

Evaluation of Nonlinear Static Procedures for Seismic Assessment of Irregular Bridges

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SUMMARY

Nonlinear static procedures (NSPs) have recently become a popular tool for seismic assessment of buildings and bridges. Many studies have demonstrated the accuracy of such procedures for regular structures. However, the reliability of their application for irregular structures still needs to be addressed. The main goal of this study is focused on the evaluation of the most commonly employed NSPs, such as CSM, N2, MPA and ACSM, applied for irregular reinforced concrete viaducts. As such, a set of irregular bridges with different levels of irregularity and configurations are investigated. A suite of seven ground motion records are selected and matched with the design spectrum. 3D Nonlinear static analyses- conventional and adaptive- along with 3D nonlinear dynamic analyses are developed for all of the cases. The accuracy of different NSPs is evaluated by comparing NSP results with nonlinear dynamic analysis. Finally, based on the comparisons and observations, suggestions for the preferred NSP for assessment of reinforced concrete irregular bridges along with recommendations for possible improvements of the applied procedures will be presented.

Keywords: Nonlinear Static Procedures, RC viaducts, Seismic assessment

1. INTRODUCTION

Nonlinear Static Procedures, NSPs, have recently become standard tools for seismic assessment and design of structures. In the last decade or so considerable efforts have been placed in the development of these methods. These efforts have led to the introduction of the methods in guideline documents such as ATC-40 (ATC, 1996), FEMA273 (FEMA, 1997), etc or design codes, such as Eurocode 8 (CEN, 2005).

These guidelines are more focused on the seismic assessment of buildings rather than bridges and consequently do not explicitly address the differences between various types of structures. As such, application of these methods without enough attention into these differences can result in unacceptable or even sometimes erroneous results. Only recently considerable attempts have been made to verify the application of NSPs for bridge structures.

A set of studies have been carried out implementing different NSPs based on single-mode (e.g. N2 method), multi-mode non-adaptive (e.g. MPA (Chopra and Goel, 2002)) and adaptive pushover methods (e.g. IRSA (Aydinoglu MN., 2004)) for short and long irregular single column bent viaducts (Isakovic and Fischinger, 2006, 2008). In general, these studies show that, in short viaducts with the first modal effective mass of more than 80% and a relative stiff superstructure, single-mode N2 method provides acceptable results compared with nonlinear dynamic analyses; in addition, this accuracy is increased for high intensity ground motions. These studies show that for long viaducts, higher modes are more significant and N2 method is not capable to address this effect. Nevertheless MPA and IRSA provide more accurate results although MPA fails to show compatible results with nonlinear time history analysis for high intensity ground motions.

A similar study was implemented for an experimentally tested short double column bent bridge (Isakovic and Fischinger, 2011). The results show that MPA and IRSA lead in to the most accurate response compared with the experimental results. In addition, it was shown that although extended N2

method (Faifar et al., 2005) tends to provide better results than N2 method, its results are less accurate than IRSA and MPA.

A parametric study in order to evaluate the ability of four different NSPs (CSM, N2, MPA and ACSM (Casarotti and Pinho, 2007)) in predicting the structural response of a wide range of single column bent viaducts was recently developed in the scope of a Ph.D thesis (Monteiro, 2011). This study show that, although the performance of single mode methods can be improved by a good selection of analysis parameters such as monitoring point, load distribution and spectral reduction factors, ACSM followed by MPA were considered as the most reliable methods for the studied cases.

Other efforts have been devoted to investigate the extension of the modal pushover method to bridges and the investigation of its applicability in the case of complex bridges (Kappos & Paraskeva, 2008). The study shows that MPA can be considered a more promising approach that yields to better results when compared to the ‘standard’ pushover analysis.

Although these studies show reliable findings about the applicability of NSPs for the design and assessment of viaducts, in all of them, a need for further research for other bridge configurations, mainly for irregular bridges, is pointed out. In addition, most of the mentioned studies were performed for single column viaducts and the definition of irregularity and its different possible types were not explicitly studied.

