

A comparison of single-run pushover analysis techniques for seismic assessment of bridges

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SUMMARY

Traditional pushover analysis is performed subjecting the structure to monotonically increasing lateral forces with invariant distribution until a target displacement is reached; both the force distribution and target displacement are hence based on the assumption that the response is controlled by a fundamental mode, that remains unchanged throughout.

However, such invariant force distributions cannot account for the redistribution of inertia forces caused by structural yielding and the associated changes in the vibration properties, including the increase of higher-mode participation. In order to overcome such drawbacks, but still keep the simplicity of using single-run pushover analysis, as opposed to multiple-analyses schemes, adaptive pushover techniques have recently been proposed.

In order to investigate the effectiveness of such new pushover schemes in assessing bridges subjected to seismic action, so far object of only limited scrutiny, an analytical parametric study, conducted on a suite of continuous multi-span bridges, is carried out. The study seems to show that, with respect to conventional pushover methods, these novel single-run approaches can lead to the attainment of improved predictions. Copyright © 2007 John Wiley & Sons, Ltd.

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KEY WORDS: displacement based; adaptive pushover; seismic analysis; bridges; DAP; FAP

1. INTRODUCTION

The term ‘pushover analysis’ describes a modern variation of the classical ‘collapse analysis’ method, as fittingly described by Kunnath [1]. It refers to an analysis procedure whereby an

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incremental-iterative solution of the static equilibrium equations has been carried out to obtain the response of a structure subjected to monotonically increasing lateral load patterns. Whilst the application of pushover methods in the assessment of building frames has been extensively verified in the recent past, nonlinear static analysis of bridge structures has been the subject of only limited scrutiny [2]. Since bridges are markedly different structural typologies with respect to buildings, observations and conclusions drawn from studies on the latter cannot really be extrapolated to the case of the former, as shown by Fischinger *et al.* [3], who highlighted the doubtful validity of systematic application of standard pushover procedures to bridge structures.

Recent years have also witnessed the development and introduction of an alternative type of nonlinear static analysis [4–10], which involve running multiple pushover analyses separately, each of which corresponding to a given modal distribution, and then estimating the structural response by combining the action effects derived from each of the modal responses (i.e. each displacement–force pair derived from such procedures does not actually correspond to an equilibrated structural stress state). As highlighted by some of their respective authors, the main advantage of this category of static analysis procedures is that they may be applied using standard readily available commercial software packages, since they make use of conventional analysis types. The associated drawback, however, is that the methods are inevitably more complex than running a single pushover analysis, as noted by Maison [11], for which reason they do not constitute the scope of the current work, where focus is instead placed on single-run pushover analysis procedures, the simplicity of which renders them an even more appealing alternative, or complement, to nonlinear dynamic analysis [12].

In this work an analytical parametric study is thus conducted applying different single-run pushover procedures, either adaptive or conventional, on a number of regular and irregular continuous deck bridges subjected to an ensemble of ground motions. The effectiveness of each methodology in reproducing both global behaviour and local phenomena is assessed by comparing static analysis results with the outcomes of nonlinear time-history runs. Adaptive pushovers are run in both their force-based [13–17] and displacement-based [18, 19] versions. With respect to the latter, it is noted that, contrary to what happens in a non-adaptive pushover, where the application of a constant displacement profile would force a predetermined and possibly inappropriate response mode that could conceal important structural characteristics and concentrated inelastic mechanisms at a given location, within an adaptive framework a displacement-based pushover is entirely feasible, since the loading vector is updated at each step of the analysis according to the current dynamic characteristics of the structure. The interested reader is referred to some of the aforementioned publications for details on the underlying formulations of adaptive pushover algorithms.

It is observed that whilst for regular bridge configurations some conventional single-run pushover methods may manage to provide levels of accuracy that are similar to those yielded by their more evolved adaptive counterparts, when irregular bridges are considered the advantages of using the latter become evident. In particular, the displacement-based adaptive pushover (DAP) algorithm is shown to lead to improved predictions, which match more closely results from nonlinear dynamic analysis.

2. PARAMETRIC INVESTIGATION: CASE-STUDIES AND MODELLING ASSUMPTIONS

The parametric study has considered two bridge lengths (four and eight 50 m spans), with regular, irregular and semi-regular layout of the pier heights and with two types of abutments; (i) continuous

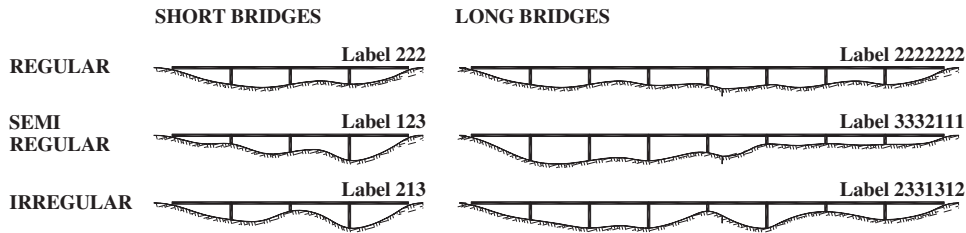


Figure 1. Analysed bridge configurations.

