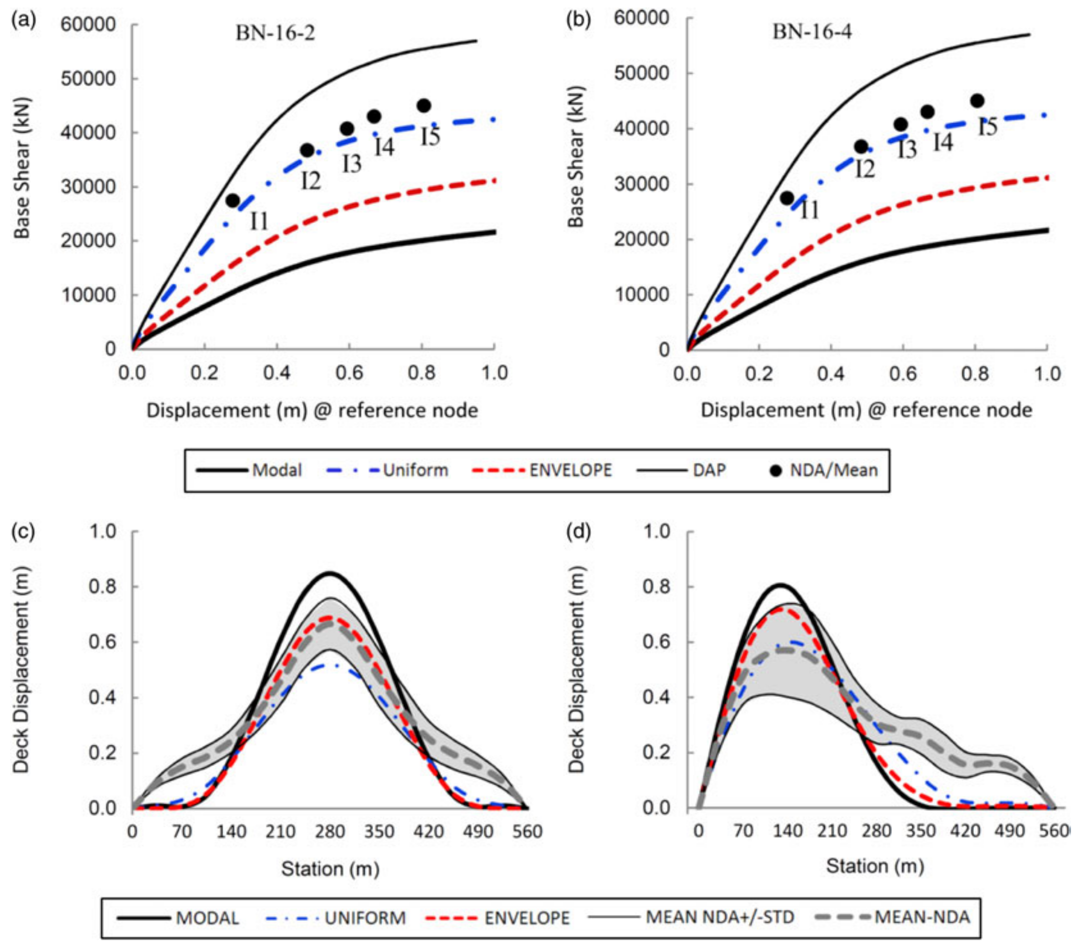

Figure 10. Comparison of pushover curves and displacement profiles according to different load patterns. Pushover curves: (a) BN-16-2, (b) BN-16-4; displacement profiles based on N2 method (Intensity I4): (c) BN-16-2, (d) BN-16-4.

the displacements corresponding to the modal load patterns for all the cases lead to the highest values followed by envelope and uniform load patterns. It should be noted that for the bridge configurations BN-10-3 and BN-10-4, the uniform and envelope load patterns yield similar results.

According to the results herein presented, it is obvious that, in application of N2 method, there are many difficulties in the selection of the load pattern for long bridges as the modal load pattern leads to underestimated displacements in the stiff side of the bridge and uniform and envelope patterns underestimated the monitoring node displacement. In this study, for the case of N2 and extended N2 methods, modal load pattern is applied, as it is the one that leads to the more conservative results.