As such, in the present work, a set of studies aiming to evaluate different NSPs, such as CSM-FEMA440, N2 method, MPA and ACSM, on different groups of double column bent motorways, typical in modern motorway construction in Europe, are performed. Several parameters have been used for definition of irregularity of bridges and proper indices are associated to each configuration. The bridges are defined as short, medium and long viaducts with four different levels of irregularity. The analyses are carried out for two different seismic intensities in order to gauge the applicability of the procedures for different pier ductility demands. Nonlinear Dynamic Analyses, NDAs, for a set of seven ground motions matched with design spectrum are also carried out in order to be used as the most precise available analysis method. Finally, a comparison between different NSPs and DNAs is presented. The main goal of this study is to determine whether or not, conclusions previously obtained in the aforementioned studies could be extended to double column viaducts with various types of irregularities.

2. CASE STUDIES AND SEISMIC ACTION

2.1. Analyzed Bridges

Double column bents with different degrees of irregularity are selected. 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 bridges. 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 in previous studies (Calvi et al, 1993), is applied here as a tool for definition of irregularity. RP value can be defined as follows:

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

The eigenvectors of the deck, with and without the stiffness of the piers, are defined, respectively, by ψ_j and ϕ_j and the mass matrix is defined by [M]. 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 zero to one. 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 to values less than 1.0 based on the level of irregularity. It is observed that the less is the relative stiffness of deck to piers, the more is the irregularity of the structure. As such, Relative Stiffness Index, RSI, (Priestley et al, 2003) is simply defined as the ratio of the lateral stiffness of the deck to the total lateral stiffness of the piers. For bridges with fixed abutments, this value can be approximately defined according to the following expression:

$$RSI = \frac{K_s}{\sum K_p} = \frac{384E_s I_s}{5L_s^3} \sum \frac{H_p^3}{C_p E_p I_p} \quad (2)$$

In which: E_s , I_s , L_s : are Young modulus, moment of inertia and total length of the superstructure, respectively. E_p , H_p , 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 in the two ends and ranges from 3 to 12 for the one built in ends and both built in ends in bridge piers, respectively. In such calculation it is suggested to use uncracked section for the superstructure and cracked-section for the piers. Therefore, a moment of inertia of $0.5I_p$ will be used to account for the cracking of the piers.

Both procedures presented here are only able to properly show the irregularity of bridge structure in elastic range, whereas for high intensity seismic actions, the bridge regularity changes by development of plastic hinges.

In this work, four types of viaducts with different regularity levels, from regular to very irregular, and three different lengths of 140, 350 and 560 m with fixed abutments have been selected. The representative of each group and the geometry of deck and the typical pier cross section are shown in Figure 2.1. Each bridge is designated by a Bridge Number which includes two numbers. The first number shows the number of bridge spans and the second number shows the irregularity level, in which 1 stands for the most regular bridge and 4 stands for the most irregular bridge, consequently, a total of 16 bridge configurations have been considered.

A summary of all considered bridges, modal properties, RP and RSI are listed in Table 2.1. As can be seen in the table, according to this study, the most irregular bridges are the ones with stiff short columns in one side of the bridge and long piers in the other side. For longer viaducts with small RSI value, more irregular behavior is observed and relatively smaller RP values are derived. However, for short viaducts, specifically 4 spans, RP value is relatively high even for very irregular column distributions which can be attributed to the high stiffness of the deck compared to piers. In only two cases of BN-10-2 and BN-16-2 the consistency between the two methods is not observed, in which the RP value is high but the modal mass participations of the first modes are small. All of the bridges have been designed based on response spectral analysis according to EC8 for a design Peak Ground Acceleration (PGA) of 0.26g.