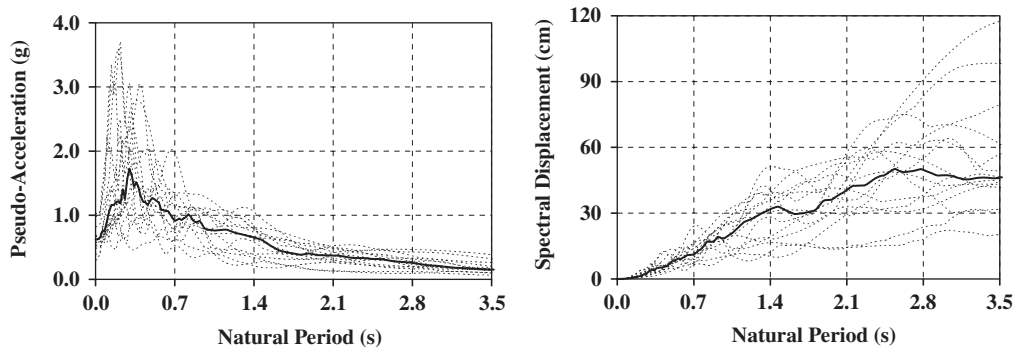


Figure 2. Elastic pseudo-acceleration and displacement spectra.

deck-abutment connections supported on piles, exhibiting a bilinear behaviour (type A bridges), and (ii) deck extremities supported on pot bearings featuring a linear elastic response (type B bridges). The total number of bridges is therefore 12, as shown in Figure 1, where the label numbers 1, 2, 3 characterize the pier heights of 7, 14 and 21 m, respectively.

Since the nonlinear response of structures is strongly influenced by ground motion characteristics, a sufficiently large number of records needs to be employed so as to bound all possible structural responses. The employed set of seismic excitation, referred to as LA, is thus defined by an ensemble of 14 records selected from a suite of historical earthquakes scaled to match the 10% probability of exceedance in 50 years (475 years return period) uniform hazard spectrum for Los Angeles [20]. The ground motions were obtained from California earthquakes with a magnitude range of 6–7.3, recorded on firm ground at distances of 13–30 km. The elastic (5% damped) pseudo-acceleration and displacement response spectra of the records are presented in Figure 2, where the thicker line represents the median spectrum, and the bounding characteristics of the records are summarized in Table I, where the significant duration is defined as the interval between the build up of 5 and 95% of the Total Arias Intensity [21].

The finite element package used in the present work, SeismoStruct [22], is a fibre-element-based program for seismic analysis of framed structures, which can be freely downloaded from the Internet. The program is capable of predicting the large displacement behaviour and the collapse load of framed structures under static or dynamic loading, duly accounting for geometric nonlinearities and material inelasticity. Its accuracy in predicting the seismic response of bridge

Table I. Bounding characteristics of the employed set of records.

	Peak ground acceleration (g)	Peak response acceleration (g)	5% Arias intensity threshold (s)	Significant duration t_{eff} (s)	Total duration t_{tot} (s)	$t_{\text{eff}}/t_{\text{total}}$ (%)
Min	0.30	0.84	1.25	5.3	14.95	9
Max	1.02	3.73	12.5	19.66	80.00	52

structures has been demonstrated through comparisons with experimental results derived from pseudo-dynamic tests carried out on large-scale models [23].

The piers are modelled through a 3D inelastic beam–column element, with a rectangular hollow section of 2.0×4.0 m and a wall thickness of 0.4 m; the constitutive laws of the reinforcing steel and of the concrete are, respectively, the Menegotto–Pinto [24] and Mander *et al.* [25] models. The deck is a 3D elastic nonlinear beam–column element, with assigned sectional properties, to which a 2% Rayleigh damping (for the first transversal modes of the structure) was also associated. The deck–piers connections are assumed to be hinged (no transmission of moments), transmitting only vertical and transversal forces, in order to model the engaging of the sub- and super-structure in the transversal direction by mean of shear keys. Further details can be found in [26].

Equivalent linear springs are used to simulate the abutment restraints, which should reflect the dynamic behaviour of the backfill, the structural component of the abutment and their interaction with the soil (type A bridges). Employed stiffness values for the bilinear and linear models were found, respectively, in Goel and Chopra [27] and from an actual bridge with similar dimensions and loads (type B bridges).

3. PARAMETRIC INVESTIGATION: ANALYSES AND RESULTS POST-PROCESSING

The response of the bridge models is estimated through the employment of (i) incremental dynamic analysis (IDA), (ii) force-based conventional pushover with uniform load distribution (FCPu), (iii) force-based conventional pushover with first mode proportional load pattern (FCPm), (iv) force-based adaptive pushover (FAP) and (v) displacement-based adaptive pushover (DAP).

Results are presented in terms of the bridge capacity curve, i.e. a plot of a reference point displacement *versus* total base shear and of the deck drift profile. Following Eurocode 8 [28] recommendations, the independent damage parameter selected as reference is the displacement of the node at the centre of mass of the deck: each level of inelasticity (corresponding to a given lateral load level or to a given input motion amplitude) is represented by the deck centre drift, and per each level of inelasticity the total base shear V_{base} and the displacements Δ_i at the other deck locations are monitored.

The ‘true’ dynamic response is deemed to be represented by the results of the IDA, which is a parametric analysis method by which a structural model is subjected to a set of ground motions scaled to multiple levels of intensity, producing one or more curves of response, parameterized *versus* the intensity level [29]. A sufficient number of records is needed to cover the full range of responses that a structure may display in a future event. An IDA curve set, given the structural model and a statistical population of records, can be marginally summarized (with respect to the independent parameter) by the median, the 16 and 84% fractiles IDA curves.