6.2.2. *Comparison between NSPs*

Figures 14 and 15 depict the BI_DD of 10- and 16-span viaducts for five intensity levels, respectively. In Figures 16

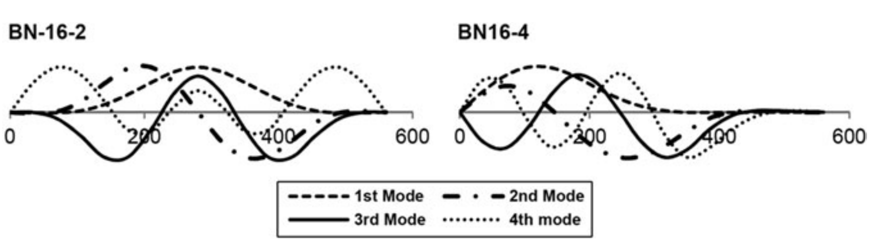

Figure 11. Elastic mode shapes of the viaducts BN-16-2 and BN-16-4.

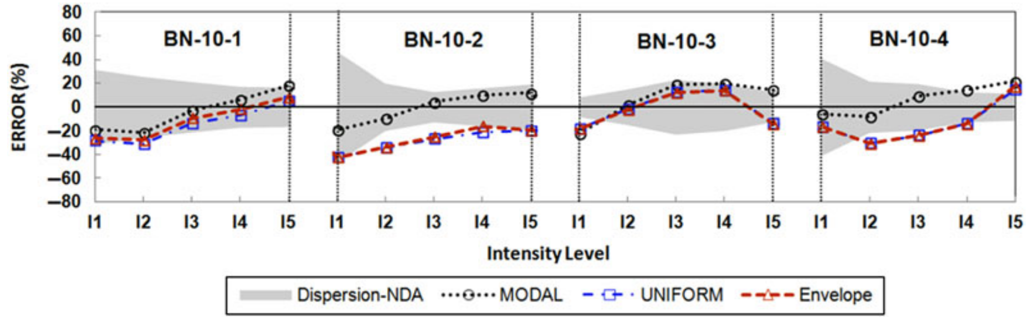

Figure 12. Accuracy of application of different load patterns for prediction of target displacement of 10-span bridges according to N2 method.

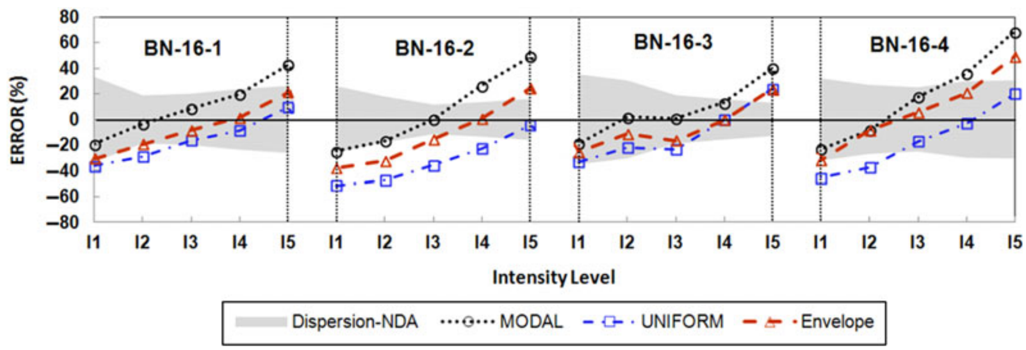

Figure 13. Accuracy of different load patterns for prediction of target displacement of 16-span bridges according to N2 method.

and 17, the displacement profiles of all configurations for 10- and 16-span bridges for intensity I4 are shown. The following can be concluded from the above-mentioned figures.

N2 method, although successfully estimates the displacement of the performance point of the examined viaducts at the monitoring node, fails to correctly predict the displacements on the stiff sides of the bridge. This fact is more emphasised for the cases with short piers in two sides (BN-10-2, BN-10-4, BN-16-2 and BN-16-4). As such, the estimated BI for these configurations is very small and even lower than the mean − 1SD. For the