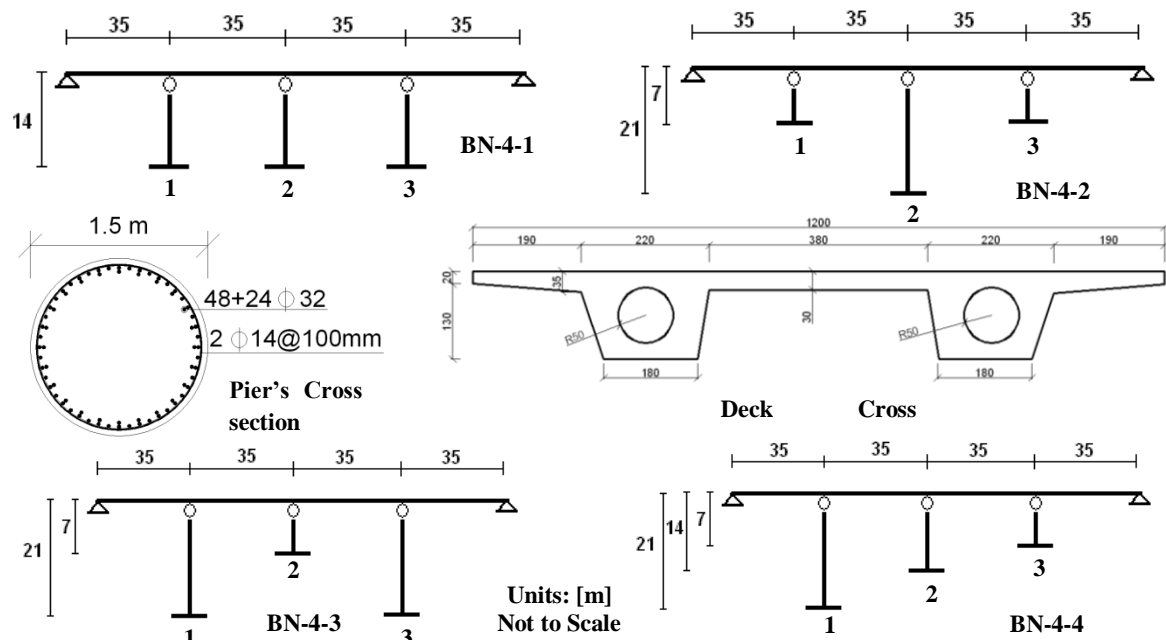


Figure 2.1. Typical selected bridge configurations and the cross section properties

Table 2.1. Selected Bridge Configurations and Modal Properties

Bridge Number	Designation ²	RSI	RP	Period (s) ¹	Modal mass participation (%)		
					1 st mode	2 nd mode	3 rd mode
BN-16-1	2222222222222222	0.008	1.000	1.41	82	9	3
BN-16-2	1112223332221111	0.002	0.952	1.72	46	19	25
BN-16-3	3332221112223333	0.004	0.884	1.54	70	6	20
BN-16-4	3333322222111111	0.003	0.429	1.98	43	11	27
BN-10-1	2222222222	0.05	1.000	1.13	82	9	3
BN-10-2	112232211	0.01	0.926	1.07	54	31	9
BN-10-3	332212233	0.04	0.892	1.02	75	13	7
BN-10-4	333222111	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

1- Period of the first transversal mode

2- 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)

2.2. Modeling Assumptions

Finite element analyses were carried out using SeismoStruct (Seismosoft, 2005). The piers are modeled through a 3D inelastic beam-column element. The constitutive laws of the reinforcing steel and of the concrete are considered with strength of 500 and 33MPa, respectively. Applied models are described in related papers (Menegotto & Pinto, 1973 and Mander et al., 1988). The deck is a 3D elastic beam-column element, fully characterized by the sectional property values, based on Young and shear modules of 25 and 10 GPa, moment of inertia of 2.15 and 67.2 m⁴ and a torsional constant of 1.46; a 2% Raleigh damping was assigned to the deck, proportional to the two first transversal modes of the structure. The shear capacity of the piers according to UCSD (Priestley et. al, 2003) model is estimated to be 3478 kN.

2.3. Seismic Action

Seven seismic excitations of real earthquakes recorded on soil type B according to EC8 are selected. All seven records were matched with design spectrum for a PGA of 0.26g (Type 1 Soil B of EC8). SeismoMatch (Seismosoft, 2011) was used to match the records with design spectrum for the period range of 0.05 to 2.0s. Figure 2.2 shows the design, matched and mean spectra and in Table 2.2 a summary of the selected records is listed. SeismoMatch is an application capable of adjusting earthquake accelerograms to match a specific target response spectrum, using the wavelets algorithm (Abrahamson, 1992 & Hancock et al., 2006).