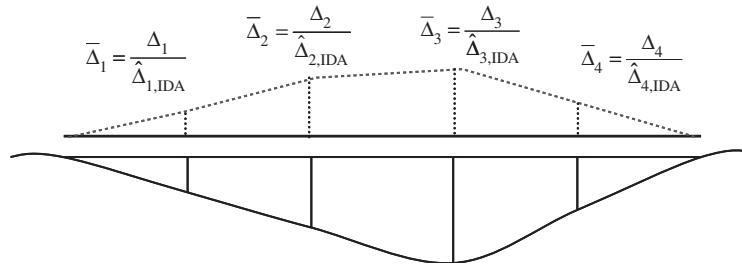


Figure 3. Normalized transverse deformed pattern.

Comparing pushover results with IDA output, obtained from ‘averaged’ statistics and fractile percentiles of all dynamic cases, allows avoiding the unreliable influence of single outlier values, which, statistically speaking, have reduced significance with respect to the population. From this point of view, robust measures of the means of the scattered data are used which are less sensitive to the presence of outliers, such as the median value, defined as the 50th percentile of the sample, which will only change slightly if a large perturbation to any value is added, and the fractiles as a measure of the dispersion.

Results of pushover analyses are compared to the IDA median value, for the 14 records, of each response quantity of interest; pier displacements (Δ_i) and base shear forces (V_{base}):

$$\hat{\Delta}_{i,IDA} = \text{median}_{j=1:14}[\Delta_{i,j-IDA}] \tag{1a}$$

$$\hat{V}_{base,IDA} = \text{median}_{j=1:14}[V_{base,j-IDA}] \tag{1b}$$

Results of adaptive pushover analyses with spectrum scaling, which thus become also record-specific, are statistically treated in an analogous way: medians of each response quantity represent that particular pushover analysis (i.e. FAP or DAP) with spectrum scaling.

The results of each type of pushover are normalized with respect to the corresponding ‘exact’ quantity obtained from the IDA medians, as schematically illustrated in Figure 3, and translated in the relationships (2a) and (2b). Representing results in terms of ratios between the approximate and the ‘exact’ procedures, provides an immediate indication of the bias in the approximate procedure: the ideal target value of each pushover result is always one.

$$\bar{\Delta}_{i,PUSHOVERTYPE} = \frac{\Delta_{i,PUSHOVERTYPE}}{\hat{\Delta}_{i,IDA}} \dots \xrightarrow{\text{ideally}} 1 \tag{2a}$$

$$\bar{V}_{base,PUSHOVERTYPE} = \frac{V_{base,PUSHOVERTYPE}}{\hat{V}_{base,IDA}} \dots \xrightarrow{\text{ideally}} 1 \tag{2b}$$

Moreover, normalizing results renders also ‘comparable’ all deck displacements (i.e. all normalized displacements have the same unitary target value), and thus a bridge index (BI) can be defined as a measure of the precision of the obtained deformed shape. Per each level of inelasticity, the BI is computed as the median of normalized results over the m deck locations, together with

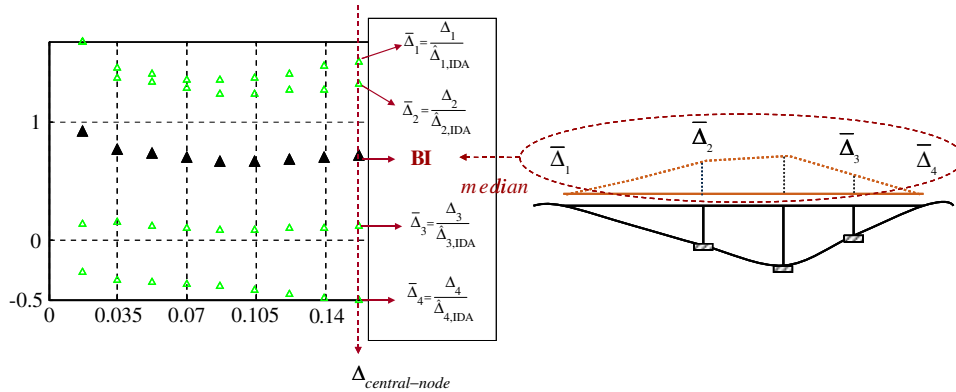


Figure 4. Graphical representation of normalized transverse displacements.

the standard deviation δ , to measure the dispersion of the results with respect to the median

$$BI_{PUSHOVERTYPE} = \text{median}_{i=1:m}(\bar{\Delta}_{i,PUSHOVERTYPE}) \tag{3a}$$

$$\delta_{PUSHOVERTYPE} = \left[\frac{\sum_{i=1}^m (\bar{\Delta}_{i,PUSHOVERTYPE} - BI_{PUSHOVERTYPE})^2}{m - 1} \right]^{0.5} \tag{3b}$$

The results obtained above can then be represented in plots such as that shown in Figure 4, where each increasing level of inelasticity (here represented by the deck central node displacement, indicated in the horizontal axis) of the BI, computed through Equation (3a), is represented with black-filled symbols, whilst the grey empty marks represent the values of which the BI is the median, i.e. the normalized deck displacements given by Equation (2a). In this manner, the extent (in terms of mean and dispersion) to which each pushover analysis is able to capture the deformed pattern of the whole bridge, with respect to the IDA average displacement values, at increasing deformation levels is immediately apparent.

4. PRELIMINARY STUDY: SELECTION OF BEST PUSHOVER OPTIONS

According to Eurocode 8 [28] pushover analysis of bridges must be performed by pushing the entire structure (i.e. deck and piers, see Figure 5 left) with the two aforementioned prescribed load distributions (uniform load distribution and first mode proportional load pattern). In this preliminary study, the possibility of pushing the deck alone (Figure 5, right) has been investigated; the choice of pushing just the deck was considered observing that, at least for the case-studies considered, the superstructure is the physical location where the vast majority of the inertia mass is found.