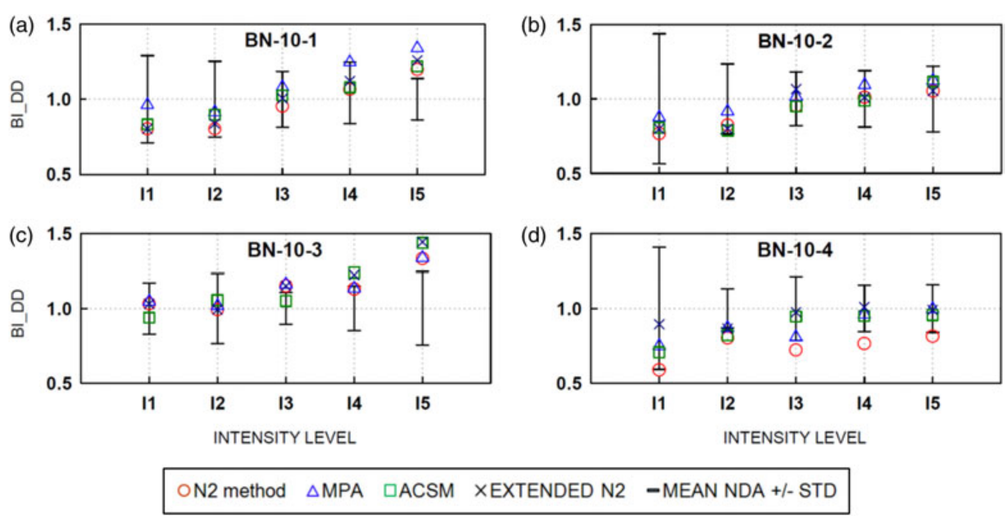

Figure 14. Comparison of BI_DD of 10-span viaducts for different intensity levels according to N2, MPA, ACSM and extended N2: (a) BN-10-1, (b) BN-10-2, (c) BN-10-3 and (d) BN-10-4.

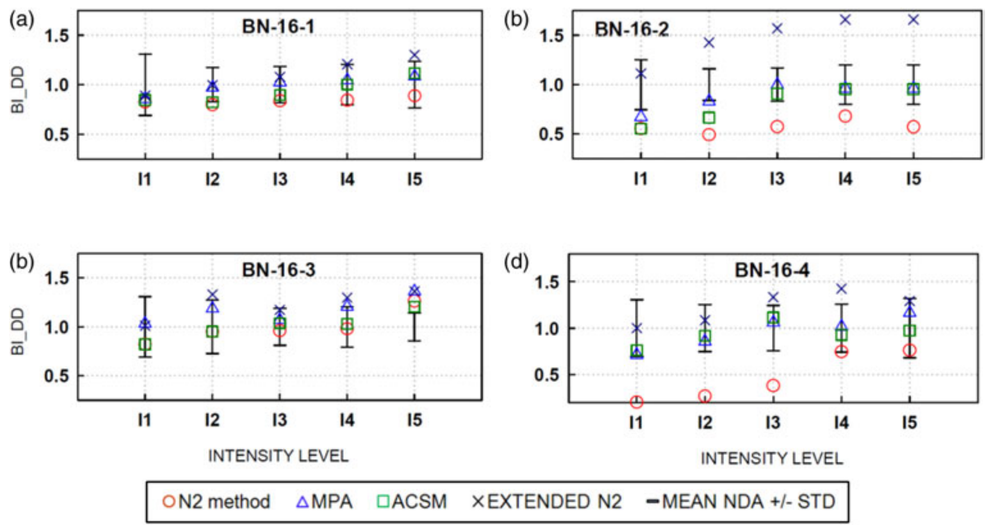

Figure 15. Comparison of BI_DD of long viaducts for different intensity levels according to N2, MPA, ACSM and extended N2: (a) BN-16-1, (b) BN-16-2, (c) BN-16-3 and (d) BN-16-4.