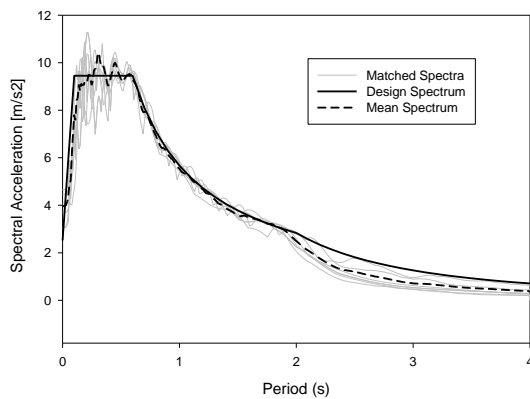


Figure 2.2. Design spectrum and matched records

Table 2.2. Details of Selected Records

Earthquake Name	Date	M_w	Fault Type	Dist ¹ (km)	PGA ² m/s ²	EC8 Site class
Friuli	5/6/1976	6.5	thrust	42	4.5	B
Campano Lucano	11/23/1980	6.9	normal	16	3.1	B
Ano Liosia	9/7/1999	6.0	normal	19	4.7	B
South Iceland Strofades	6/17/2000	6.5	strike slip	15	3.7	B
Aigion	11/18/1997	6.6	oblique	38	3.1	B
South Iceland	6/15/1995	6.5	normal	43	4.1	B
Iceland	6/21/2000	6.4	strike slip	14	3.8	B

1. Epicentral Distance
2. PGA of the scaled fitted records

3. APPLICABILITY OF DIFFERENT NSPs

3.1. Short Viaducts - 4 Spans

The results obtained for short bridges are presented in this section in Figure 3.1.

Figure 3.2 shows the comparison of shear demand in bridge piers. In the following, the main issues observed in the analyses of the 4 spans bridges are presented:

According to the suggestions by Kappos et al. the monitoring node was selected as the node with maximum displacement. In all of the cases, preferably modal load distribution was implemented. In the application of N2 method, because of the relatively large hardening slope of the pushover curves after the plastification of all of the columns, it is observed that the use of bilinear idealization yields to better results than the suggested elasto-plastic idealization proposed by the original method.

Because of the slightly high stiffness of the deck compared to piers' stiffness as well as the restrained condition in the abutments, the deck displacement profile rather than piers' specifies the governing displacement profile. In addition, a considerable portion of the base shear, especially after yielding of the columns, is carried by the abutments.

Since the effective modal mass of the first transversal mode in all of the models is higher than 80%, as was expected, the higher modes do not have significant effect on the response of the structure. As such, single mode procedures lead to good estimation of the displacement profile for the superstructure. However, N2 method rather than CSM seems to be a more appropriate method for the estimation of the maximum displacement as well. CSM-FEMA440, on the other hand, underestimates the maximum displacements. This fact can be explained by the relatively high effective damping proposed by this method which consequently leads to a high spectral reduction factor. MPA in almost all of the examined cases provides the closest results to the median dynamic analysis for high intensity level; however it overestimates the responses for low intensity level.

Displacement based Adaptive Pushover analysis (Antonio and Pinho, 2004) was applied in the ACSM procedure. Since the higher modes and more importantly the changes in the modes are not much relevant for short viaducts, the adaptive pushover yields more or less to the same curves as the conventional methods.