An additional preliminary investigation regarded the understanding of which adaptive pushover modality (with or without spectral amplification) would lead to the best results. Including *spectrum scaling* in the computation of the load vector allows accounting for the influence that the frequency

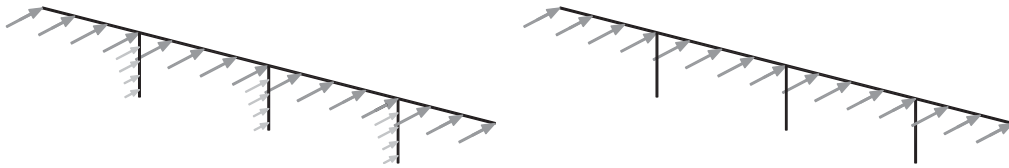


Figure 5. ‘Pushing deck and piers’ (left) and ‘pushing just the deck’ (right) loading scheme options.

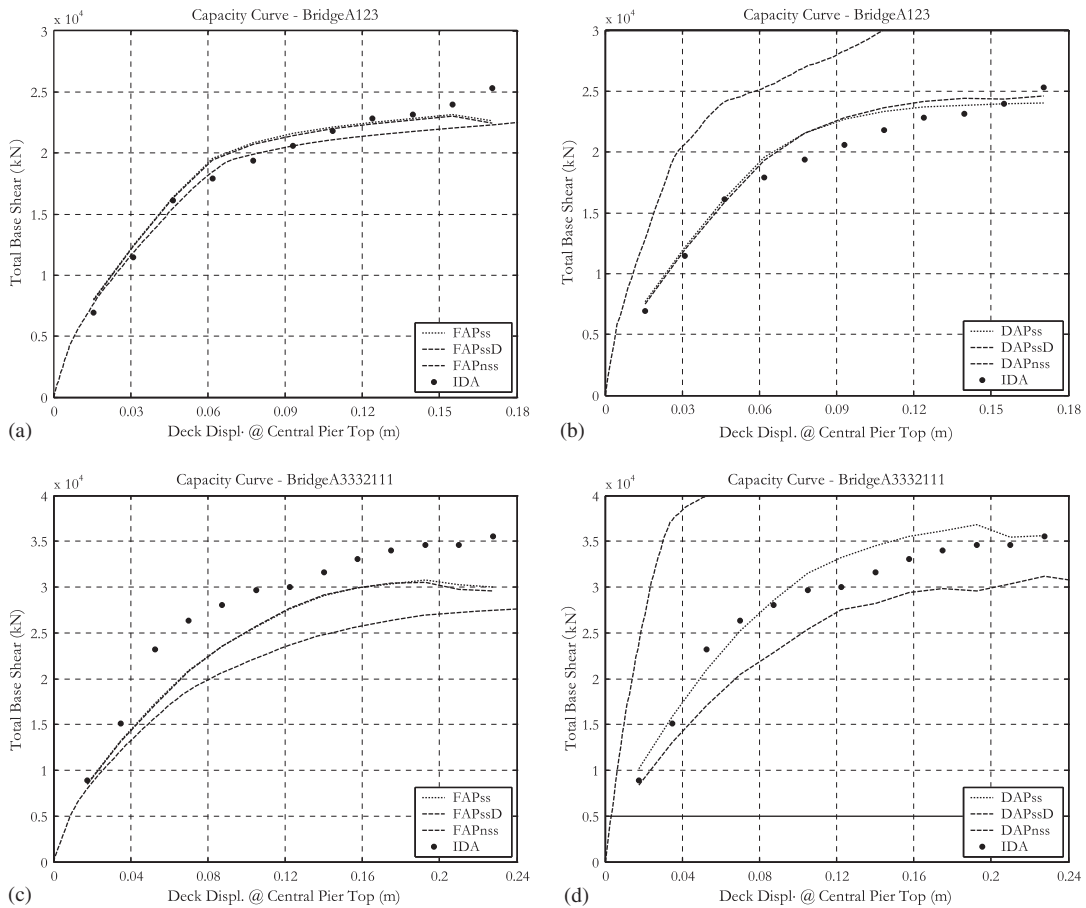


Figure 6. Capacity curve results.

content of a given input motion has on the contributions of different modes: the spectral ordinate at the instantaneous period of each mode is employed to weigh the contribution of such mode to the incremental load shape at any given analysis step (the reader is referred to [19, 30] for further details).