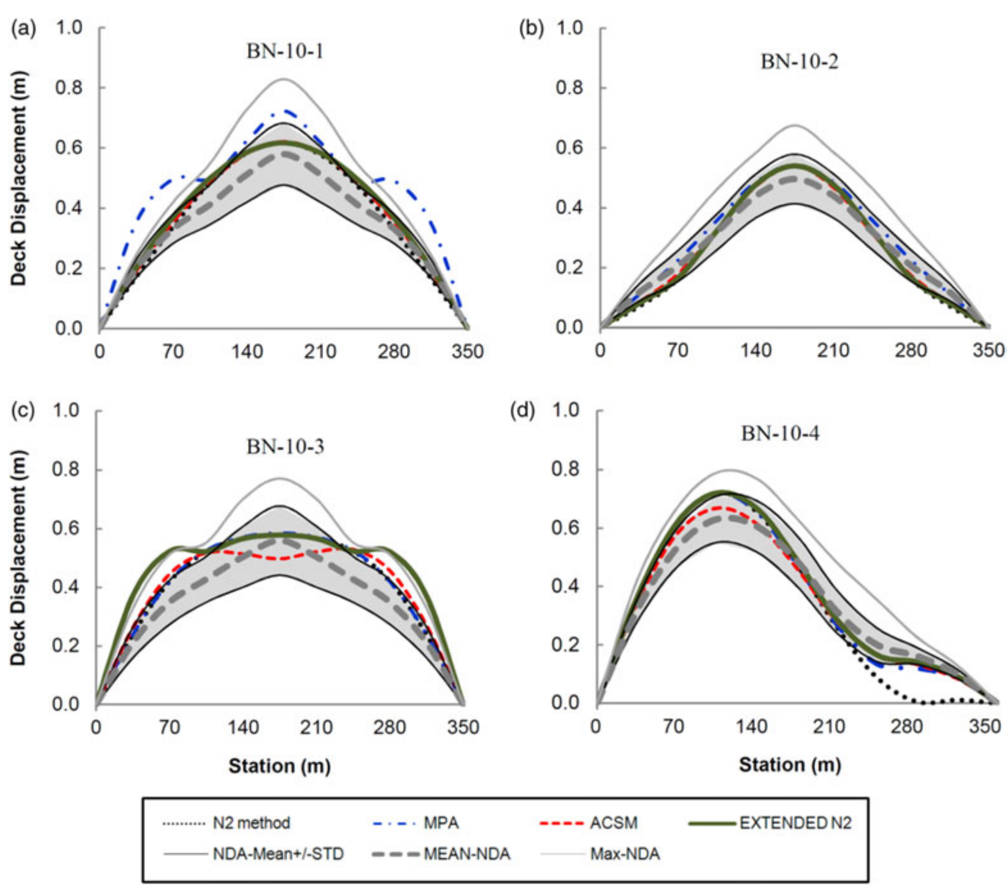

Figure 16. DD profiles for 10-span bridges obtained according to different NSPs compared with mean of NDA for intensity I4: (a) BN-10-1, (b) BN-10-2, (c) BN-10-3 and (d) BN-10-4.