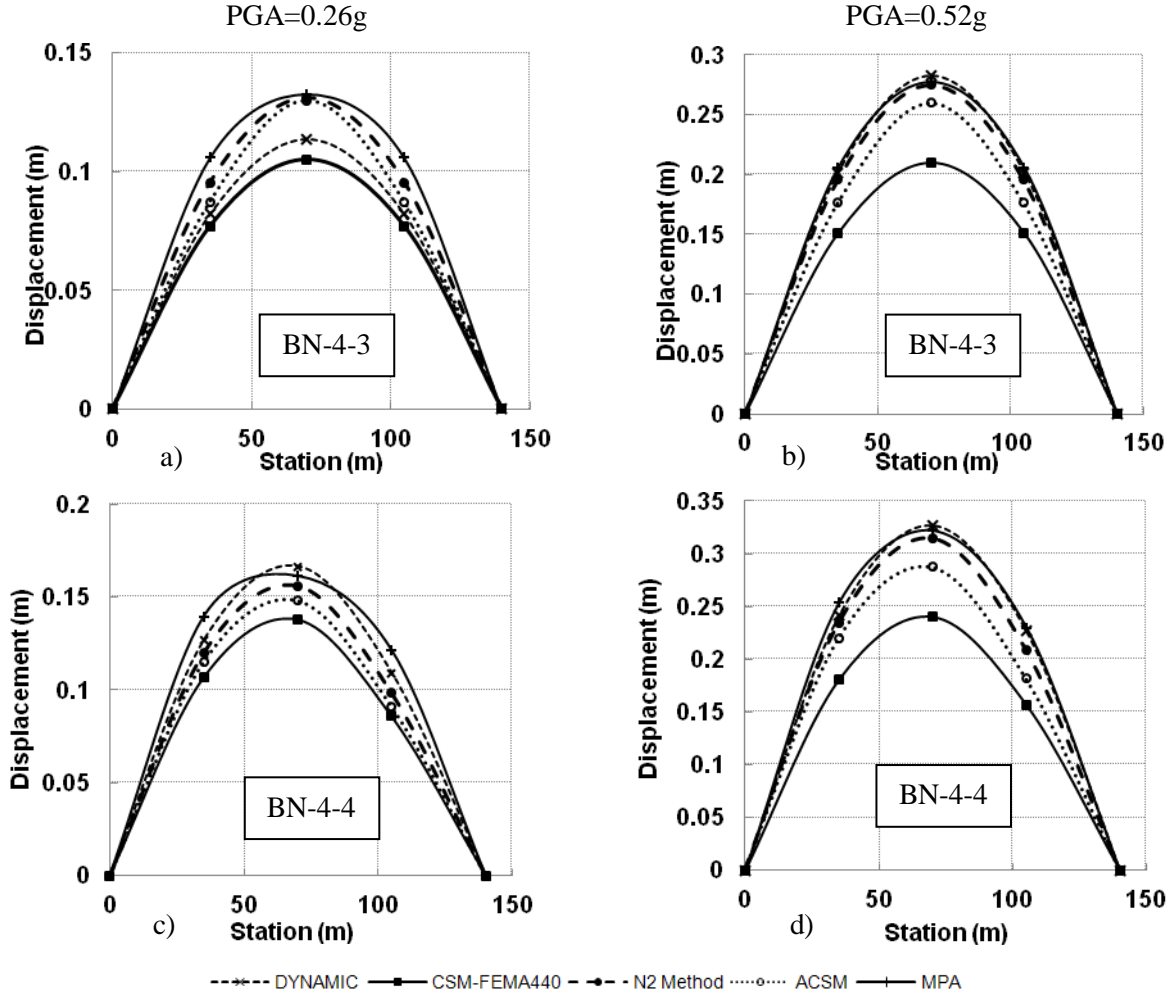


Figure 3.1. Displacement response of short bridges obtained from different NSPs compared with Nonlinear Dynamic for: a) PGA=0.26g, BN-4-3, b) PGA=0.52g, BN-4-3, c) PGA=0.26g, BN-4-4 & d) PGA=0.52g, BN-4-4

As mentioned before, regardless of the high ductility demand in the columns and their high plasticity especially for the higher intensity level, since a high portion of the forces are transferred to the abutments, the pushover curve seems to be linear or with very high hardening slope in the nonlinear range. As such, in the application of ACSM method a small ductility factor is estimated from the pushover curve. On the other hand, the spectral reduction factor applied for this study (Priestley et al. 2007), which is a common use for ACSM and is based on the ductility factor and the equivalent viscous damping which seems to be less accurate for low ductility levels. Therefore, the results provided by ACSM for short viaducts don't properly capture the correct transversal performance of short viaducts. Shear demand estimated by different methods are relatively similar and most of them provide results close to the median dynamic response. However, as can be seen in Figure 3.2, it is observed that MPA overestimates the shear demand in the short columns.