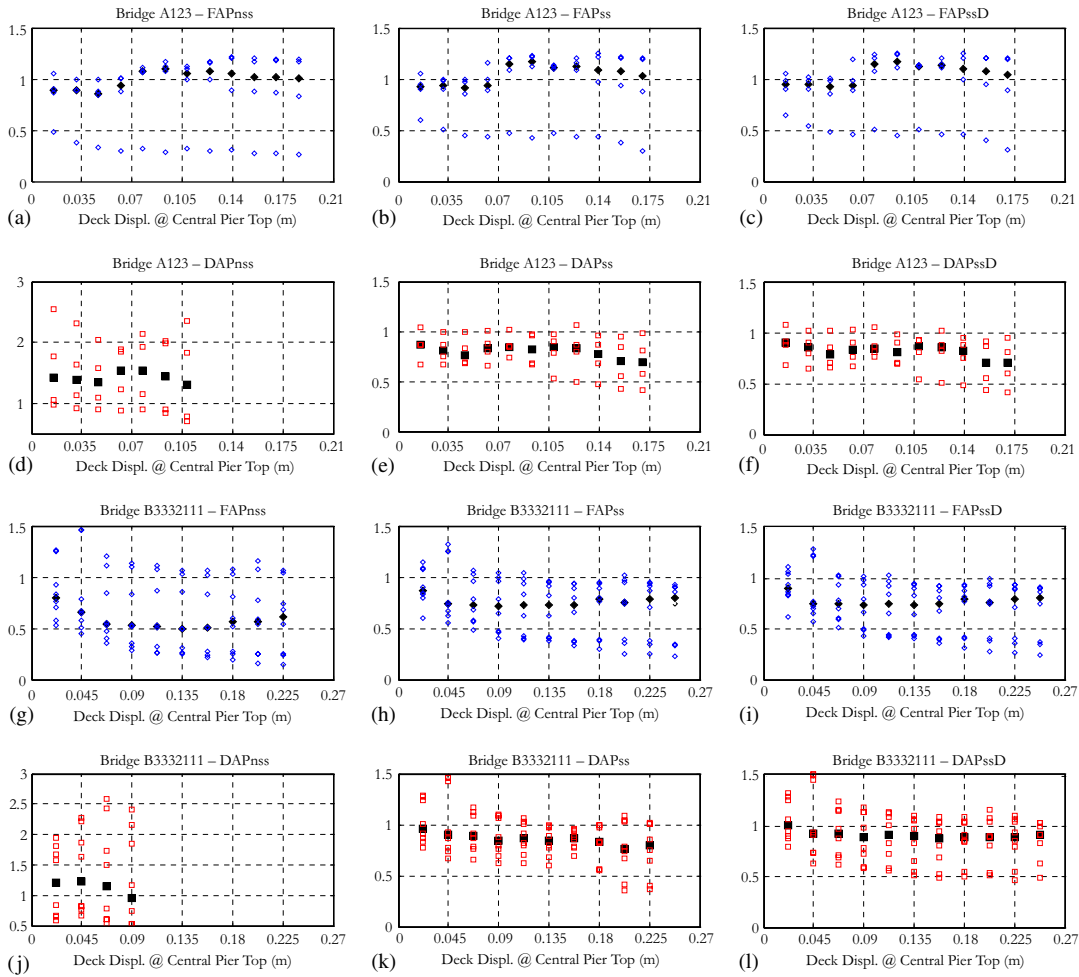


Figure 7. Prediction of the deformed pattern: BI and relative scatter, plotted separately for each pushover type.

Table II. Global averages of the summaries of results for different adaptive methods.

	Bridge index			Dispersion			Normalized base shear		
	Mean	Min	Max	Mean	Min	Max	Mean	Min	Max
FAPnss	0.77	0.67	0.92	0.28	0.20	0.38	0.95	0.85	1.09
FAPss	0.86	0.76	0.99	0.23	0.14	0.35	1.00	0.90	1.12
FAPssD	0.88	0.78	1.01	0.22	0.13	0.34	0.99	0.89	1.10
DAPnss	1.10	0.95	1.22	0.38	0.27	0.49	1.47	1.20	1.82
DAPss	0.84	0.74	0.96	0.18	0.12	0.26	1.08	0.96	1.24
DAPssD	0.87	0.78	0.99	0.19	0.14	0.27	1.03	0.95	1.13

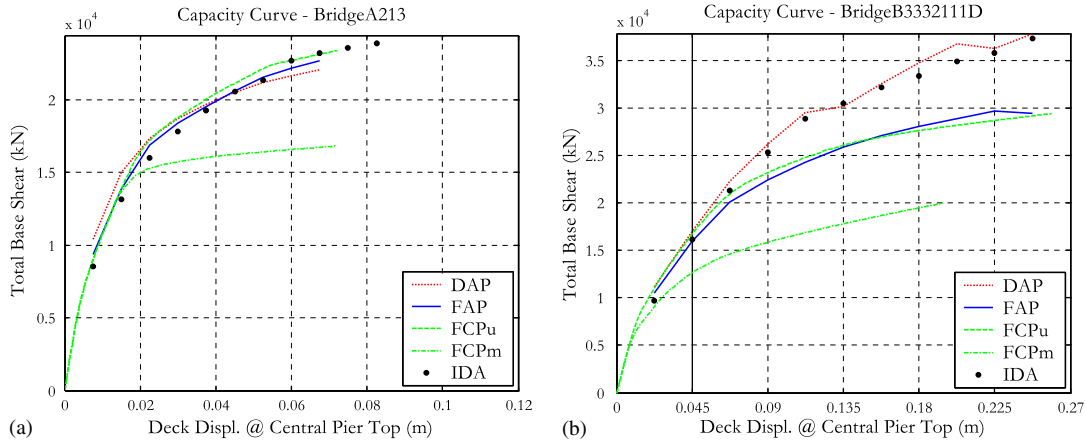


Figure 8. Capacity curve results.

Within the above framework, FAP and DAP analyses have thus been performed with and without spectral scaling and pushing both deck and piers or just the deck, for a total of four types of FAP and DAP. In the following, such analyses will be referred to as FAPnss/DAPnss, FAPss/DAPss, FAPss-D/DAPssD, respectively, with the 'nss'/'ss' suffix standing for the exclusion/inclusion of spectrum scaling, and the 'D' suffix meaning that only the deck is pushed. Results are presented in terms of the bridge capacity curve (reference point displacement *versus* total base shear) and of the deck drift profile.