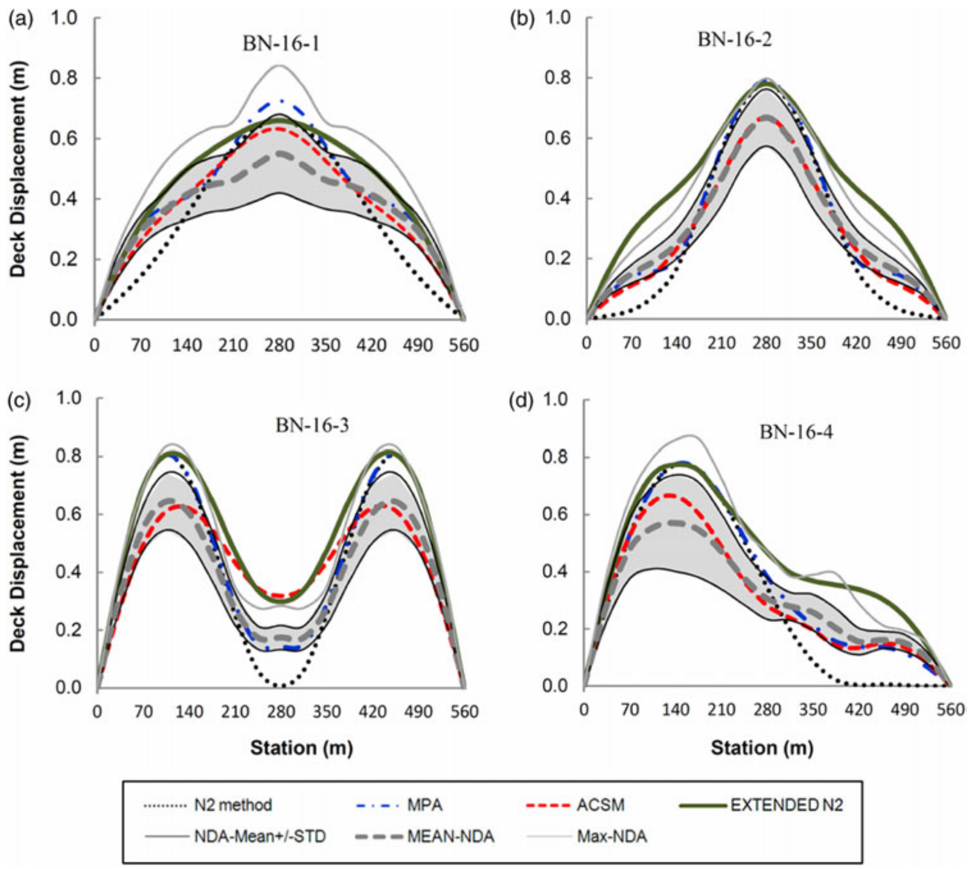

Figure 17. DD profiles for 16-span bridges obtained according to different NSPs compared with mean of NDA for intensity I4: (a) BN-16-1, (b) BN-16-2, (c) BN-16-3 and (d) BN-16-4.

most regular cases (BN-10-1 and BN-16-1), even though the modal mass participation factors are more than 80% (see Table 1), the displacement profile is underestimated on both sides close to the abutments.

The MPA, which considers higher mode effects, improves the displacement profiles. The BI_DD values derived according to this method, except for low-intensity levels and the very irregular cases are higher than one, both for 10- and 16-span bridges. ACSM, benefiting from the DAP analysis, is the best of all in capturing the displacement profile with consideration of higher mode effects and the changes in the mode shapes. The BI_DD values estimated by this method, except for low-intensity levels, estimate the closest results to the mean of NDA for all the cases.

Although ACSM, using the powerful pushover method of DAP, provides the best results for higher intensity levels, it is observed that, at lower intensity levels, this method underestimates the response. As mentioned before, although the displacement profile estimated by DAP provides the best match to the mean NDA, the equivalent viscous damping and the spectral reduction factors used in this method seem to have a poor performance for the lower ductility levels.

Extended N2 method, which is the simplest method for consideration of higher modes in the response, provides the most conservative results with respect to the other methods. In this method, the pushover analysis assesses the behaviour of those parts of the bridge where major plastic deformations occur, whereas the response spectral analysis determines the seismic demand at those parts where the Figures 16 and 17, extended N2 method produces the envelope of the results, and in some cases, it even exceeds the results obtained with maximum NDAs. In this method, the displacements at the maximum displacement point of the deck are similar to N2 method ($Cr = 1$) and the displacements in the other points are correspondingly corrected, based on RSA.

It should be noted, however, that except for N2 method in the most irregular cases, the BI factors estimated by all methods are between the mean $\pm$ SD of NDA results, which shows the good approximation of the multi-mode methods. BI_CS for two intensities of I1 and I3 for 10- and 16-span bridges are shown in Figure 18. For the lowest intensity level, similar to what was observed for BI_DD, the N2 method, especially for 16-span bridges underestimates the shear demand in the piers. The accuracy of MPA, ACSM and extended N2 method for low and high

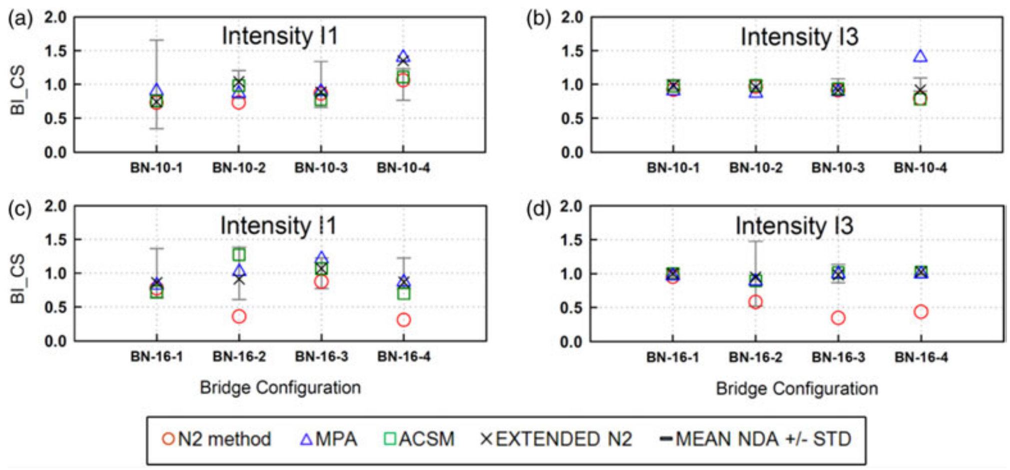

Figure 18. BI_CS of long viaducts for different intensity levels according to N2, MPA, ACSM and extended N2: (a) 10 spans (I1), (b) 10 spans (I3), (c) 16 spans (I1) and (d) 16 spans (I3).