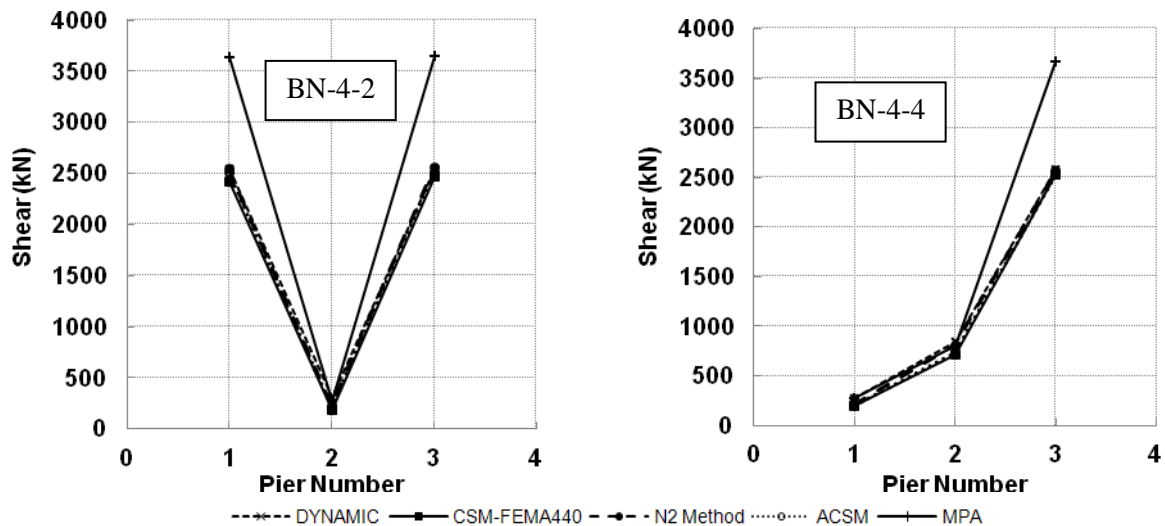


Figure 3.2. Shear demand of piers of short bridges obtained from different NSPs compared with Nonlinear Dynamic for seismic intensity of 0.26g
 (Note: the lines in the figure are just to emphasize the trend and do not have any physical meaning)

3.2. Long Viaducts - 10 and 16 spans

In this section the results obtained from the analysis of long viaducts are discussed.

Figure 3.3 shows the results of the deck displacement response for NSPs and NDAs for the most irregular cases and for two intensity levels. In

Figure 3.4 the shear demand of the piers are depicted. Some of the important issues observed from the results are presented in the following.

Selection of monitoring node to develop the pushover curve of single mode and in-adaptive multi mode methods in long viaducts is observed to be more important compared to short viaducts, in which selection of different nodes can lead to different results. Thus the selection of the control node is more relevant for the highly irregular cases. In the regular configurations the centre of mass and maximum displacement points are usually in the same location and it was 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 by with the intensity level. In this study, the predicted maximum displacement point by linear modal analysis was selected as the monitoring point and this fact is left as an uncertainty in the analysis.

For long viaducts with short piers along the bridge, the application of modal and uniform load distributions (recommended by EC8) leads to poor results. The former underestimates the deck displacements response in the stiff part of the viaduct and the latter underestimates the response in the more flexible side. It is observed that, an envelope of the results derived from the two load patterns can improve the results and this approach is applied in this paper.

In the most regular cases (BN-10-1 and BN-16-1) in which the modal mass participation factor of the first mode is 82% and the RP value is 1.0, the maximum displacement in the centre of the bridge is well estimated by N2 method based on the modal load distribution. However, the deck displacement in two sides close to abutments is underestimated. For viaducts with short piers close to the abutments and longer piers in the middle (BN-10-2 and BN-16-2) the same behaviour is observed. In these configurations, the displacement in the sides of bridge is subject to the second mode of vibration, as such MPA and ACSM tend to properly capture this mode. Even though, MPA is preferred for the lower intensity level and ACSM for the higher intensity.