The following observations were withdrawn from capacity curve plots and the prediction of the total base shear (Figure 6): (i) generally DAPnss strongly overestimates the base shear across the whole deformation range (Figure 6(b) and (d)), (ii) such over-prediction is reduced if spectrum scaling is included (DAPss), particularly if only the deck is pushed (DAPssD); (iii) FAPnss capacity curve is generally quite close to FAPss, even if it is generally lower (Figure 6(a) and (c)).

As for what concerns the prediction of the inelastic displacement pattern, it was noted that the drift profiles obtained from the adaptive algorithms were generally very similar to those obtained from the nonlinear dynamic analyses, with a general trend of underestimation of displacements (Figure 7(e)–(l)), except for DAPnss, that often over-predicts the response and lead also to numerical instability (analysis reproduced in Figure 7(d) and (j) could not be carried out for the entire target displacement range).

In addition to the percentage under/overprediction of the deformed shape, it is important to check the relative scatter, because a good BI estimate associated however to a large scatter means simply that along the deck all predicted displacements are very small or very large with respect to the corresponding IDA results. On the contrary, a less precise prediction of the BI coupled with a lower scatter may indicate a more stable, and thus preferred, estimate of the displacements along the deck. Table II summarizes global averages of means, maximum and minimum values of the BI (with the corresponding dispersion) and of the normalized total base shear, over the entire bridge ensemble. It is noted that (i) DAPnss is the worst pushover option, showing significant scatter and overestimation (especially for base shear estimates), and (ii) the option of pushing just the deck

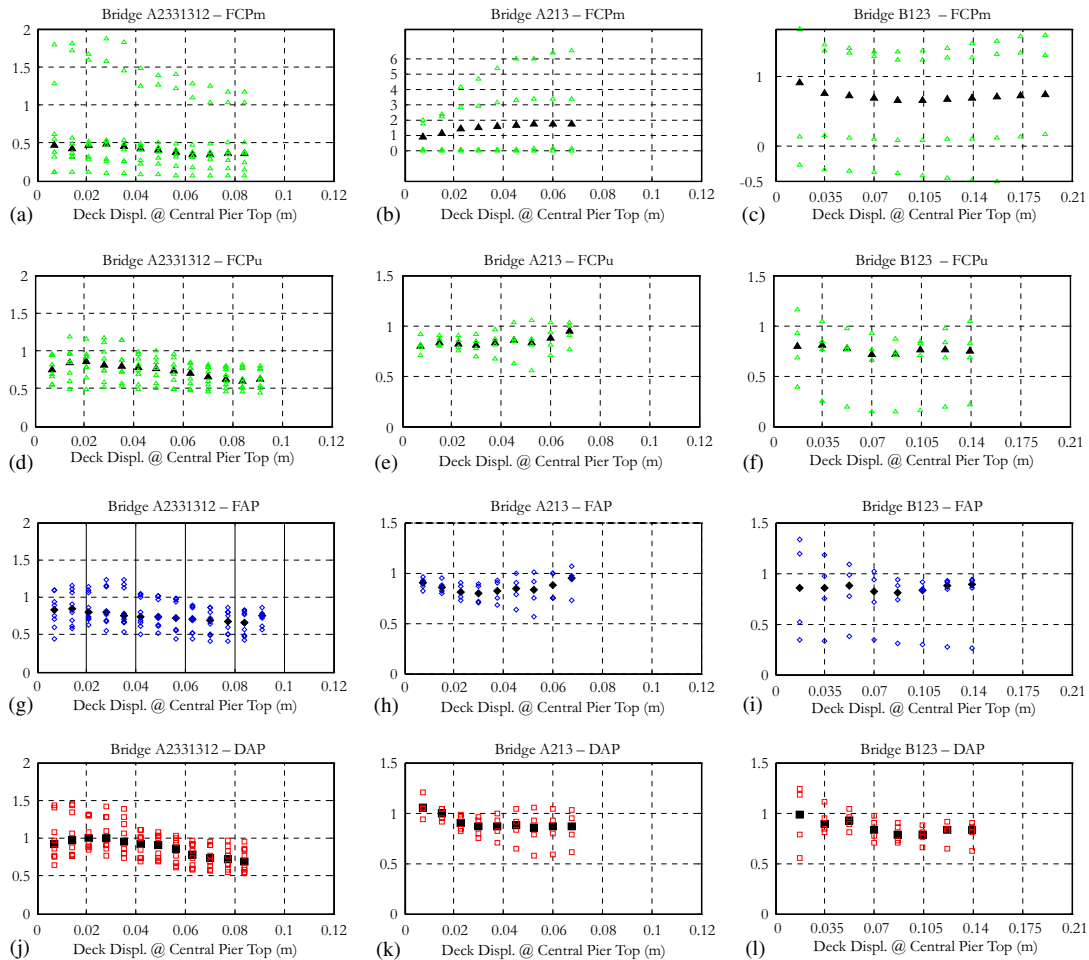


Figure 9. Prediction of the deformed pattern: BI and relative scatter, plotted separately for each pushover type.

affects only marginally the predictions (whilst in terms of stability and velocity of the analyses it proved to be very advantageous).

In conclusion, the best performance in terms of both shear and deformed shape predictions was given by those adaptive techniques that included spectrum scaling and where the loads were applied to the deck alone, this being thus the reason for which such analysis options have been adopted in the subsequent parametric study, where comparisons with conventional pushover procedures are carried out.

5. PARAMETRIC STUDY: RESULTS OBTAINED

A myriad of capacity curve plots, obtained for the different pushover analyses and compared with the IDA envelopes, were derived in this parametric study, the full collection of which can be

Table III. Global averages of the obtained results.