intensities is good and BI values close to one are derived for all cases. By increasing the seismic intensity, the dispersion of the shear demand in the columns obtained from the NDA decreases.

Finally, it can be observed that lower values of RSI, related to medium and long span bridges, are associated with less accuracy of NSPs. Actually, the displacements profiles obtained by means of NSPs applied to medium and long span bridges (BN10 and BN16 configurations) are less concentrated near the mean value of the NDAs (Figures 16 and 17) than in the case of short span bridges (BN4 in Figure 8). However, the same trend cannot be associated with the RP. A physical explanation for these tendencies is the fact that RSI accounts for the relative stiffness of the deck and the columns, regardless of their location along the length of the bridges. On the other hand, the RP index depends also on the distribution of columns of different heights. This means that the factor that contributes more to the accuracy of the NSPs is the large stiffness of the deck in relation to the columns, as it softens the effect of the irregular distribution of columns.

7. Conclusions

Bridges and viaducts are key components of transportation infrastructure, whose potential performance in extreme situations is relevant, as inadequate behaviour leads to large economic disruption. In this paper, double-column bent viaducts with different lengths and levels of irregularity for a wide range of seismic intensity levels were assessed. RP and effective modal mass participation factor were applied for predefinition of the regularity level for different viaduct configurations. Although both parameters are related to the elastic state of the structure, it is observed that they are capable of gauging the irregularity of viaducts in the transversal direction. However, it is also observed that the irregularity effects also depend on the applied seismic intensity level. Moreover, the results show that the lower the RSI of the deck to piers is, the higher is the irregularity, and consequently the irregularity level is more emphasised for longer viaducts. This fact is also confirmed by evaluating modal mass participation factors.

Results of NDA performed for one set of seven matched records provides reliably close results compared to the use of three sets of seven matched records with a very small discrepancy from the mean of NDA, for the studied cases. This fact supports the idea of sufficiency of the use of one set of seven matched records in order to reduce analysis time and post processing efforts.

In short viaducts with four spans, due to the relatively high stiffness of the deck compared with piers, which is a common situation for short viaducts, the response is mainly governed by the first mode of vibration. Therefore, single-mode procedures as well as multi-mode NSPs, adaptive or non-adaptive, provide good results. This applicability is improved for higher intensity levels in the case of single-mode procedure. For the N2 method, despite the close results predicted for different load distributions, modal load pattern is preferred over uniform and envelope.

For the case of long bridges with 10 and 16 spans, even for the most regular cases, single-mode method (N2) fails to properly predict the seismic response, especially the shape of the displacement profile. Although this method works relatively well in estimating the displacement of the monitoring node (better for modal load pattern), the displacement profile, as a consequence of the weaknesses of pushover analysis, is underestimated in the stiff parts of the viaduct. In general, uniform and envelope load patterns provide slightly improved base shear capacity and displacement profile prediction; however, they lead to the underestimation of the displacement at monitoring node.

MPA, ACSM and extended N2 method, on the other hand, by considering higher modes in the estimation of the response, although in different ways, succeed to provide more precise results, which are mainly between the mean ± 1SD of the NDA. In addition, this accuracy increases as the seismic intensity increases. Although it was expected that ACSM, by using the powerful DAP algorithm, would match the NDA results, it underestimates the results, especially for lower intensity levels.

Finally, it should be noted that NSPs are introduced as simple methods for approximate assessment or design of structures and precise prediction by these methods cannot be expected for every structure and for every ground motion. However, this study shows that MPA, ACSM and the extended N2 method are reliable methods for the case of irregular bridges, although the latter keeps the simplicity of the method while providing the most conservative results. Although MPA and ACSM are shown to be adequate for irregular bridges, the authors would suggest the use of extended N2, as it is simpler and provides safe results.

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