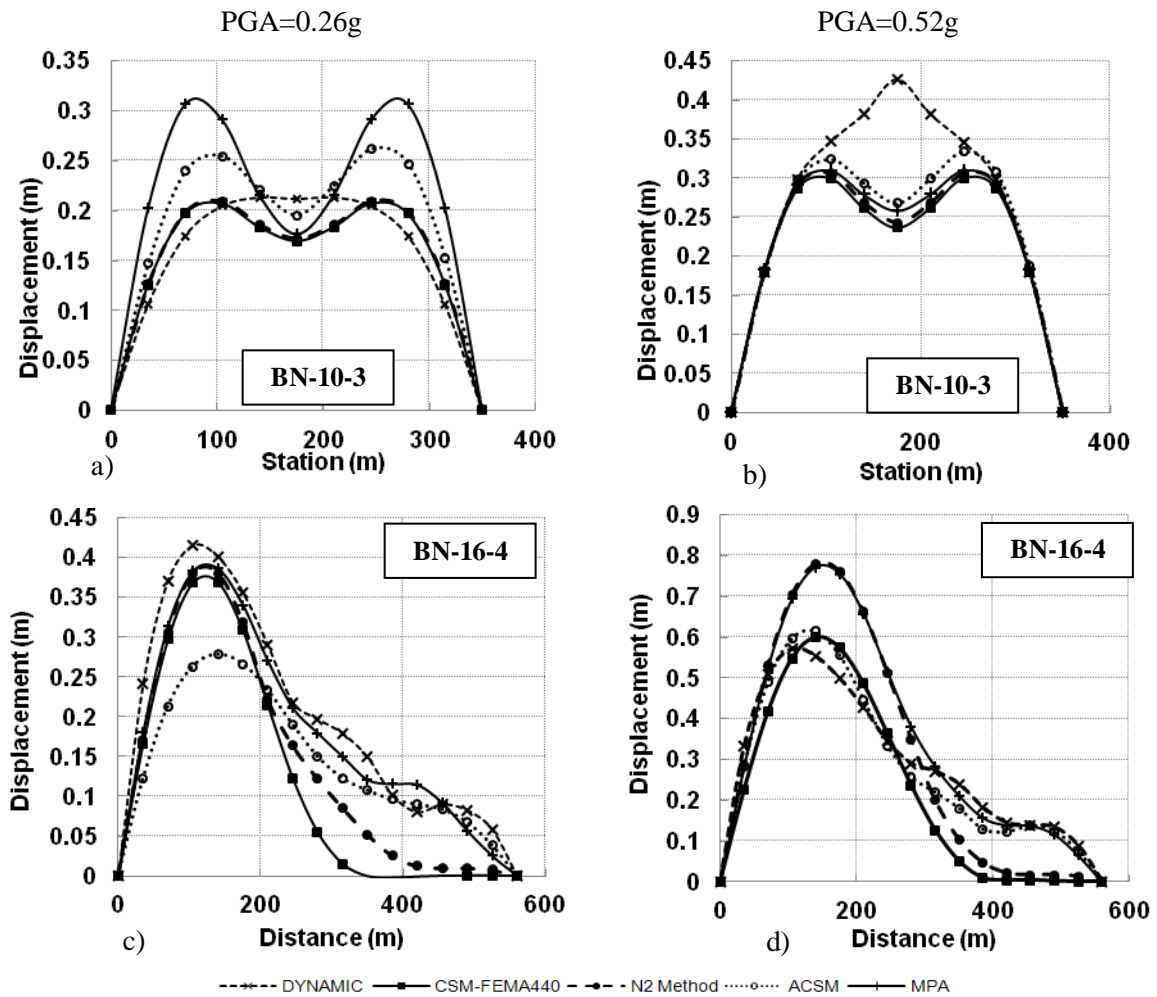


Figure 3.3. Displacement response of long bridges obtained from different NSPs compared with Nonlinear Dynamic: for:

a) PGA=0.26g, BN-4-3, b) PGA=0.52g, BN-4-3, c) PGA=0.26g, BN-4-4 & d) PGA=0.52g, BN-4-4

Unless for the 10 spans bridge with very stiff pier in the centre of the viaduct (BN-10-3), both conventional and adaptive pushover analyses can predict the deck displacement pattern very well. In BN-10-3, the central pier yields in the early stages of the loading. This leads into a drop of stiffness in this pier and consequently higher displacement on top of this pier is predicted by nonlinear dynamic analysis which is overlooked by conventional pushover analysis. Moreover, although Adaptive Pushover Analysis provides the closest displacement pattern, it fails to match completely with the median of dynamic response, especially in the stiff part of the viaduct. The shear demand in the piers is overestimated almost by all of NSPs compared with Nonlinear Dynamic Analyses results. However, for the most irregular configurations (BN-10-4 and BN-16-4), the shear demand in short piers, unless for MPA, is underestimated. Therefore, seems to be the most conservative method for estimation of shear demand in the piers. It should be mentioned that this overestimation by MPA method is more emphasized for higher intensity level than for the low intensity. This can be explained by the fact that MPA does not take in to account the modal coupling effect and the changes in the modal shapes as the structure moves in to the nonlinear range.