	Bridge index			Dispersion			Normalized base shear		
	Mean	Min	Max	Mean	Min	Max	Mean	Min	Max
<i>Regular</i>									
FCPm	0.67	0.52	0.80	0.34	0.27	0.40	0.81	0.72	0.93
FCPu	1.07	0.91	1.27	0.19	0.12	0.26	1.03	0.92	1.21
FAP	0.93	0.81	1.10	0.19	0.11	0.26	0.94	0.87	1.05
DAP	0.79	0.70	0.89	0.17	0.12	0.22	0.94	0.88	1.02
<i>Semi-regular</i>									
FCPm	0.65	0.55	0.79	0.70	0.60	0.84	0.70	0.62	0.84
FCPu	0.79	0.70	0.94	0.33	0.24	0.51	0.99	0.89	1.14
FAP	0.87	0.78	0.99	0.33	0.21	0.53	0.97	0.87	1.10
DAP	0.83	0.75	0.94	0.22	0.18	0.33	1.03	0.96	1.11
<i>Irregular</i>									
FCPm	0.89	0.64	1.15	1.33	0.86	1.77	0.88	0.74	1.08
FCPu	0.74	0.64	0.87	0.19	0.14	0.25	1.06	0.96	1.17
FAP	0.82	0.74	0.94	0.15	0.08	0.22	1.04	0.94	1.15
DAP	0.98	0.88	1.13	0.19	0.11	0.25	1.13	1.01	1.27

consulted in [26]. Herein, and for reasons of succinctness, only the most pertinent observations, together with some representative plots, are included:

- (i) FCPm tends to underestimate the stiffness of the bridge, mainly due to the fact that, for the same base shear, central deck forces are generally higher compared to the other load patterns, thus resulting in larger displacement at that location; FCPm capacity curve constitutes an evident lower bound, often already in the ‘elastic’ range, where one would, at least in principle, expect a correct prediction of the response (see Figure 8).
- (ii) On occasions, a ‘hardening effect’ in the pushover curve occurs: once piers saturate their capacity, the elastic abutments absorb the additional seismic demand, fully transmitted by the much stiffer and elastic superstructure, thus proportionally increasing shear response and hence ‘hardening’ the capacity curve. This effect, observed also in the dynamic analyses, is sometimes reproduced only by the DAP procedure, as can be observed in Figure 8(b).
- (iii) The adaptive techniques show often capacity curves quite close one to the other, and very close to FCPu (Figure 8(a)). In the elastic range, and as expected, the adaptive capacity curves lie within FCPm and FCPu curve, the same often occurring also in the inelastic range (Figure 8(a)). In some cases, however, adaptive load patterns lie above the FCPu capacity curve.

Examining instead the predictions of inelastic deformation patterns, the aforementioned under-performance of FCPm is further confirmed, with poor predictions of deformed shape and/or large scatter being observed (Figure 9(a)–(c)). On the other hand, the drift profiles obtained with adaptive methods seem to feature the best agreement with those obtained from the nonlinear time-history

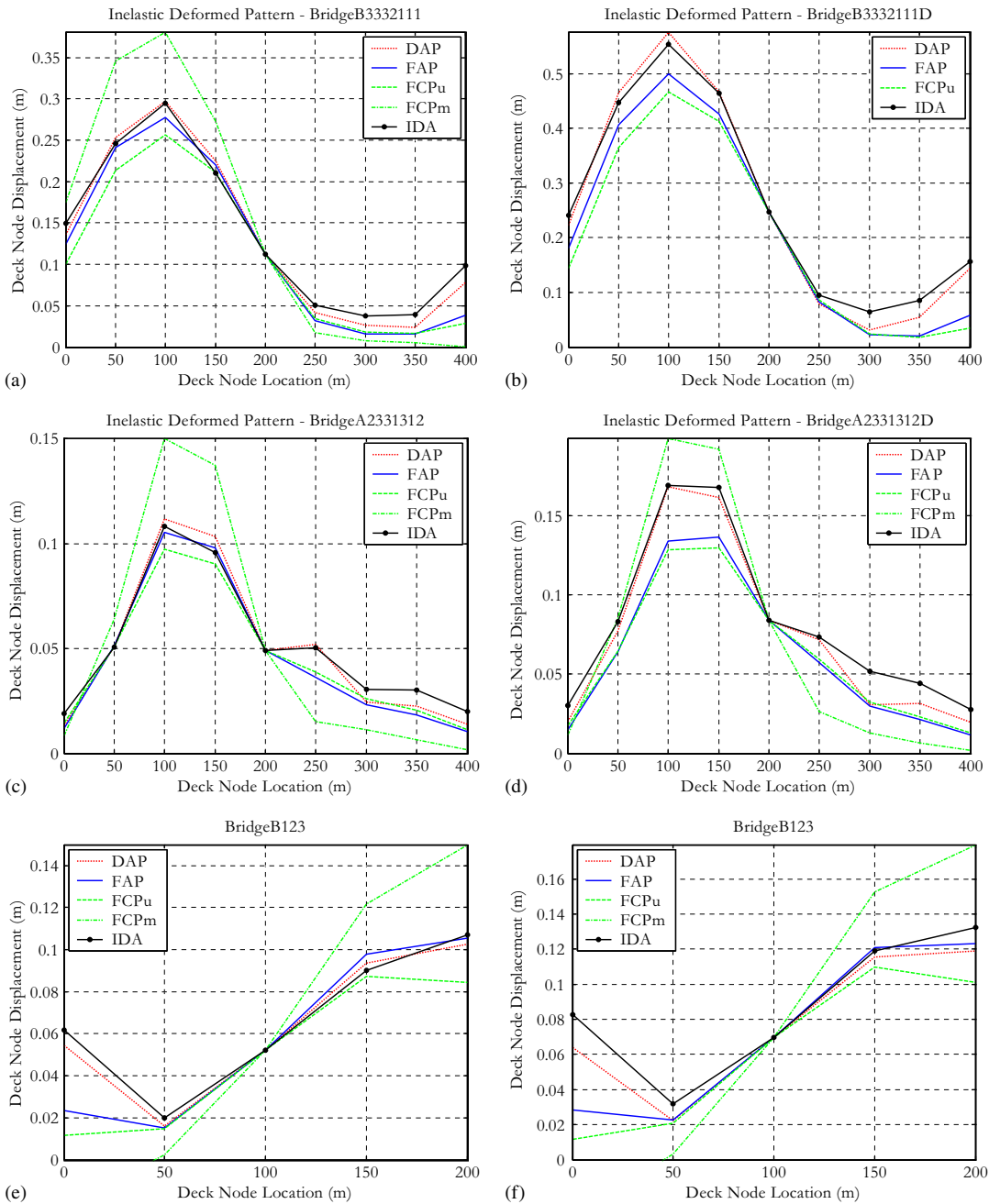


Figure 10. Representative examples of deformed pattern results.

analyses (Figure 9(g)–(l)), with DAP seemingly presenting the lower scatter. These observations may be further confirmed through examination of Table III, where averages of means, maximum and minimum values of the BI (plus corresponding dispersion) as well as of the normalized total base shear, over the entire bridge ensemble, are given. It is noted that:

- (i) FCPm heavily underestimates predictions of both deformed shape and base shear, featuring also excessively high BI dispersion values. This pushover modality is, therefore, in the opinion of the authors, not adequate for seismic assessment of bridges.
- (ii) FCPu performs rather well for regular bridges (it leads to the best predictions in this category), however, its performance worsens considerably as the irregularity of the case-study structures increases.
- (iii) FAP leads to averagely good predictions throughout the entire range of bridge typology (clearly with better results being obtained for regular bridges), noting however that a relatively high value of dispersion is observed in the case of semi-regular configurations.
- (iv) DAP produces also averagely good results throughout the entire set of bridges considered in the study, featuring in particular a very high accuracy in the case of irregular bridges (most regrettably, such high accuracy is conspicuously not present in the case of regular structures). The values of dispersion are very low, independently of bridge regularity.

Finally, and for the sake of completeness, Figure 10 shows the inelastic deformed pattern of three bridge configurations at two different levels of inelasticity, with the objective of rendering somewhat more visual the statistical results discussed previously. It is readily observed how adaptive methods, and particularly DAP, are able to represent the inelastic behaviour of the bridge with a higher level of accuracy, when compared with conventional methods.

6. CONCLUDING REMARKS

In the framework of current performance-based design trends, which require, as a matter of necessity, the availability of simple, yet accurate methods for estimating seismic demand on structures considering their full inelastic behaviour, a study has been carried out to gauge the feasibility of employing single-run pushover analysis for seismic assessment of bridges, which have been so far the object of limited scrutiny, contrary to what is the case of building frames.

Within such investigation, both conventional as well as adaptive pushover methods were used to analyse a suite of bridge configurations subjected to an ensemble of seismic records. It is noted that the bridges feature particularly non-standard shapes, both in terms of pier height distribution (certainly more irregular than what is typically found in the majority of bridges/viaducts), as well as in terms of spans length (typically, the end spans tend to be slightly shorter than their central counterparts). Such bridge configurations were intentionally adopted so as to increase the influence of higher modes in the dynamic response of the structures and in this way place the numerical tools under as tough as possible scrutiny.

The results of this analytical exercise show that, while on the average along regular and irregular configurations some conventional static force-based procedures (namely, using a uniform load distribution pattern) give results comparable to adaptive methods in estimating seismic demand on bridges, the displacement-based variant of the adaptive method associates good predictions in terms of both shear and deformed shape with a reduced scatter in the results. The latter has also

proved to be most effective in representing the ‘hardening effect’ that piers’ capacity saturation can sometime introduce in the capacity curve. In other words, whereas the application of a fixed displacement pattern is a commonly agreed conceptual fallacy, the present work witnesses not only the feasibility of applying an adaptive displacement profile, but also its practical advantages, with respect to other pushover methods.

It is important to note that the inadequacy of using conventional single-run pushover analysis to assess non-regular bridges is already explicitly recognized in Eurocode 8 [28], where it is stated that such analysis is suitable only for bridges which can be ‘reasonably approximated by a generalized one degree of freedom system’. The code then provides additional details on how to identify the cases in which such condition is and is not met, and advises the use of nonlinear time-history analysis for the latter scenarios. The results of the current work seem to indicate that the use of single-run pushover analysis might still be feasible even for such irregular bridge configurations, for as long as a displacement-based adaptive version of the method is employed. The authors feel that the latter could perhaps constitute an ideal alternative or complementary option to (i) the use of nonlinear dynamic analyses, (ii) the adoption of multiple-run pushover methods or (iii) the use of alternative definitions of reference point and force distributions, all of which might also lead to satisfactory response predictions for irregular bridges, as shown in [2].

Finally, it is re-emphasized that the scope of this paper was confined to the verification of the adequacy with which different single-run pushover techniques are able to predict the response of continuous span bridges subjected to transverse earthquake excitation. The employment of such pushover algorithms within the scope of full nonlinear static assessment procedures, such as the N2 method [31, 32] or the Capacity Spectrum Method [33, 34], is discussed elsewhere [35, 36].

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