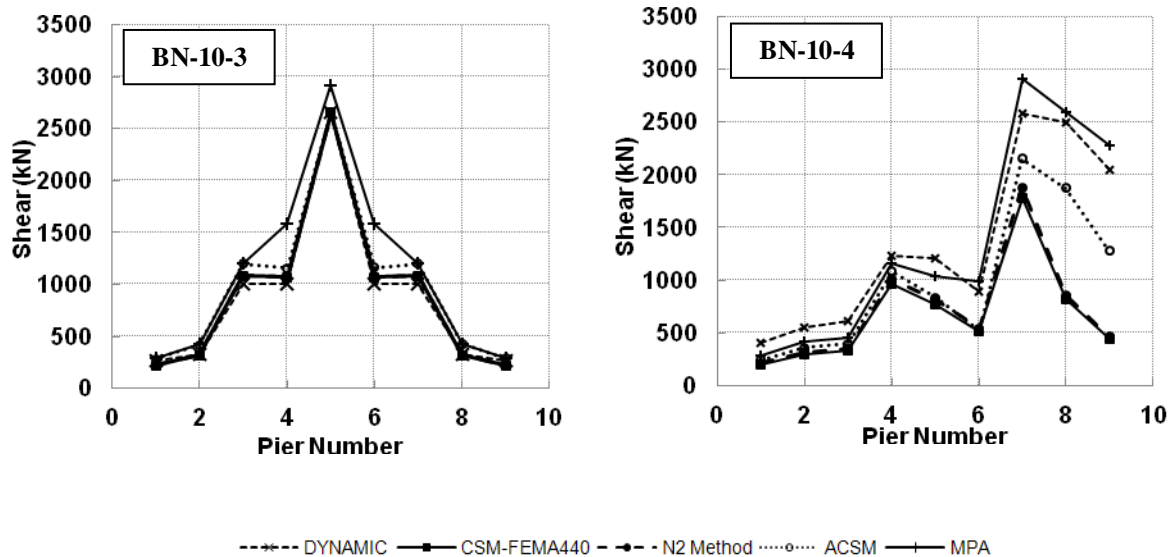


Figure 3.4 Shear demand of piers of 10 spans bridges obtained from different NSPs compared with Nonlinear Dynamic for seismic intensity of 0.26g
(Note: the lines in the figure are just to emphasize the trend and do not have any physical meaning)

4. CONCLUSIONS

Double column viaducts with different lengths and levels of irregularity for different seismic intensity were examined. Regularity Parameter (RP) and effective modal mass participation factor are applied for predefinition of the regularity level for different viaduct configurations. Although both parameters are related to elastic state of the structure, it is observed that they are highly capable of gauging the irregularity of viaducts in transversal direction. However, it is also observed that the irregularity of the viaduct also depends on the applied seismic intensity level. Moreover, the results show that the lower is the Relative Stiffness Index (RSI) of the deck to piers, the higher is the irregularity, and consequently the irregularity level is more emphasized for longer viaducts. In short viaducts with 4 spans, because of the relatively higher stiffness of the deck compared to piers, which is a common case for short viaducts, the response is more governed by the first mode of the structure. Therefore, single mode procedures as well as multi mode NSPs provide acceptable results. This applicability is improved for higher intensity level in the case of single mode procedures. In long bridges with 10 and 16 spans, unless for the most regular case, single mode methods (N2 and CSM) fail to properly predict the seismic response. They overestimate the displacement response in the flexible part of the viaduct and underestimate the ones in the stiffer part. However, it is observed that application of envelope of the results derived from modal and uniform pushover analysis can slightly improve the results of N2 method. Moreover, it seems that for specific bridge configurations (e.g. BN-10-2& 3 and BN-16-2& 3), single mode methods can predict the results better for higher intensity which is due to the significance of the first mode in higher intensity level. MPA and ACSM, because of their multi modal procedure, seem to be the best methods for long bridges; however, it is observed that MPA provides better results for low intensity level. This fact is due to the small changes of the modal properties of the structure in lower intensity level. On the other hand, ACSM leads to better results for higher intensity level. It seems that the damping and spectral reduction factors, proposed by this method, are well defined for high structural plasticity, or better saying for higher ductility demands, which are not applicable for low ductility demands or linear states.

Finally, based on the results of the performed study, further studies seem to be necessary to improve the results of ACSM for lower intensity levels. In addition, in order to improve the results of the original N2 method for irregular viaducts further studies are being performed by the authors examining, whether or not, the extended N2 method is capable of capturing the higher mode effects.

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